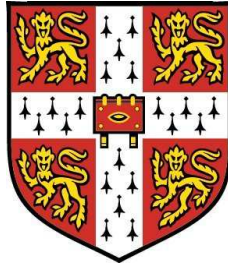


MATERIAL EFFICIENCY IN CONSTRUCTION



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Declaration

This dissertation is the result of my own work and includes nothing which is the outcome of work done in collaboration except where specifically indicated in the text. No part of this dissertation has been or is being submitted for any other qualification at this or any other university.

This dissertation is 44,600 words in length and contains 52 figures. This is within limits set by the Degree Committee.

Muiris Moynihan

2014

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Publications list

Parts of this thesis have been published in journals and presented at conferences, and have contributed to several reports and a book. A full list of these publications is given here.

Published journal articles

1. **Moynihan, M.C.** & Allwood, J.M. 2012. The flow of steel into the construction sector. *Resources, Conservation and Recycling*, 68:88–95.
2. **Moynihan, M.C.** & Allwood, J.M. 2014. Viability and performance of demountable composite connectors. *Journal of Constructional Steel Research*, 99:47–56.
3. **Moynihan, M.C.** & Allwood, J.M. 2014. Utilisation of structural steelwork in buildings. *Proceedings of the Royal Society A*, 470:20140170.
4. Cooper, D.R., Skelton, A.C.H., **Moynihan, M.C.** and Allwood, J.M. 2013. Component level strategies for exploiting the lifespan of steel in products. *Resources, Conservation and Recycling*, 84:24–43.
5. Carruth, M.A., Allwood, J.M. and **Moynihan, M.C.** 2011. The technical potential for reducing metal requirements through lightweight product design. *Resources, Conservation and Recycling*, 57:48–60.

Books

1. Allwood, J.M., Cullen, J.M., Carruth, M.A., Cooper, D.R., McBrien, M., Milford, R.L., **Moynihan, M.C.**, Patel, A.C.H. 2012. Sustainable Materials: With Both Eyes Open. UIT Press, Cambridge, UK. ISBN 978-1-906860-05-9.

Conference Papers

1. **Moynihan, M.C.** & Allwood, J.M. 2012. Deconstruction and reuse: realities of design, commerce and logistics for portal frames. *Proceedings of the First International Conference on Performance-based and Life-cycle Structural Engineering*, 5–7 December 2012, Hong Kong, 1379–1387. Faculty of Construction and Environment & Research Institute for Sustainable Urban Development, The Hong Kong Polytechnic University, Hong Kong, China. ISBN 978-988-15439-3-6
2. **Moynihan, M.C.** & Allwood, J.M. 2013. Viability and performance of demountable composite connectors. *Proceedings of the 2013 World Congress on Advances in Structural Engineering and Mechanics*, 8–12 September 2013, Jeju, South Korea, 3215–3227, 2013. Techno-press, Daejeon, Korea. ISBN 978-89-89693-37-6-93530

Other Publications

1. Cullen, J.M., Carruth, M.A., **Moynihan, M.C.**, Allwood, J.M. and Epstein, D. 2012. Reducing embodied carbon through efficient design. Learning Legacy, London 2012 Olympics.
2. Allwood, J.M., Cullen, J.M., Carruth, M.A., Milford, R.L., Patel, A.C.H., **Moynihan, M.C.**, Cooper, D.R. and McBrien, M. 2010. Going on a metal diet — using less liquid metal to deliver the same services in order to save energy and carbon. University of Cambridge, Cambridge, UK. ISBN 978-0-903428-32-3
3. Allwood, J.M., Cullen, J.M., Patel, A.C.H., Cooper, D.R., **Moynihan, M.C.**, Milford, R.L., Carruth, M.A. and McBrien, M. 2011. Prolonging our metal life — making the most of our metal services. University of Cambridge, Cambridge, UK. ISBN 978-0-903428-33-0.
4. Allwood, J.M., Cullen, J.M., McBrien, M., Milford, R.L., Carruth, M.A., Patel, A.C.H., Cooper, D.R. and **Moynihan, M.C.** 2011. Taking our metal temperature — energy and carbon savings by managing heat in steel and aluminium supply chains. University of Cambridge, Cambridge, UK. ISBN 978-0-903428-34-7.

Abstract

Producing steel causes 6% of global anthropogenic carbon dioxide emissions. Experts recommend that these emissions are reduced by half by the year 2050 in order to avert the worst consequences of climate change. Demand for steel is predicted to double in the next 36 years, meaning that a 75% reduction in emissions per unit of steel produced is necessary to reach the recommended limit. Process efficiency improvements cannot deliver this magnitude of reduction; however if steel is used more efficiently so that less new material is required to deliver the same service — a concept termed ‘material efficiency’ — then this could allow demand to be satisfied whilst emissions targets are achieved.

Construction is the single largest use of steel globally, therefore using steel more efficiently in construction will reduce emissions. Three material efficiency strategies are identified as having most potential for this industry: using less material, using products for longer, and reusing components. In order to prioritise areas for research, steel flows into construction are mapped, finding that industrial buildings and utility infrastructure are the largest users of steel, while superstructure is confirmed as the main use of steel in a typical building.

To estimate the potential to use less steel in buildings, 23 steel-frame designs are studied, sourced from three leading design consultancies. The utilisation of each element is found and the building datasets are analysed to infer the amount of steel over-provided. The results suggest that such buildings contain almost twice as much steel as necessary for structural performance, and indicate that this amount of over-provision occurs to minimise labour costs, which are a larger proportion of total costs than materials.

To investigate how buildings and infrastructure could be used for longer, reasons for their failure are reviewed. Based on interviews with industry professionals a set of strategies is proposed, tailored to each failure

cause and distinguishing between cases where failure can and cannot be reasonably foreseen.

Steel sections could be reclaimed from old buildings and reused in new buildings but this does not occur because they are damaged during demolition. Designing for deconstruction would facilitate reuse but is not practised due to its cost. Data from interviews and a commercial working group are analysed to identify three aspects of designing for deconstruction that provide financial and operational benefits to clients, thus encouraging their use.

One remaining technical barrier to deconstruction is composite steel-concrete systems, where welded connectors make it impractical to separate the steel beam from the concrete slab without damage. A novel bolted composite connector is proposed and tested in three beam experiments. The bolted connector allows successful separation of the components, facilitating reuse. Its structural performance is similar to that of welded connectors and can be predicted using current design standards.

Each of the investigations reveals significant opportunities to reduce steel use in construction by using material more efficiently. Achieving these savings would reduce demand for new steel production and thereby decrease carbon dioxide emissions.

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Nomenclature

Roman Symbols

CO_{2e} Carbon dioxide equivalent

CO_2 Carbon dioxide

BCSA British Constructional Steelwork Association

BREEAM Building Research Establishment Environmental Assessment Method

BSI British Standards Institute

CCS Carbon capture and storage

DfD Design for deconstruction

ECCS European Convention for Constructional Steelwork

EMGT Equivalent million gross tonnes

GHG Greenhouse gas

Gt Gigatonne

IPCC Intergovernmental Panel on Climate Change

ISSB International Steel Statistics Bureau

JISF Japan Iron and Steel Federation

kt Kilotonne

LCA Life-cycle assessment

MFA Mass flow analysis

Mt Megatonne

Nomenclature

rebar Reinforcement bar

RuFUS Reuse of Foundations on Urban Sites

SCI Steel Construction Institute

t Tonne

U/R Utilisation ratio

VAT Value added tax

Chapter 1

Introduction: carbon and steel in construction

The construction industry has transformed the human habitat, using natural resources to build an environment that allows record numbers of people to live in comfortable, secure homes and work in ever-larger cities, serviced by increasingly efficient and extended infrastructure networks. It is unimaginable that transformation of this scale could have occurred without having a major impact on the planet — therefore it is unsurprising that producing the vast quantities of energy and material that constitute the modern built environment is impacting negatively on the planet’s ecosystem. In order to protect future generations from the worst outcomes of this impact, this generation must change its use of energy and materials.

1.1 The need to reduce carbon dioxide emissions

The Fifth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC) states as “unequivocal” that the Earth’s atmosphere and oceans have warmed in recent decades; this is thought to be causing the world’s climate to change (Stocker et al., 2013). The IPCC report attributes global warming to increases in greenhouse gases (GHG), of which carbon dioxide has the largest cumulative impact; it notes that “atmospheric concentrations of carbon dioxide (CO₂)... have increased by 40% since pre-industrial times, primarily from fossil fuel emissions”. While the IPCC acknowledges that the effects of climate change are uncertain, it predicts that sea levels will rise and that heat waves, large storms and other extreme weather events will become more likely. These effects could negatively impact upon much of the world’s population through decreased food production, damage to infrastructure

and diminished economic development. The IPCC warns that “[c]ontinued emissions of greenhouse gases will cause further warming and changes in all components of the climate system”. To avert the worst consequences of climate change, the IPCC recommends “substantial and sustained reductions of greenhouse gas emissions” of between 50% and 80% compared to 2000 levels by 2050 (Metz et al., 2007). These targets are being translated into law by national governments, for example the United Kingdom legislates for an 80% reduction in annual emissions by 2050 relative to 1990 levels (United Kingdom, 2008).

1.2 The contribution of construction to carbon dioxide emissions

Figure 1.1 shows three pie-charts of global anthropogenic carbon dioxide emissions for the year 2005, taken from Allwood et al. (2012) (with data from the International Energy Agency). Figure 1.1a shows that the majority of emissions are from energy generation (overwhelmingly by combusting fossil fuels) and industrial processes rather than land use changes. Figure 1.1b shows that these energy and process emissions are divided in three categories: energy use in buildings (mainly heating and cooling space and water (Blok et al., 2007)), transporting things and people, and making products in industry. Figure 1.1c shows that producing just five materials accounts for over half of industrial emissions and that steel is the largest of these, followed by cement.

1.2 The contribution of construction to carbon dioxide emissions

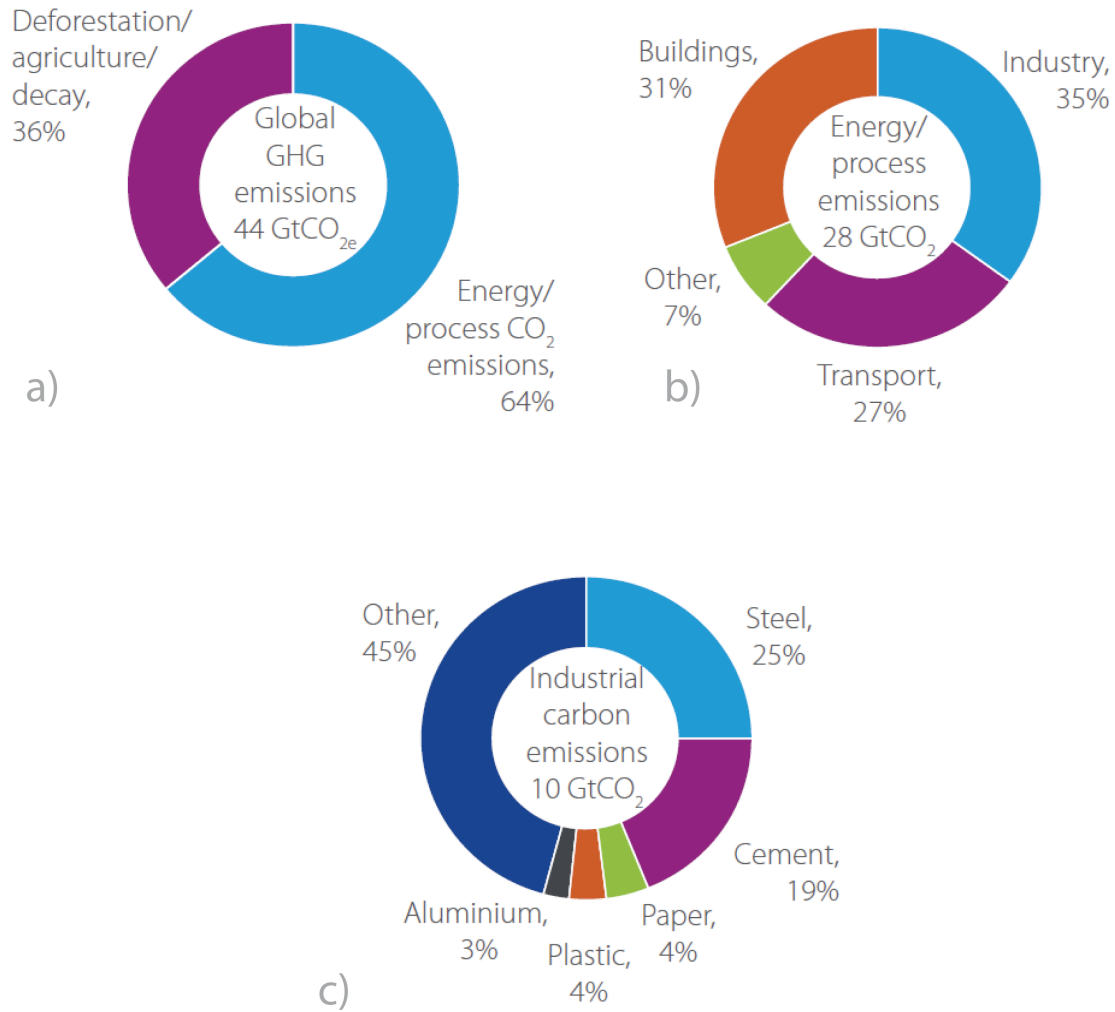


Figure 1.1: Global anthropogenic carbon dioxide emissions (a) for all anthropogenic activity, (b) related to energy and processes, (c) for industry. From Allwood et al. (2012)

Notes: CO_{2e} is ‘carbon dioxide equivalent’ — found by converting the global-warming effects of other gases into units equivalent to CO₂. Units of gigatonnes (Gt) are used; 1 Gt = 10⁹ tonnes.

The construction industry contributes directly or indirectly to the emissions described in figure 1.1b: it constructs the buildings that require heating and cooling, it provides the infrastructure that vehicles travel on or to, and it uses products from industry to do both of these. In fact construction is the single largest use of steel and cement, with half of the 1,000 megatonnes (Mt) of steel entering global society each year finding use in buildings or infrastructure (Wang et al., 2007) along with all of the 2,800 Mt of cement produced annually (Allwood et al., 2012; USGS, 2010). Therefore changes in how the industry constructs buildings and infrastructure, and uses materials to do both, will have a significant impact on CO₂ emissions.

1.3 Options to reduce carbon dioxide emissions

Given that CO₂ emissions should be reduced by at least 50% within the next 36 years, what options are there in each of the three categories of energy and process emissions? And how can the construction industry contribute to each?

1.3.1 Options to reduce energy use in buildings and in transport

Most of the emissions from buildings and transport in figure 1.1b are directly due to the combustion of fossil fuels to provide energy for heating/cooling space/water (in buildings) or for propelling vehicles (for transport) (International Energy Agency, 2008). Therefore reducing energy use in these categories will correlate directly with emissions reductions; current efforts aim to do this by increasing energy efficiency. Examples of best available technology demonstrate the large potential for emissions reduction in the buildings and transport categories: the German ‘passivhaus’ house design standard requires 80% less energy annually than standard UK houses (McLeod et al., 2012); the world record for car fuel efficiency is 5,000 kilometres per litre (Brown, 2006), compared with 12km/l achieved by a typical car in the UK (MacKay, 2008). IPCC reports on mitigation strategies confirm the scale of reduction possible: Blok et al. (2007) review 80 studies on emissions reduction potential in buildings and conclude that “substantial reductions in CO₂ emissions from energy use in buildings can be achieved over the coming years using mature technologies [such as more efficient boilers and insulation] for energy efficiency”, calculating that reductions of 27% can be achieved “cost-effectively” and that some areas have potential for a 70–80% decrease in emissions; Kahn Ribeiro et al. (2007) state that “[i]mproving energy efficiency offers an excellent opportunity” to decrease emissions and they forecast potential reductions of 50% for certain vehicles by 2030, with up to 20% further reductions possible through improved vehicle maintenance and better traffic management.

How can construction contribute to emissions reductions in these categories? Designing and building structures that require less energy in use is an obvious step; correspondingly the European Union requires that new buildings become increasingly energy-efficient so they are ‘nearly zero energy’ in use (EU, 2010), i.e. producing almost as much energy as they consume annually (Banfill and Peacock, 2007). A number of authors, such as Ewing and Cervero (2010), suggest that construction can

reduce transport emissions also by designing and building cities with high-density residential and employment areas in close proximity.

1.3.2 Options to reduce emissions from materials production

Is there the same potential to reduce emissions in the largest category, industry? Allwood et al. (2010b) predict that demand will approximately double for steel and cement — the materials used most in construction — between 2006 and 2050 due to increasing population and economic prosperity in developing nations. When combined with the IPCC emissions reductions target, meeting this increase in demand necessitates a reduction of 75% in emissions per tonne of steel and cement produced. How can this reduction be achieved? Three options are discussed: implementing process technology improvements, generating low carbon energy, and using different materials.

Process technology improvements

Can existing or emerging improvements in materials production technologies deliver large reductions in emissions per tonne? Worrell et al. (2008) report that the majority of energy required to produce steel and cement is used to make the liquid metal and clinker (unground cement) respectively. As energy constitutes about one-third of production costs for both steel (Allwood et al., 2012) and cement (Lafarge, 2007), both industries have been financially motivated to become more energy-efficient over their histories. Therefore it is not surprising that Allwood et al. (2010b) estimate possible reductions, incorporating all existing and emerging technology improvements, of just 34% and 40% in steel and cement production emissions per tonne respectively. This is consistent with Gutowski et al. (2013)'s assertion that major improvements in energy efficiency in these industries are unlikely, in part due to thermodynamic limits.

Low carbon energy

Three options for low carbon energy exist which would allow material production to increase while emissions decrease: carbon capture and storage (CCS), low-carbon energy and fuel substitution. However all three options face substantial challenges to be implemented at the scale required to significantly reduce emissions by 2050:

1. INTRODUCTION: CARBON AND STEEL IN CONSTRUCTION

- Smil (2010) describes the substantial logistical and cost challenges of constructing a CCS infrastructure at the scale required in the next 36 years, while Global CCS Institute (2012) report that the number of planned and installed CCS facilities is already an order of magnitude lower than required to meet 2020 emissions reductions targets;
- MacKay (2008) performs simple calculations to show that renewable energy facilities would have to be “country-sized” in order to generate energy at the scale of current consumption. MacKay also observes that nuclear fission offers lower carbon energy but that it is a politically-controversial solution and remarks that it is “reckless” to assume nuclear fusion will become viable;
- Industrial and domestic solid waste can be combusted in a cement kiln, substituting fossil fuel and thus reducing emissions. Waste displaces only 17% (on average) of fuel in European cement-making currently (IEA, 2007) as it requires treatment beforehand and causes social concerns about toxic emissions (WBCSD and IEA, 2009). Additionally, fuel substitution cannot reduce the emissions from the chemical reactions (primarily converting limestone into lime) which account for half of emissions due to cement production (Rehan and Nehdi, 2005).

Using different materials

Could we use other materials instead of steel and cement which cause fewer CO₂ emissions? Performance plots (called ‘Ashby charts’) of material properties against embodied energy (the sum of energy used to produce the material) reveal that timber and stone could be lower-emission substitutes for steel and concrete respectively (Allwood et al., 2011); indeed these were the materials used prior to the advent of steel and concrete. However timber and stone are more difficult to use and have other disadvantages: stone cannot be moulded, transported or reinforced as easily as concrete; timber is less stiff than steel and not as strong (meaning more timber is needed to achieve the same performance), as well as being anisotropic and more vulnerable to fire. Thus while timber and stone will continue to be used in construction, it is unlikely they can be substituted for the millions of tonnes of steel and cement used each year.

What about more modern materials? ‘Advanced’ materials such as carbon-fibre reinforced plastic have strength and stiffness similar to steel, but they cannot be recycled and Ashby (2009) shows that such materials have greater embodied energy.

Fly ash, blast furnace slag and pozzolanic materials are widely substituted into concrete to partially reduce cement content, however combined annual production of these materials is 850 Mt (WBCSD and IEA, 2009), less than one-third of cement demand. WBCSD and IEA (2009) describe claims that novel cements can be made with significantly less emissions, or even can absorb CO₂ — such as Novacem, a proprietary technology based on magnesium silicate. Gartner (2012) reviews such cements, finding a lack of verifiable information about them and citing concerns about their durability. Gartner’s concerns are echoed by WBCSD and IEA (2009), who state that novel cement technologies have not been “tested at scale for their long-term suitability”.

1.3.3 Assessing options

As outlined in section 1.3.1, there appear to be sufficient opportunities to reduce emissions from the buildings and transport categories to make significant progress towards the IPCC’s emissions reduction target. Is the same true for the industrial category? Allwood et al. (2012) complete a robust analysis of the options listed in section 1.3.2 for steel production, and concise analysis for concrete production. They conclude that steel’s emissions can remain constant while cement’s will increase by around 20%, assuming production of both approximately doubles. Whilst these represent substantial reductions in emissions per tonne, they are still not sufficient to meet the IPCC’s emissions targets. This research therefore explores further options to reduce emissions from materials production.

1.4 Strategies to reduce material demand in construction

Given industrial CO₂ emissions targets cannot be met solely by improvements to processes, low carbon energy or substituting materials, what other options exist? Allwood et al. (2010b) propose ‘material efficiency’ — reducing the amount of material produced, while still providing the same service — as another method to decrease CO₂ emissions. Allwood et al. (2012) outline six material efficiency strategies:

- reducing yield losses during the production of products;
- diverting manufacturing scrap;

- reducing final demand;
- using less material by design;
- using products for longer;
- reusing components.

Which of the six material efficiency strategies have most impact on construction materials? Hatayama et al. (2010) report that steel products used in construction are made with only 6% yield loss (meaning little manufacturing scrap is created), whilst concrete is cast with almost no waste. Thus there is little saving to be made by implementing the first two material efficiency strategies for construction products. The third strategy, artificially ‘reducing final demand’ (i.e. rationing materials) is an option of last resort — material use has been successfully rationed during emergencies in the past but it results in reduced service to society, and thus is not a desirable outcome and is not discussed further. The three remaining strategies — using less by design, using products for longer, reusing components — could reduce demand for construction materials and therefore are investigated further.

Steel and concrete are both used in vast tonnages in construction, and both cause substantial fractions of industrial CO₂ emissions, thus research is merited on either material to establish how material efficiency strategies can reduce demand. This research chooses to focus on steel.

Two advantages from the choice of steel are: the steel industry is more centralised, meaning that more information is available on the production and use of steel currently and in the past; this research was inspired by, and done alongside, the Well-Met2050 project whose remit was to examine material efficiency in steel and aluminium, thus focusing on steel in construction benefited from the knowledge and contacts already gained by the WellMet2050 team.

1.5 Research objectives

The aim of this research is to reduce carbon dioxide emissions from steel production by reducing demand for new steel from the construction industry. Three strategies have been identified as having most potential to do so: using less steel, using steel products for longer, reusing more steel — the objective of this research is to identify and assess opportunities for the implementation of these strategies.

1.6 Thesis structure

This thesis contains seven chapters and explores different aspects of material efficiency for steel in the construction industry.

Chapter 2 reviews the previously published literature on steel use in construction, identifying four gaps in the knowledge base.

Before specific opportunities for material efficiency could be investigated, it first had to be understood how steel is used in construction — the distribution of components between and within different building and infrastructure types. Chapter 3 describes how this analysis was undertaken, charting the destination of the steel used in construction annually and identifying a ‘typical’ structure.

The following three chapters examine the potential in construction for the three selected material efficiency strategies:

The results of chapter 3 indicate that the majority of steel is used in buildings. To investigate how much steel demand could be reduced by **using less** in a typical building design, chapter 4 describes the analysis of structural steelwork data to infer the tonnage of steel over-provided.

Chapter 5 proposes a set of design strategies that allow structures to be **used for longer**, tailored to the reasons for end-of-life. Where it is not possible to use entire structures for longer, it may be possible to **reuse components**, such as steel sections, if they are removed undamaged. Designing for deconstruction makes this achievable but is not practised due to cost. Chapter 5 investigates commercial advantages of designing for deconstruction which will encourage its occurrence and thereby enable reuse.

Composite structures are difficult to reuse; chapter 6 describes laboratory testing of a novel, demountable composite connector that enables this structure type to be reusable.

Chapter 7 outlines the key findings of this research and discusses opportunities for future work.

1. INTRODUCTION: CARBON AND STEEL IN CONSTRUCTION

Chapter 2

Review of published literature on steel use in construction

In order to identify and evaluate opportunities to use steel more efficiently in construction, an understanding is required of how steel is currently used in the industry and of the knowledge gained from previous research. This chapter reviews published literature on each of the four topics identified in section 1.6:

- Section 2.1 reviews published literature quantifying the flows of steel into the construction industry;
- Section 2.2 reviews publications on the provision of structural steel in buildings over the past century, identifying changes in design guidance that influence the amount of material used;
- Section 2.3 reviews reasons for failure of buildings and infrastructure, and strategies to overcome them. Literature on reuse and deconstruction is also reviewed, investigating why the significant potential to do both in construction is not realised;
- Composite construction is identified as one of the barriers to deconstruction, so section 2.4 examines previous research on composite connectors and efforts to develop deconstructable connectors.

Section 2.5 summarises the gaps in published knowledge found in each of the previous sections; these in turn form the starting point for the research described in chapters 3 to 6.

2.1 Review of published literature on steel flows into construction

To have most impact, material efficiency strategies should be applied to the largest uses of steel in construction — the types of structures most prolifically built, the structures most representative of a ‘typical’ construction, and the products most widely employed. Three bodies of knowledge are reviewed to identify these uses: analyses of anthropogenic steel flows (section 2.1.1), industry literature on typical buildings (section 2.1.2) and studies of materials in the building stock (section 2.1.3).

2.1.1 Published accounts of current steel flows

Published studies of current steel flows into construction have all been top-down mass flow analyses (MFAs), mapping movements of steel from production to their final use (termed ‘end-use’) for a given year, both nationally and internationally. Two institutions, the World Steel Association (worldsteel) and the International Steel Statistics Bureau (ISSB), publish total steel tonnages consumed per country, grouped in broad product categories. Data from worldsteel are compiled from its members — the major steel producers and national trade associations — in year-books such as worldsteel (2011a). The worldsteel publications list product tonnages produced by country, with broadly aggregated values for consumption, and require comparative analysis with trade data to determine product demand of a specific nation. The ISSB maintains records on international steel trade and hence is able to calculate values for product supply/demand/apparent consumption in a country. However, its publications list aggregated data, and lack detail on specific products.

Two estimates of the proportion of global steel flowing into construction have been made: worldsteel (2008) estimate that approximately half of steel worldwide is used in construction; Wang et al. (2007) study the ‘anthropogenic iron cycle’ and allocate seven intermediate product types to five broad use categories, to produce a similar result. Neither of these contains breakdowns by product or by sector, nor do the national estimates of steel flow into construction. Dahlström et al. (2004) study iron and steel use in the UK for 2001, concluding that 26% is used in construction. This agrees with UK Steel (2010), an industry publication based on ISSB data, which states that 27% of steel is consumed by construction. Two studies examine the UK construction industry alone: Smith et al. (2002) look at all materials consumed by the industry, using government statistics to arrive at a steel tonnage similar to

2.1 Review of published literature on steel flows into construction

Dahlström et al.; Ley (2003) conducts a more detailed study on iron and steel use in construction only, but concludes that it uses a lower proportion, 21%, of national steel consumption. The Japan Iron and Steel Federation (JISF) publish annual steel use statistics (Japan Iron and Steel Federation, 2011), which studies such as Hatayama et al. (2010) have used for MFAs. Müller et al. (2011) split steel flows into four sectors for six developed countries based on a ‘product-to-use matrix’, finding that construction accounts for between 25% and 50% of total steel use in these countries. Pauliuk et al. (2012) examine China’s steel consumption, noting that fully half of it is used in construction. While these articles confirm the proportion of the total consumption used by construction, data indicating use within construction are scarce; the only such breakdown is provided by Hu et al. (2010a), based on statistics from the Chinese government. To improve upon this single data point, and add more detail, further research is therefore required.

2.1.2 Published sources on steel use in a typical structure

No literature could be found identifying a ‘typical’ structure, instead publications on typical buildings and infrastructure were reviewed separately to ascertain their steel contents.

A direct study has yet to be published on the distribution of steel within a ‘typical’ building; however there are three types of source which provide some of the necessary information: cost models; design guides; case studies. Cost models are frequently published by trade associations or industry magazines to compare options or update professionals on current practice. These models can contain itemised lists of components, from which data on steel use can be inferred — Goodchild (1993) and Concrete Centre (2011) are examples of this. Design guides are used by professionals to produce early-stage outlines of projects, including costs. Practising engineers use handbooks such as Arup (2008) to convert rough designs into material quantities; while quantity surveyors use books such as Davis Langdon (2010) to price construction projects, which also contain limited steel intensity information. Construction case studies are published for various reasons, but can provide data on steel use: the ‘Target Zero’ reports (e.g. Target Zero, 2011) from the British Constructional Steelwork Association (BCSA) focus on energy use and carbon, but include chapters with structural steelwork quantities; Goggins et al. (2010) study embodied energy in concrete yet report reinforcement tonnages also.

Published information on steel use within infrastructure has focused on specific types of infrastructure, for example Oh et al. (2013) provides steel quantities and

costs for a steel box-girder bridge but do not consider other bridge constructions nor other infrastructure sectors. Similarly, design guidance (e.g. BSI (2000a) for steel bridges) and case studies (e.g. Dilley (1994)’s description of an airport structure) are all specific to infrastructure applications, indicating the difficulty in defining a typical installation.

While industry publications provide limited information on the distribution of steel within individual building and infrastructure applications, no study has yet published the steel content of a typical structure. Therefore research is required to identify such a structure and to estimate the steel it contains.

2.1.3 Published estimates of steel in the construction stock

Stocks of steel in buildings and infrastructure — i.e. the steel accumulated in society as structures are built over time — have been studied using both top-down and bottom-up methods. The limitations of these approaches are noted in the literature: the top-down studies rely on life-span estimates and lack detail; the bottom-up studies are limited to small areas due to time and labour constraints.

A dynamic MFA is a top-down method that compares input and output flows across a boundary over time, thereby computing the stocks built up within the boundary (usually a country). While inward steel flows can readily be found from the above steel production/consumption sources, outflows are not centrally recorded, and must be estimated either through discard rates (where available) or from lifespan estimates. Müller et al. (2011) perform such an analysis on six countries, allocating the flows into four broad categories, the largest of which is construction. Hatayama et al. (2010) complete a similar analysis for the world, and for each continent, but go on to provide a breakdown between buildings and ‘civil engineering’ (i.e. infrastructure) within construction. Even though Ley (2003) completes a dynamic MFA to calculate a tonnage for UK construction stocks only, he does not provide any detail on the types of structures that contain this steel, nor do Michaelis and Jackson (2000) in their MFA of the UK steel stock over 40 years.

Conventional bottom-up studies of steel stocks have taken place in Europe and the state of Connecticut, USA, while an innovative approach has been tried in east Asia. However the information captured in each is of limited interest. Bruhns et al. (2000) create a model for the British non-domestic building stock based on detailed surveys and national statistics, but as their focus is on building energy use, they do not record sufficient structural information to allow estimates of steel content. Kohler

2.1 Review of published literature on steel flows into construction

and Hassler (2002) outline similar work in Germany but do not report steel contents, nor does Müller (2006) in his study of the Dutch residential stock. Drakonakis et al. (2007) estimate iron and steel stocks in the city of New Haven, Connecticut, USA, by multiplying steel intensities (kg/m^2) for different building and component types with relevant data on their square meterage. The intensities are formulated from design ‘rules of thumb’ and conversations with engineers, and are assumed by the authors to have error margins of $\pm 30\%$. As part of the same research, Eckelman et al. (2007) produce a working paper estimating the stock of steel within the entire state of Connecticut by similar methods. Both sets of authors point out the limitations to their methods and the factors which prevent their results being reliably scaled for estimates elsewhere; however these studies are the only data sources which reveal in detail what types of buildings and infrastructure use the most steel, and what the main applications of steel are within these structures. A novel approach, taken by Hsu et al. (2010), is to use light emission as a proxy for steel stocks, so that by analysing satellite images of countries their steel stocks can be estimated. This is correlated using JISF data for steel stocks, achieving a distinction between building and infrastructure stocks.

Smith et al. (2002) highlight that the building stock is not homogeneous, having been built in a decentralised way over 200 years during which technologies, materials and fashions have all changed. Therefore they conclude it is impossible to accurately verify estimates about the building stock, such as the steel contained in it. Also, as stock data indicates historic steel use rather than current steel use in construction, studies of stocks are less useful for identifying opportunities for material efficiency than studies of current flows.

2.1.4 Findings from literature review on steel flow into construction

Published literature does not contain the knowledge required to appraise the material efficiency potential of steel in construction. Previous analyses of anthropogenic steel flows simply allocate annual tonnages to ‘construction’ as a whole, and although there is good agreement between sources on the proportion of steel entering the industry, detail is scarce on the dominant uses within the industry. Industry publications provide some information on the distribution of steel within an individual building and for specific infrastructure applications, however because none of the sources were compiled specifically to quantify steel use, gaps and limitations within the data render their usefulness minimal without further information and

analyses. Studies of stocks have gone further to identifying which types of buildings and infrastructure contain the most steel, but such work is difficult to verify and does not describe what the significant end-uses of steel are currently.

2.2 Review of published literature on steel provision in construction

Austin (1998) states that a structural engineer’s objective is to produce a safe design “in the most economic[...] way possible”. Determining the potential to reduce the steel in buildings therefore requires an understanding of both structural design and construction economics. Structural design guidance is reviewed in section 2.2.1 to chart the advances in engineering knowledge over the past century and the subsequent changes in material requirements to provide a safe structure. Guidance on designing structures economically is reviewed over this period in section 2.2.2, examining the evolution of labour costs relative to those of materials and how this impacts on achieving an economic structural design.

2.2.1 History of structural steel design guidance

As human understanding of structural steel’s material properties and behaviour has improved it has become possible to design safely for the same applied load with less and less material. Bates (1984) describes the history of iron and steel in construction, listing the improvements in steel production that resulted in more consistent material properties. Production improvements have been so successful that Eurocodes, described by Nethercot (2012) as “technically the most advanced” design standards, allow engineers to assume full material strength when designing structural steel (BSI, 2005). Beal (2011) builds on Bates’ chronology to chart the reduction of ‘safety margins’ (allowances in design for uncertainty in material property, behaviour or loading) in UK design standards from a factor of 4 — when steel sections were first produced in quantity for construction in the 1880s — to 1.3–1.45 for Eurocode 3 (BSI, 2005), which superseded previous design codes in 2010. Beal notes that advances in engineering theory — such as the introduction of plasticity theory in 1959 — as well as material property improvements allowed more of steel’s strength to be exploited. Continuation of this trend is borne out in recent comparisons between the British Standards and the Eurocodes which are replacing them: Webster (2003) finds that Eurocode uses 2% less material for a concrete framed

building, while Moss and Webster (2004) conclude that Eurocode “offers scope for more economic structures”.

Despite a century of progress there is further scope for improvement of design standards as highlighted by studies such as Hicks (2007), which compares experimental results with predictions from Eurocode, finding strengths double those predicted in a majority of cases. This is affirmed by other studies which show ways for even less material to be used: Carruth (2012) demonstrates that an optimised, varying cross-section beam could have 30% less mass than a standard beam; Thirion (2012) corroborates Carruth’s finding and further finds that up to 30% less reinforcing steel is necessary in concrete flat slabs if detailed “using an infinite number of bar sizes and spacings”; Chan (1992)’s optimisation method reduces steel mass of a 60-storey building by 3.5%, while Liang et al. (2000) presents a performance-based topology optimisation method to minimise the mass of structural bracing systems.

2.2.2 Review of strategies to reduce building cost

At the beginning of the 20th century, when steel was becoming widely used in construction, BCSA (2006) notes that labour was comparatively cheap. Even so, Bates (1984) relates that in 1901 the industry reduced the number of section sizes into a standardised list (a forerunner of the modern catalogue: SCI (2009) (Steel Construction Institute)) to reduce manufacturing costs. During the Second World War, Beal (2011) reports that safety factors were reduced to “economise on scarce materials”, suggesting that material costs still outweighed those for labour.

Gibbons (1995) states that since the 1960s “the cost of plain steel sections in the UK has decreased dramatically relative to the unit cost of labour”, with the ratio of fabrication labour costs to material costs rising from 1 in 1960 to 2.88 by 1990. This change caused Needham (1971) to advise that “only rarely does [minimum weight design] achieve lowest cost”. He and Gibbons (1995) both describe how a design using a small number of different section sizes in a repetitive configuration requiring little extra fabrication will be one which is easier to detail, fabricate and construct, thus saving labour, hence cost, despite weighing more. Gibbons terms this practice ‘rationalisation’ and SCI (1995) gives further guidelines for it, noting that procurement costs are also reduced by large, repetitive orders resulting from rationalisation.

Needham and Gibbons estimate the threshold of weight increase beyond which rationalisation gives no cost saving at 5–10% and 20% respectively, though neither

provides justification for their value, and no published study has been completed examining the validity of these estimates. In the only published study of the efficiency of built designs, Sadek et al. (2006) analyse six concrete residential structures in Kuwait — a country with different material and labour costs to the UK — finding some contain twice as much reinforcement as necessary which increases costs; they attribute this to poor design practice rather than rationalisation however.

2.2.3 Findings from literature review on steel provision in construction

Published literature on the provision of structural steel in buildings reveals that it has become possible to use less material to support the same loads safely due to increasing human knowledge of steel production and performance; there remains further scope to use less material still. However strategies to produce economic structures have changed in the UK as the ratio of material to labour cost has changed in construction, with current practice favouring rationalisation as a method of achieving lower costs by adding extra material (where this allows a reduction in labour costs). This extra material does not supplement structural performance, it does not enhance safety, it is merely surplus material. No study published to date has quantified the amount of this extra material provision.

2.3 Published literature on structural failure, reuse and deconstruction

Extending the life of products requires an understanding of the reasons for their failure. Published literature is therefore reviewed in section 2.3.1 to ascertain the causes of structures reaching ‘end-of-life’. Previous research on strategies to overcome these causes is reviewed in section 2.3.2.

Even when an entire structure fails, there could be potential to extend the life of each component by reusing it, explored in section 2.3.3. Reasons why this does not occur are explored in section 2.3.4, leading to a review of deconstruction strategies and barriers in section 2.3.5.

2.3.1 Review of causes of end-of-life

There are many potential reasons for deeming a structure to have reached its end-of-life (also referred to as ‘failure’, regardless of cause) and therefore being demolished, from purely economic concerns to major structural defects. Literature is reviewed from land economics and from surveys of building and infrastructure end-of-life to determine the main causes of failure.

Land economists have examined owners’ motivations to replace buildings, finding that commercial concerns dominate decision-making and that demolition is preferred to refurbishment. Bullen and Love (2009) state that “[a] primary reason for the disposal of a building is because it does not meet the immediate needs of owners and their occupiers” and note that economic performance is increasingly important to building owners. Childs et al. (1996) observe that property developers upgrade buildings when greater density is required; Williams (1997) agrees with this, generalising redevelopment as giving ‘higher quality space’. Although refurbishment offers an opportunity to retain the structure while upgrading, Shipley et al. (2006) find it is more expensive and perceived as having more uncertainty. Bullen (2007) reports that refurbishment “remains an anathema to architects and most of the building professions” because they lack the requisite skills for it; thus demolition and reconstruction is preferred. Power (2008) decries the “perverse [financial] incentive for demolition” given by the UK government by charging Value Added Tax (VAT) on most refurbishments but exempting ‘new build’ projects from this tax.

Surveys of buildings find that few are demolished because they are structurally deficient, instead changes in use usually trigger end-of-life. Athena Institute (2004) find that only 3% of demolitions in Minnesota, USA over a 30-month period were caused by structural defects — ‘change of use’ and ‘area redevelopment’ were the most common causes. Ball (2002) surveys vacant industrial premises in England, finding that only 13% had ‘unsound’ structure, and these were overwhelmingly classed as ‘persistently vacant’ buildings; half of the buildings were reoccupied with no improvement work. Itard and Klunder (2007) and Thomsen and van der Flier (2009) study the increasing rate of house demolitions in the Netherlands, attributing it to functional and economic, rather than structural, causes. Similarly Durmisevic (2001) attributes demolition to buildings’ inability to accommodate changes in users’ demands.

In contrast to the multiple publications on building failure, no study reviewed addressed general reasons for infrastructure end-of-life — instead information is gleaned from studies specific to bridges, pipelines and rails, finding that physical degradation is a leading cause of failure. Wilson (2010) reviews 103 bridges built

2. REVIEW OF PUBLISHED LITERATURE ON STEEL USE IN CONSTRUCTION

in the UK during the 1950s and '60s, finding that 90% showed signs of physical degradation by 2010. Several bridges had to have “major structural components replaced” due to degradation but only two were demolished for this reason. Changes in requirements for bridges — wider or stronger structures necessary — caused upgrading works on 20% of bridges and five further demolitions, indicating that this failure is potentially as widespread as physical failures. Grigg (2013) states that water pipe failures are overwhelmingly due to physical degradation such as corrosion, joint failures and punctures; literature concerning oil and gas pipelines, for example Alamilla et al. (2013), imply that corrosion, fatigue and other physical failures dominate for these products. Choi et al. (2013) state that rail lifespan is mainly governed by wear, echoed by Milford and Allwood (2010).

Given the infinite number of specific building and infrastructure failures, it is not surprising that frameworks have been proposed to group similar failure types/reasons: Thomsen et al. (2011) attempt to generalise the reasons for dwelling ‘obsolescence’ (failure), propose four failure types along two axes (endogenous-exogenous and physical-behavioural), while Cooper (2005) distinguishes between failures caused by absolute and relative under-performance for consumer goods. Cooper et al. (2013) draw upon these works in their review of reasons for failure (termed ‘failure modes’) across all products and propose the failure framework shown in table 2.1. They categorise four failure modes — degraded, inferior, unsuitable and worthless — distinguishing between failures caused by the state of a product or by the user’s desires, and whether the failure affects one specific item or all such units.

	Degraded	Inferior
The performance of the product has declined ...	... relative to when it was bought	... relative to what is currently available
	Unsuitable	Worthless
The desire for the product has changed ...	... in the eyes of its current user	... in the eyes of all users

Table 2.1: Product failure framework, from Cooper et al. (2013)

Cooper et al. (2013) apply these failure modes to the major uses of steel globally taken from Cooper and Allwood (2012) using the detailed reasons for failure listed in table 2.2. They find that building obsolescence is mainly caused by ‘unsuitable’ failures and that infrastructure usually suffers from ‘degraded’ failure; these results are in agreement with the literature reviewed above.

2.3 Published literature on structural failure, reuse and deconstruction

Degraded	Inferior
Wear	Rival product offers lower costs
Fatigue	Technology superseded
Accidental damage	Rival product offers enhanced
Product spent	functionality
Product repair not economically viable	
Scheduled life reached	
Unsuitable	Worthless
Change in circumstance	Legislation that prohibits use
Change in preferences	Changes in the environment in which
Changes in legislation that effect	immobile products are used
requirements placed on products	

Table 2.2: Detailed reasons for product failure, used to apply failures to products, from Cooper et al. (2013)

2.3.2 Review of strategies to prevent failure

Literature that examines strategies to prevent end-of-life falls into two categories: either high-level or solution-specific. Examples of the former are Thomsen et al. (2011)'s overview framework to 'manage obsolescence' in buildings (advocating 're-design', without further details, if poor design causes failure) and Cooper (2005)'s discussion of 'design for longevity', neither of which link design strategies to the specific failures identified. Examples of the latter are Ertzibengoa et al. (2012)'s investigation of stainless steel reinforcement (to increase durability of concrete structures) and Slaughter (2001)'s design suggestions to achieve adaptable structures, neither of which address all failure modes. Therefore there is a gap in published knowledge linking failure frameworks to design strategies.

2.3.3 Review of potential to reuse structural components

It is improbable that the life of all structures can be prolonged indefinitely, but even when end-of-life of the entire structure is reached there could be potential to extend the life of the individual components by reusing them. Can structural elements be reused?

Addis (2006) identifies three characteristics a component must have to be reusable: it is not worn, yielded or corroded; it is not a superseded technology; it can still interface with new components. The literature reviewed in section 2.3.1 above suggests that infrastructure components will not meet the first criterion and Cooper

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and Allwood (2012) find that sheet applications and reinforcement bar often fail the second and third criteria respectively. Structural steel sections from buildings however meet all three requirements (provided they have not been exposed to fire, seismic or other extreme loading scenarios) as their standard sizes and connection technologies have not changed in the past 50 years (Addis, 2006); thus they are ideal candidates for reuse.

Three articles in the past decade have examined reuse in construction and identified barriers to it. Densley Tingley and Davidson (2011) review eight published articles on reuse, extracting 24 barriers, while Kibert (2003) lists seven factors that inhibit ‘closing material loops’, based on experience in the Netherlands. Astle (2008) completes a literature review on barriers to using reclaimed steel in construction, distinguishing nine key barriers at six stages between concept design and end-of-life. The identified barriers can be split into two categories: either constructors of new buildings cannot, or do not want to, design with reclaimed sections; or there is no supply of suitable, reclaimed sections. Table 2.3 lists the main barriers within each category.

Barriers to a supply of reused materials
1. Storing reclaimed materials
2. Cleaning, refabrication and testing requirements
3. Lack of supply of suitable beams from deconstruction sites
4. No information on availability and location of potential supplies
Barriers to designing with reclaimed materials
1. Negative perceptions, preference for new materials
2. Design codes and guidance not written for reclaimed materials
3. Additional design time may be necessary
4. Lack of certification causing insurance problems
5. Ignorance of possibility to design with reclaimed materials

Table 2.3: Barriers to supplying and designing with reclaimed materials

Which of these barriers is the most critical? Gorgolewski (2008) describes two successful reuse projects in Canada, and Sergio and Gorgolewski (2008) document the reuse of steel on the BedZED project in the UK. Both articles list a major challenge as the sourcing of reclaimed steel, inferring that design problems can be overcome; the major barriers relate to locating a supply of steel. Addis (2006) concurs, noting that the market for reclaimed materials is not mature enough to efficiently match buyers and sellers. Astle (2008) reports that 10% of structural

steel was deconstructed in 2000, but Kay and Essex (2009) note that this had fallen to 1.5% by 2007. Both studies state that most buildings are demolished, which damages the elements, thus they cannot be reused and instead are recycled.

2.3.4 Comparison of demolition and deconstruction

Why are most buildings demolished? Four studies compare costs of demolition and deconstruction, finding that while deconstruction involves more labour, it is cheaper on average once disposal costs are taken into account. Guy and McLendon (2001) deconstruct six residential structures, and obtain demolition estimates for them, finding that initial costs are 21% higher on average for deconstruction, but when salvage value and disposal costs are included, deconstruction is 37% cheaper on average. Greer (2004) states initial costs for houses are twice as high if deconstructed rather than demolished, but this reverses to a saving once reclamation tax credits are included. Coelho and de Brito (2011) review comparisons of different building types across Europe, each of which shows a net saving for deconstruction once transport, recycling and disposal costs are included, overcoming the higher initial costs. Their own study of townhouses in Lisbon finds demolition to be slightly cheaper even with all additional costs included, however they note the sensitivity of demolition costs to ‘mixed waste’ disposal charges. Unlike the above studies, Lazarus (2005) includes costs of storage, sales and transport of salvaged steel in his study of two office blocks, but still calculates a net saving for deconstruction.

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If deconstruction costs less than demolition, why have economic forces not prevailed to make it common practice? Within the barrier to reuse “lack of supply of suitable beams from deconstruction sites” (in table 2.3) there are sub-barriers given by Densley Tingley and Davidson (2011) and Astle (2008). These barriers, specific to deconstruction, are listed in table 2.4. Associated with each barrier is also the risk it might happen — risk of overrun, risk of damage, risk of testing problems, etc.

Challenges to deconstruction
1. Time constraints — deconstruction can take longer; often on ‘critical path’
2. Ensuring materials are salvaged safely & successfully (e.g. without damage or contamination)
3. Inaccessible joints & irreversible connections (e.g. in-situ concrete)
4. Lack of deconstruction skills, tools, specifications and guidance
5. Lack of information about materials & construction techniques originally used

Table 2.4: List of challenges to deconstruction

Which of these barriers are the biggest challenges to deconstruction? Astle (2008) concludes non-cost issues of risk, skills/tools optimised for fast demolition and an immature reuse market mean demolition is preferred. Lazarus (2005) identifies risk of overrun (as demolition often on project ‘critical path’) and risk of damage (hence reduced value) as factors which inhibit deconstruction. Allwood et al. (2010a) draw attention to the short timeframe available to raze buildings in developers’ programmes, where any extra time (e.g. for deconstruction) delays rental income from the completed property, so favouring demolition in the final commercial calculation. They note that buildings are often vacant for months before the final decision is taken, during which deconstruction could take place. These indirect costs of deconstruction result in deconstruction becoming more expensive than demolition, explaining the latter’s prevalence.

2.3.5 Review of strategies to aid deconstruction

If deconstruction is to become a mainstream industry practice it must offer a viable logistical and commercial alternative to demolition. In the first case this necessitates reducing deconstruction time and risk — the main barriers to deconstruction. Achieving this requires both technical and non-technical solutions.

2.3 Published literature on structural failure, reuse and deconstruction

Published literature over the past decade has focused on the technical principles that will aid future deconstruction, termed ‘design for deconstruction’ (DfD, the same acronym is used for ‘design for disassembly’ which has the same ultimate aim), to be included in designs for buildings currently being built. The primary driver for these is not specifically reducing deconstruction time or risk, but broadly making deconstruction ‘easier’, which implicitly decreases time taken and risks of overrun and damage. Densley Tingley and Davidson (2011)’s review of published articles lists 33 DfD strategies, while Kibert (2003) catalogues 27 DfD design principles. Durmisevic (2001) distinguishes the different ‘systems’ within a building that change at different rates before suggesting a range of DfD strategies appropriate to different systems and change rates. Pulaski et al. (2004) outline the issues for consideration in DfD to produce 10 principles for achieving it. Table 2.5 summarises the principal DfD strategies, the most crucial of which is choosing reversible connections.

Principal DfD strategies	
1.	Select reversible fittings, fasteners, adhesives and sealants that allow for quicker disassembly and facilitate the removal of reusable materials
2.	Design for prefabrication, preassembly and modular construction
3.	Simplify and standardise connection details, ensure accessible
4.	Design to accommodate deconstruction logistics and safety
5.	Simplify and separate building systems, designing with reusable materials
6.	Design for flexibility and adaptability, simplifying and standardizing components and materials
7.	Ensure design and ‘as built’ information & drawings are recorded and preserved

Table 2.5: Summary of design for deconstruction strategies

Given the amount of knowledge on DfD strategies, why are they not prevalent, thus allowing reuse? Non-technical aspects have not been studied to the same extent as DfD strategies, but have received increased attention in recent years. In their editorial, Thomsen et al. (2011) outline eight practical (mainly non-technical) questions on deconstruction processes, skills, risk, certification and policy that merit research. Allwood et al. (2012) suggest policy changes and certification solutions, and Astle (2008) puts forward ideas to deal with processes and risk. However, successful reuse of steel at BedZED (Sergio and Gorgolewski, 2008) implies that these issues are not crucial. Densley Tingley and Davidson (2011) identify the extra cost of DfD strategies as the most common barrier in the articles they survey; it is this cost premium that probably prevents their adoption. Jaillon and Poon (2013)’s review of DfD literature only notes cost savings from reduced waste disposal charges; however

Morgan and Stevenson (2005)’s design guidance suggests building flexibility may also provide commercial benefit — but this is not verified by research.

2.3.6 Findings from review of structure failure, reuse and deconstruction

The review of published literature found that causes of failure for structures vary but, in general, infrastructure fails due to physical degradation and buildings due to a change in use. Sections from buildings offer an excellent opportunity for reuse, but this does not occur due to a lack of supply, in turn due to a preference for demolition over deconstruction which renders sections unfit for reuse. Demolition is preferred to deconstruction primarily because of the time saving and lower risk it offers, which outweigh the potential cost savings of deconstruction. Published literature proposes DfD strategies as ways to reduce deconstruction time and risk. The plethora of these strategies and principles suggests that engineering knowledge is not the limiting factor in this area, instead it is the extra cost of DfD that prevents its adoption. Commercial clients will only accept this cost premium and instruct design teams to implement DfD strategies if they judge the strategies to give other advantages — termed ‘co-benefits’. Few commercial co-benefits to DfD have been identified apart from waste disposal savings; therefore research is necessary to ascertain further co-benefits.

2.4 Review of published literature on composite connectors

Section 2.3.5 notes that reversible connections are crucial to allowing deconstruction. Composite floor systems, in which the steel beam and concrete slab are irreversibly connected via a stud welded to the beam, therefore is a barrier to deconstruction (Densley Tingley and Davidson, 2011), with Webster and Costello (2005) recommending it be avoided in designs for deconstruction. However composite floors are the most common structural system for multi-storey buildings in the UK, accounting for approximately 40% of such floor area built annually (BCSA, 2011). Section 2.4.1 reviews the published research on composite construction, investigating the methods used to test designs and section 2.4.2 examines previous efforts to make composite systems deconstructable, thus enabling their reuse.

2.4.1 Review of literature on traditional, welded-connector composite beams

Engineering understanding of composite steel-concrete construction systems has evolved over the past century mainly based on ‘push tests’ supplemented by modelling. Design guidance has been continually updated to incorporate developments in understanding and research.

‘Push test specimens’ (an example of which is shown in figure B.1 of BS EN 1994-1-1:2004 (BSI, 2004) — referred to hereafter as ‘Eurocode 4-1-1’) were developed in the 1930s (Hicks, 2007) to determine the behaviour of composite connectors (called ‘studs’). Lloyd and Wright (1990) report that at this time composite steel-concrete beams were mainly used in bridge construction, with a flat-soffit slab cast on top of a beam with factory-welded connectors to transfer shear between components. They go on to explain that, as composite slabs were adopted in building construction, profiled steel decking was used as permanent formwork; this eliminated direct contact between the beam and concrete and necessitated site-welding of studs. That design standards BS 5950-3.1:1990 (BSI, 1990) and Eurocode 4-1-1 (BSI, 2004) only provide guidance for welded connectors is evidence of the ubiquity of this form of composite construction in buildings. Mottram and Johnson (1990) recommend geometric adjustments to the standard push test specimen, defined in 1965, to make it suitable for use with profiled decking.

All literature reviewed uses results from push tests (either new or previously published) to validate theoretical models of composite behaviour and, almost always, to appraise and update design guidance. Hawkins and Mitchell (1984) conclude from 23 push test results that connector spacing and geometry greatly impact the failure load; Mottram and Johnson (1990) undertake 35 further tests to appraise design formulae. Greater computing power has allowed increasingly detailed modelling of beam and connector behaviour: Johnson and Molenstra (1991) input a mathematical model from first principles to calculate strength and slip, while Ellobody and Young (2006) and Qureshi and Lam (2012) create finite element models to do the same; such models are validated against push test results.

Over time an increasing number of failure modes has been identified from push tests: Hawkins and Mitchell (1984) describe four (stud shear, concrete pull-out, rib shear, rib punching), Johnson and Yuan (1998) distinguish three more (splitting failure and two combination modes) and Patrick (2004) classifies four additional, less common modes. Patrick claims that existing guidance for trapezoidally-profiled

decking considerably underestimated strengths and slip capacities for welded studs. Responding to these claims, Hicks (2007) performed six push tests and two beam-bending tests and showed that the two sets of results have a poor correlation. Given composite floors in buildings are subject to loading in bending, Hicks concludes that the push-test specimen is deficient when evaluating beams with profiled decking; hence the design specifications are still safe (though a few minor corrections are needed). Smith and Couchman (2010) concur with Hicks, recommending minor updates to design guidance based on the results of 27 push tests from a rig modified to better correlate with beam tests.

2.4.2 Review of literature on demountable connectors

Relatively little research on demountable composite connectors has been undertaken, and motivations for doing so have changed over the past 50 years. Oehlers and Bradford (1995) catalogue different composite connector types, some of which are bolted or otherwise demountable. Dallam (1968) and Marshall et al. (1971) performed tests in the 1960s and '70s investigating the behaviour of friction-grip bolts as composite connectors but focused on the effect of pretensioning on the connection and did not demount them. More recently, Kwon et al. (2010) post-installed bolts to strengthen existing structures, investigating their performance under fatigue loading. In conference papers, Lee and Bradford (2013) develop a 'quasi-elastic mechanics based' theoretical model for the behaviour of pretensioned bolts and validate it against push test results, while Lam and Saveri (2012) describe experiments using connectors machined from traditional studs with threads (shown in figure 2.1) so they can be bolted onto a beam and disassembled. Both sets of authors show that the bolted connection performs suitably in a push test, but beam tests were not completed. Although not supported by published research, the Australian building standard AS 2327.1-2003 "Composite structure: Part 1: Simply supported beams" (SAI, 2003) depicts a bolted connector with a comment that they should be treated as if the same as manually welded connectors; however no references to the bolted connector are made in the standard's main text.



Figure 2.1: Demountable connectors machined from traditional studs, taken from Lam and Saveri (2012)

2.4.3 Findings from literature review

In the body of published work on composite steel-concrete construction there have been a large number of push tests but few beam tests — despite poor correlation between the two and beam tests being closer to actual use of connectors in construction. None of the modelling of connector failure modes, for example Yuan and Johnson (1998), inherently precludes using bolts as none require moment resistance at the connector base. Of the studies that have examined demountable connectors, few have examined demountability and none present results from beam tests or on tests using non-preloaded bolts as connectors.

2.5 Summary of literature review

The findings of each of the sections 2.1 to 2.4 are summarised, identifying gaps in the knowledge base that require research; these form the starting points for chapters 3 to 6.

From the review of literature on material flows into construction in section 2.1, it is concluded that the information required to guide research on material efficiency has not yet been published. Specifically, it is not known which types of structures use the largest aggregated tonnage of steel each year, nor the predominant products that these structures are constructed from, nor what components within a typical structure contain the most steel. Chapter 3 outlines a methodology to determine each of these items by combining existing knowledge sources and applies it to produce an estimate of steel end-use in the UK and globally, as well as steel end-use within

2. REVIEW OF PUBLISHED LITERATURE ON STEEL USE IN CONSTRUCTION

a ‘typical’ structure. An estimate of steel present in the building stock, though potentially useful for assessing scope for reuse, is not attempted due to difficulty in assembling and verifying such an estimate, and because targeting material efficiency strategies is not as dependent on steel stock estimates as it is on current use estimates.

Section 2.2 finds that, to date, there is no published analysis quantifying the extra mass actually added to building designs due to rationalisation. Chapter 4 puts forward a method to do so and applies it to an extensive data set gathered from recent practice, permitting estimation of the potential for steel saving from decreasing rationalisation.

Section 2.3 finds that although reasons for structure end-of-life have been suggested and strategies to prevent these failure have been proposed, no research to date has connected the two. Chapter 5 therefore produces a framework of life-extension strategies tailored to Cooper et al. (2013)’s failure framework. Section 2.3 further concludes that a main barrier to reusing structural steel is the lack of supply of suitable elements, itself due to a preference for demolition over deconstruction. Designing for deconstruction (DfD) would allow a greater supply of suitable elements but is not practised due to its cost. Chapter 5 therefore investigates co-benefits that will permit DfD strategies to be employed on a commercial project, hence facilitating deconstruction and reuse.

The review of composite connectors in section 2.4 reveals that a limited number of articles have examined demountable connectors and none present results from beam tests on demountable connectors, or on tests using non-preloaded bolts as connectors. Chapter 6 therefore reports on the investigation of the behaviour of such bolts used as composite connectors in three beam tests.

Chapter 3

The flow of steel into the construction sector

To have most impact, material efficiency strategies should be applied to the largest flows – hence end-uses — of steel in the construction industry. Specifically of interest are the types of buildings and infrastructure that consume the most steel annually when aggregated across a country or the globe and the product-types that these involve. The identity of a ‘typical’ structure, if it can be found, would allow specific findings to be applied generally, as would distinguishing the largest uses of steel within such a structure. A review of literature in section 2.1 could not locate this information, so a methodology is proposed in section 3.1 to uncover and assemble it. This methodology is then applied to produce estimates of steel flows into construction for the UK and the world, and to estimate the steel distribution within a typical structure, both presented in section 3.2. Implications of these results extend beyond material efficiency and are discussed in section 3.3. This chapter is based on the published journal article “The flow of steel into the construction sector” (Moynihan and Allwood, 2012a).

3.1 Methodologies for determining steel flows into construction

Section 3.1.1 presents a methodology to determine steel use by construction sector. A methodology is also proposed for estimating the proportional steel use between different applications within a ‘typical’ structure in section 3.1.2. Uncertainty is inherent due to incomplete data sets; section 3.1.3 describes measures to manage this uncertainty to ensure confidence in the results.

3.1.1 Method to determine steel distribution by construction sector

A methodology of five parts is presented to determine the distribution of steel by sector within construction: categorise the main intermediate product types used in construction; calculate the tonnage of each used within the industry and convert to end-use product tonnages; classify sectors within construction; allocate each product between the construction sectors; sum the allocations to find the total steel tonnage in each sector. Top-down sources were used for the former three steps and bottom-up for the latter two.

The five main intermediate product types used in construction are: sections; reinforcement (rebar); sheet/plate; rails; tubes (Ley, 2003; Wang et al., 2007) — Wang et al. (2007) indicate castings are also used in construction, however worldsteel (2011a) show that the tonnage of castings is small compared with the others. Within these broad headings product sub-category terminology was distinguished from correspondence with manufacturers and trade associations. Product definitions were taken from worldsteel (2011b), supplemented with industry terminology for sheet and tubes. The level of sub-category selected follows classifications used in industry and a sufficient number of sub-categories were chosen such that their total approximated the aggregate top-down value.

The tonnage of each product used in 2006 was taken from top-down sources: UK Steel (2010) for the UK and worldsteel (2011a) for the world, ensuring that all relevant sub-categories, such as reinforcing mesh and light sections, were also included. What worldsteel and UK Steel refer to as ‘finished’ products were termed ‘intermediate’ products, with ‘end-use’ referring to final products that actually make up buildings and infrastructure. The proportion of each product going into construction was calculated from Wang et al. (2007)’s product-to-use matrix or from industry estimates and government data where available. For global estimates it was assumed that all sections and rebar are used in construction, and the sheet/plate tonnage was back-calculated from the other values. Product yields — the percentage of intermediate product mass that is retained in the final product (i.e. less the material lost during manufacturing) — were taken from Cullen et al. (2012) to convert intermediate totals to end-use ones. By dividing the overall end-use tonnage by the overall intermediate tonnage an overall yield ratio for the entire construction sector was calculated.

The year 2006 was chosen for this analysis for four reasons: there was good data availability for this year; steel consumption in 2006 does not contrast starkly with

3.1 Methodologies for determining steel flows into construction

adjacent years (subsequent years are not selected as their data show large variations due to the global economic recession); any omissions in the data could be estimated from 2005 data, which is probably comparable; 2006 was the most recent year for which these criteria are true.

Construction comprises two distinct categories: buildings and infrastructure; the former being structures to provide shelter and the latter being supply and communication networks required to service society. Within these there are a number of options for defining sectors: by end-use/market segment; by structural system; by size or other metric. End-use/market segment was selected because it could be related to the other metrics through assumptions and because most data were found in this form. Six building sectors were chosen: industrial; commercial; offices; public; residential; other. The four infrastructure categories selected were: utilities; rail; bridges; other. The definition of each sector is given in table 3.1 based on frequent occurrence in published literature. Table 3.1 also lists ‘preferred’ structural types for each sector determined from interview with professionals from the BCSA and Arup; infrastructure has many different structural types specific to each sector.

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Sector	Definition	‘Preferred’ structure
Buildings		
Industrial	Factories and warehouses	Portal frame
Commercial	Retail and leisure facilities	Portal frame (single-storey) Braced steel frame (multi-storey)
Offices	All office workspaces, including in mixed-use	Braced steel frame
Public	Education, health and administration	Braced concrete frame
Residential	Houses and apartments	Braced concrete frame
Other	Stadia, agricultural & miscellaneous	Long-span roof
Infrastructure		
Utilities	Energy, water and waste generation, processing, distribution and collection networks and plants	Specific to application
Rail	Tracks and sleepers	-
Bridges	Road and rail bridges	Specific to project
Other	Airports, harbours & miscellaneous	Specific to application

Table 3.1: Sector definitions as used in this study

The product tonnages were allocated between sectors using bottom-up data gleaned from industry publications, interviews with trade associations and manufacturers (BCSA, Celsa UK, Tata UK, Metsec, Arcelor Mittal UK), and personal communications with other industry professionals, shown in table 3.2.

3.1 Methodologies for determining steel flows into construction

Product	Allocation source
Sections	BCSA (2010)
Rebar	Celsa UK ^a values with cement data from MPA - Cement (2011)
Sheet/plate	By product:
Roofing/cladding	Following sections' allocation
Decking	As per sections for multi-storey buildings only
Sheet piles	Data from Arcelor Mittal UK ^a
Cold-formed components	Following sections' allocation
Plate girders	Data from BCSA ^a
Rail	By inspection
Tubes	By product:
Pipelines	By inspection
Structural	Following sections' allocation
Non-structural	Data from Tata UK ^a
Generic	Buildings only, following sections' allocation

Table 3.2: Allocation sources for products used in UK construction

Notes:

a. Data from interview with named company or trade association

Published values were preferred over interview data, and where direct information could not be found, estimates and proxies were used. Worldwide sources were not available for any product allocation so data from a region, or combination of regions, was taken as indicative. For example, trade associations publish annual by-sector statistics for sections, thus the UK allocation was taken from BCSA (2010), and European statistics from ECCS (2009) (European Convention for Constructional Steelwork) were used as indicative to make the global allocation. By comparison, rebar associations do not hold such data, and only limited information is available from manufacturers. Cement was identified as a proxy for rebar, so trade association data (MPA - Cement, 2011) was used, once calibrated using rebar data from Celsa, to make UK allocations. Data from USA (Portland Cement Association, 2011) and Turkish (Akansa Cement, 2012) cement industries was then combined with this to estimate global allocations. Sheet has many disparate uses so allocation was based on manufacturer or trade association data for five main applications

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(roofing/cladding, decking, piles, cold-formed sections, plate girders). Similarly, tube has different applications so allocation was informed by manufacturer data for four predominant applications: pipelines; structural; non-structural; generic. The few global datapoints available for sheet and tube allocations were combined with UK values to formulate worldwide allocations of these products. Rail was only used in one category ('rail') so it was all allocated to it. Detailed reasoning and calculation for each product is included in appendix A. Once complete, the allocations were summed to find the total steel used in each sector.

3.1.2 Distribution of steel within a typical structure

Determining where steel is used in a typical structure was undertaken in four parts: a typical structure was first defined; categories of steel within it were identified; individual applications of steel were investigated and ranges for steel intensity found; proportional distributions of steel were determined. Data were obtained from both top-down and bottom-up sources, combining surveys, case studies, design calculations and industry 'rules of thumb'.

A 'typical' structure can be either a 'typical' building or a 'typical' infrastructure installation. From the interviews with industry professionals about the differing forms of structure within infrastructure categories, described in table 3.1, and the review of literature (in section 2.1) it was apparent that a typical installation cannot be identified because most structures are specialised to their particular application. Although a truly typical building does not exist (because almost all construction projects are bespoke), similar structures are used across all categories, allowing a typical building to be identified. This was achieved by combining bottom-up market survey data, obtained from interview with the BCSA, with the top-down sector analysis results. The choice of building was checked against Goodchild (1993)'s and Concrete Centre (2011)'s cost models to ensure it is one the construction industry itself regards as typical.

Three categories of steel function were distinguished:

superstructure: beams, slab, walls and columns;

substructure: basements and foundations — both 'shallow' (pads and strips) and 'deep' (steel and concrete piles);

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non-structural: façades; service systems, ducts and machines; fixtures & fittings — e.g doors, frames, handles, handrails (but not including furniture due to a lack of sources on its steel content).

Other categories of steel function, such as product types or ‘fit-out’ stages, could have been selected, but the chosen function categories align well with product categories, are the same across all structures, and fit with available data sources. To find the steel content of each category, four techniques were applied: steel tonnage values were taken directly from publications where given; knowledgeable individuals from BCSA (trade association) and Arup (engineering consultancy) were interviewed, with further data obtained from communication with Explo’re Manufacturing (contractor) and Davis Langdon (cost-consultants); design calculations were completed to determine steel tonnages directly; aggregated steel rates from building case studies were used to check that results are broadly consistent.

Superstructure steel intensities for different floor-systems were calculated for both steel- and concrete-framed designs for the designs given in Goodchild (1993). These build on the general ranges in Arup (2008) and those calculated from Target Zero (2011) and Goodchild. Substructure intensities are given per m³ of concrete, in line with industry practice, as site-specific ground conditions govern design and hence this measure is most appropriate. ‘Rule of thumb’ intensities from Arup (2008) were enhanced by professional opinion from Arup interviews and case study values from Chau et al. (2008). Unlike structural items, the steel content of non-structural items was difficult to quantify as their steel content is often small or not a design consideration. Additionally they are specified by a range of professionals and are usually prefabricated so information about them is disperse. Intensities for mechanical services, façades and fixtures & fittings were taken from Eckelman et al. (2007) and augmented with ducting and mechanical data from Explo’re Manufacturing and Davis Langdon, and fixtures data from Arup.

The proportion of steel in each category was assessed by calculating tonnages and then computing percentage proportions. The tonnages were the product of the obtained steel intensities for each category, and Goodchild (1993)’s building plans, as well as values taken from Arup interviews. The percentage ranges reflect the typical values found, omitting outliers.

3.1.3 Methods to manage uncertainty

Because data are drawn from different sources, many of which are unclear about how the information was gathered and its limitations, uncertainty is present throughout the analysis. Use of proxies, estimates and calculation (detailed in appendix A for each product) to combine and derive results further add to error margins. This is particularly true for the worldwide distribution where a lack of information required extrapolation of UK, European and USA data — an assumption which is probably inaccurate because developing countries such as China and India (which are large consumers of steel (worldsteel, 2011a)) are likely to use steel differently than developed countries. Two strategies were employed to bound this error: bottom-up estimates; and cross-checking between sources. Uncertainty in the results was judged by calculating the difference between the results and the checking sources. Results are reported to the nearest 100 kilotonnes (kt) for the UK, and to the nearest 10 megatonnes (Mt) for the world, each of which is approximately 1% of the total steel mass flow within that boundary annually.

The allocation of steel within UK construction by sector is checked by a bottom-up estimate: combining sector steel intensities with population data. Values of steel intensity in kg/m^2 for different building types are calculated based on published values, interview data and estimates relative to the ‘typical’ building results. The intensities are multiplied by built floor-areas obtained from interview with the BCSA (but independent of top-down BCSA data) to give tonnages by sector. Infrastructure values are calculated from lengths/numbers of installations built and steel intensities, for example the distance of rail track laid in the UK from Network Rail (2006) was multiplied by a calculated mass of rail steel. The only population data for utility networks available was gas pipelines so this was compared with the corresponding top-down value. It was not possible to make steel intensity or population estimates for the ‘Other’ sectors, as these are too varied. Bottom-up results are rounded up to compensate for units not included in the population data.

The worldwide distribution could not be checked by a bottom-up estimate as no sources of built areas or units could be found. The world results therefore are cross-checked with published national values and stock estimates, as listed in section 2.1, to verify the data. The sensitivity of the results to developing country values was estimated by re-calculating the global allocation using section and cement (a proxy for rebar) data from Turkey (details assumptions and results are provided in appendix A.3) and comparing these values with the global results. The typical building results were implicitly verified by their use in the UK bottom-up check

and by their inclusion of previously published studies and estimates from practising professionals as described in section 3.1.2.

3.2 Results for steel use within construction

The above methodologies were applied to UK and world data, producing estimates for the distribution of steel within construction for the year 2006, including breakdowns by product, and an estimate of the distribution of steel within a typical building. These are presented below in turn.

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3.2.1 Distribution of steel within UK construction

Table 3.3 shows the tonnage of each steel product used in construction in the UK, along with the source for each. As can be seen the tonnage of sections is largest, but sheet steel and reinforcement tonnages are only 10% smaller. Interestingly the magnitude of cold-formed sections is similar to that of heavy sections, implying that more cold-formed sections are used as these are lighter on average (SCI, 2009) (Steel Construction Institute).

Product	End-use products in construction (kt)	End-use estimate source
Sections	1700 ^b	
Heavy	1200	BCSA (2010)
Light	500	Celsa UK ^c
Reinforcement	1500 ^b	
Bars	1200	Celsa UK ^c
Wire rod	300	Celsa UK ^c
Sheet	1500 ^b	
Roofing/cladding	200	MCRMA (2011)
Decking	100	BCSA ^c
Sheet piles	100	Arcelor Mittal UK ^c
Cold-formed sections	1000	Metsec ^c
Plate girders	100	BCSA ^c
Rail	200	Wang et al. (2007)
Tubes	900	Wang et al. (2007)
Pipelines	300 ^b	
Structural	200	Tata UK ^c
Non-structural	300	Tata UK ^c
Generic	100	Tata UK ^c
Total	5800 ^a	

Table 3.3: Steel use in UK construction by product

Notes:

- a.** Table values do not sum to total due to rounding.
- b.** Values without sources were back-calculated from values with sources
- c.** Values obtained from interview with professionals from the named company or trade association.

3.2 Results for steel use within construction

The results for the use of steel in UK construction by sector are shown in table 3.4, which lists allocation of each product between the sectors. Overall, three-quarters of steel is used in buildings, half of which is in industrial buildings and the rest spread relatively evenly. Infrastructure is dominated by utility applications of steel, notably large oil and gas pipes which consume 300 kt annually. Although similar amounts of sections, sheet and rebar are used annually in the UK, sections and sheet are used almost exclusively in buildings (particularly industrial buildings), while rebar is split more evenly between sectors. The table shows that sectors with similar totals can have different product compositions, for example office and public buildings.

Sector	Sections	Rebar	Sheet	Rail	Tubes	Total
Buildings	1600	800	1400	0	500	4300
Industrial	800	0	700	0	200	1800
Commercial	300	200	200	0	100	800
Offices	200	100	200	0	100	600
Public	100	300	100	0	0	500
Residential	100	200	100	0	0	400
Other	100	0	100	0	0	200
Infrastructure	100	700	100	200	400	1400
Utilities	0	400	0	0	300	800
Rail	0	0	0	200	0	200
Bridges	0	0	0	0	0	100
Other	0	200	0	0	100	300
Total	1700	1500	1500	200	900	5800

Table 3.4: 2006 allocation of steel products by sector for the UK (kt/year)
Note that values do not sum due to rounding

The UK allocation is validated by a bottom-up study, which provides robust validation with absolute error margins. The bottom-up calculation multiplied sector steel intensities by population data. Steel intensities were calculated from interview data, published sources and estimates, with results given in table 3.5.

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Sector	Steel intensity	Units	Calculation [value <i>item</i> (source/assumption)]
Buildings			
Industrial	80	kg/m ²	35 kg/m ² <i>superstructure</i> (Interview with Arup) + 18 kg/m ² <i>foundations</i> (a) + 11 kg/m ² <i>cladding</i> (MCRMA, 2011) + 15 kg/m ² <i>services</i> (b)
Offices	100	kg/m ²	Interview with Arup,
Commercial & Public	100	kg/m ²	confirmed by ‘typical’ values found in section 3.1.2
Residential	20	kg/m ²	50% houses: (NHBC, 2006): 6 kg/m ² <i>foundations</i> (c) + 5 kg/m ² <i>superstructure</i> (d) + 5 kg/m ² <i>fixtures and fittings</i> (e) 50% apartments: (NHBC, 2006): 15 kg/m ² <i>foundations/structure</i> (f) + 10 kg/m ² <i>fixtures and fittings</i> (e)
Infrastructure			
Utilities — pipes	740	kg/m	L555MB linepipe: 1219 mm in diameter, 25 mm in thickness
Rail	60	kg/m	Milford and Allwood (2010)
Bridges	70	t/bridge	‘Average’ UK bridge estimated from Wallbank (1989), its steel intensity calculated from Das (1997)

Table 3.5: Calculation of steel intensities for UK bottom-up estimate

Assumptions:

- a. Shallow foundation 0.3 m thick with 60 kg/m³.
- b. High end of services range from section 3.2.3.
- c. Shallow foundations 0.2 m thick with 30 kg/m³ — lower than (a) as not all house foundations have rebar.
- d. Made up of small beams (e.g. Rolled Steel Joists).
- e. Houses are low end of fixtures and fittings range from section 3.2.3, apartments higher due to elevators, handrails, etc.
- f. Apartment foundations and superstructure more steel intensive than houses as high-rise blocks have reinforced concrete frame, while low-rise are unreinforced masonry blocks.

Population data is taken from a survey of buildings (square meters of floorspace constructed, commissioned by BCSA and independent of top-down BCSA data)

3.2 Results for steel use within construction

and from industry literature as outlined in table 3.6. The table also shows the calculation to obtain the bottom-up results.

Sector	Steel intensity	Unit	Population	Unit	Pop. source	Result (kt)
Buildings						
Industrial	80	kg/m ²	20x10 ⁶	m ² floor-area	[1]	1600
Offices	100	kg/m ²	6.2x10 ⁶	m ² floor-area	[1]	700
Commercial & public ^a	100	kg/m ²	9.1x10 ⁶	m ² floor-area	[1]	1000
Residential	20	kg/m ²	14.1x10 ⁶	m ² floor-area	[1]	300
Infrastructure						
Utilities — pipes ^b	740	kg/m	350	km pipe laid	[2]	300
Rail	60	kg/m	1200	km track laid	[3]	200
Bridges	70	t/bridge	450	bridges built	[4]	100
Total						4200

Table 3.6: Inputs to, and results from, bottom-up estimate of steel use in UK construction in 2006

Sources: [1] Market survey data, obtained via interview with the BCSA [2] National Grid (2007) [3] Network Rail (2006) [4] Wallbank (1989)

Notes:

- a.** Population data not available separately for ‘commercial’ and ‘public’ buildings, so these sectors combined.
- b.** Population data was only available for pipelines (which constitute half of utility sector) so only these analysed.

The bottom-up estimate is compared with the top-down result in table 3.7. Differences everywhere are less than 300 kt, with no difference in tonnages for infrastructure. That the bottom-up estimates are lower than top-down is characteristic of both methods (Hirato et al., 2009), as small-but-significant populations were omitted, for example commercial floor-space data omitted single-storey retail buildings (which these results imply account for half of commercial steel use). Aggregated disagreement is 500 kt, or 11%.

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Sector	Bottom-up	Top-down	Difference
Buildings			
Industrial	1600	1800	200
Offices	700	600	-100
Commercial & Public	1000	1300	300
Residential	300	400	100
Infrastructure			
Utilities - Pipes	300	300	0
Rail	200	200	0
Bridges	100	100	0
Total	4200	4700	500

Table 3.7: Comparison of bottom-up and top-down estimates for 2006 steel use in UK construction (kt)

3.2.2 Distribution of steel within construction globally

Table 3.8 outlines the tonnage of each product used in construction annually. Unlike the UK, rebar constitutes over one-third of steel used in the industry, with sections only accounting for half as much.

Product	End-use product in construction (Mt)	Basis
Sections	80	Assumed all production used in construction
Rebar	190	Assumed all production used in construction
Sheet	140	Back-calculation from other values
Rail	10	Wang et al. (2007)
Tube	50	Wang et al. (2007)
Total	480	Wang et al. (2007)

Table 3.8: Steel use in global construction by product

The estimates for steel use within construction globally by sector are given in table 3.9, which also lists the allocations per product. Overall, buildings account for almost two-thirds of steel use, and like the UK, the largest sector is industrial, with a magnitude equal to the commercial, office and public sectors combined. Infrastructure is over one-third of steel consumption in construction, with utilities constituting over half of this. Rebar is the dominant steel product globally, having twice the tonnage of sections and 25% more than sheet. As for the UK, the distribution of

3.2 Results for steel use within construction

products between sectors varies substantially. From the product analysis, the overall yield ratio for construction products was calculated as 0.94 by the method described in section 3.1.1, which confirms Hatayama et al. (2010)’s previously published value.

Sector	Sections	Rebar	Sheet	Rail	Tubes	Total
Buildings	60	100	110	0	20	290
Industrial	30	10	60	0	10	110
Commercial	10	20	10	0	0	40
Offices	10	10	10	0	0	30
Public	10	20	10	0	0	40
Residential	0	40	10	0	0	50
Other	10	0	10	0	0	30
Infrastructure	20	90	30	10	30	180
Utilities	10	60	10	0	20	100
Rail	0	0	0	10	0	10
Bridges	0	0	10	0	0	10
Other	10	20	10	0	10	50
Total	80	190	140	10	50	480

Table 3.9: 2006 allocation of steel products by construction sector for the world (Mt/year)

Note that values do not sum due to rounding

World results are verified by comparing building and infrastructure proportions with two sets of published values: national steel use estimates in table 3.10; national and local steel stock estimates in table 3.11, with data for New Haven and Connecticut having sufficient resolution for a comparison by sector. Additionally, global values calculated using data from Turkey (a developing nation) are included in table 3.10 to estimate the bias in the results due to primarily developed-world data sources being used.

Both sets of published results are within 13% of the published estimates, apart from residential and non-residential proportions in China — it is not known why the China results are so different, although one hypothesis might be that urbanisation in China has lead to large-scale construction of apartment blocks, which contain large amounts of steel. If the China data is assumed to be representative of the world, then this implies an error margin of 11% in the overall buildings and infrastructure categories but 27% in residential buildings and 15% in non-residential buildings. By comparison, the analysis using data from Turkey only produced a distribution within 6% of the global analysis.

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Sector	UK ^a	World	Turkey ^b	China	Japan
Buildings	75%	64%	63%	75%	70%
Residential	7%	11%	16%	38%	-
Non-residential	69%	53%	47%	38%	-
Infrastructure	25%	36%	37%	25%	30%
Source:	This chapter		[1]	[2]	

Table 3.10: Comparison of global results with published national estimates of steel use

Sources: [1] Hu et al. (2010b) [2] Japan Iron and Steel Federation (2011)

Notes:

a. Values do not sum due to rounding.

b. Values for a global analysis based on data from Turkey only, as described in appendix A.3.

Sector	Consumption		Stocks			
	UK	World	New Haven	Connecticut	Japan	China
Buildings	75%	64%	75%	81%	68%	73%
Industrial	23%	32%	35%	21%	-	-
Commercial/Offices	15%	25%	26%	19%	-	-
Residential	11%	7%	18%	13%	-	-
Public	9%	9%	-	21%	-	-
Infrastructure	25%	36%	25%	19%	32%	27%
Utilities	21%	14%	14%	15%	-	-
Bridges	2%	4%	4%	5%	-	-
Rail	2%	4%	2%	2%	-	-
Other	11%	5%	1%	5%	-	-
Source	This chapter		[1]	[2]	[3]	[3]

Table 3.11: Comparison of global results with published estimates of steel stocks
Sources: [1] Drakonakis et al. (2007) [2] Eckelman et al. (2007) [3] Hsu et al. (2010)

3.2.3 Distribution of steel within a typical building

A ‘typical’ building is identified as a three-storey office block of braced-frame construction. This building type has significant overlap with public, high-rise residential and multi-storey commercial construction, and is commonly manufactured

3.2 Results for steel use within construction

in both steel and reinforced-concrete (industrial buildings, though a slightly larger proportion of steel use, only overlap with commercial single-storey structures). The distribution of steel within such a building is shown in figure 3.1.

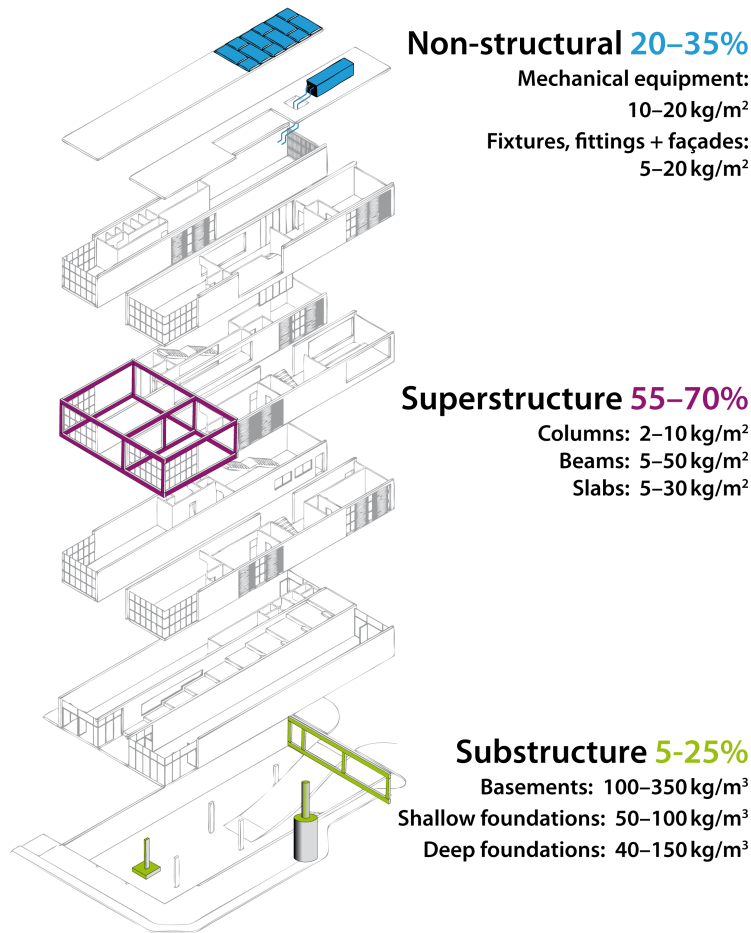


Figure 3.1: Distribution of steel within a typical office building, adapted from Allwood et al. (2012)

Most steel is found in the floor-structure (the slabs that support a floor and any beams supporting the slabs) of a building, regardless of whether the frame is made from steel sections or reinforced concrete. The ranges noted in figure 3.1 reflect the variability of steel intensity between different systems — where long-spans and thin floors are desired the steel tonnage required is high, where shorter spans and a deeper floor are permissible then much less steel is required. In either case relatively small amounts of steel are present in columns. In general, the lower data points of the ranges are for reinforced-concrete-framed systems, while the higher points are for steel-framed systems; however the proportions by category do not vary significantly with frame material. Simple foundations, such as pads, might only contain 10% of a building's steel, although a substructure consisting of both piled foundations and

a basement can contain much more to resist soil and water loads. Non-structural steel is usually the smallest category; it can nonetheless constitute over one-third of the total amount if a steel façade is used (e.g. corrugated iron).

3.3 Discussion of results

The results have implications beyond providing targeting for material efficiency strategies and can be used by researchers and professionals investigating steel or material use nationally, internationally or in typical structures. They imply that studies of steel flows into construction and building life cycle assessments may contain discrepancies, outlined in section 3.3.1. The variance of the results over time is discussed in section 3.3.2 and opportunities to enhance the results are proposed in section 3.3.3. The types of structures and products that should be targeted first for material efficiency analyses are identified in section 3.3.4.

3.3.1 Key findings and implications of results

Key findings from the results are discussed in turn for the UK, world and typical building distributions. The implications from the results are that: more steel is used within UK construction than had previously been estimated; steel proportions within a typical building are insensitive to the main frame material; non-structural steel use is non-trivial, hence should not be omitted from life-cycle analyses; the increased knowledge of current steel flows improves the accuracy of future forecasts.

Discussion of UK results

The results show that the UK constructs more new buildings than infrastructure, and that industrial buildings are the largest sector, accounting for the same tonnage as the next three largest building sectors combined — unexpected when manufacturing is reported to be in decline (Crawley and Hill, 2011), but it is plausible that manufacturing used to be larger as found in section 3.3.2 below. The distribution of products shows that sections and sheet together constitute most of the industrial sector’s total, while the commercial, office and public sectors include more rebar.

The proportion of steel flowing into UK utility networks does not agree with the interviewed professionals’ expectations (namely BCSA and Arup), perhaps because these individuals have experience mainly in the building sector. These networks are

broadly hidden from public view but are a major end-use of steel. By comparison, high-profile buildings, such as stadia, and prominent infrastructure, such as bridges, are a small proportion of constructional steel use.

While the literature cited in section 2.1 agrees that approximately 25% of total UK steel consumption is in construction, the results show that in 2006 this fraction was over 40%. Causes for this discrepancy are unclear, but it is possible that previous studies either omitted products (e.g. sheet, rails or tubes), omitted sub-sections within products (e.g. mesh within rebar), or had different boundaries (e.g. narrower definition of infrastructure). A similar error could exist in previous estimates at the global level, but further data and analysis would be required to test this.

Discussion of global results

Globally, infrastructure accounts for one-third of construction steel use, as opposed to one-quarter in the UK. This is not unexpected as developing nations are building up their infrastructure networks, while the UK already has these in place — indeed, Yellishetty et al. (2010) note that steel production has dramatically increased in China and India over the last 50 years. Industrial buildings are a smaller fraction of buildings globally than in the UK, which is unexpected because reinforced-concrete frames are preferred for non-industrial buildings in most countries, while the UK is one of only three markets (the other two being the USA and Japan) where steel frames are traditionally dominant (BCSA, 2011). That residences globally contain twice as much steel as those in the UK can be explained by the British preference to houses over apartments (Williams, 1997); in continental Europe the preference is for apartments (Meijer et al., 2009), which contain more steel per inhabitant than houses. The relative magnitude of reinforcement compared with the other products is a final difference between the world and the UK results: the former has a larger tonnage of rebar than sheet or sections, while the latter has comparable tonnages for all three.

The ratio of buildings to infrastructure steel use globally is similar to other consumption estimates in table 3.10. Hu et al. (2010b)’s breakdown for China, a developing nation, shows proportionately less steel going into infrastructure than Japan Iron and Steel Federation (2011)’s breakdown shows for Japan, a developed nation, which is unexpected. Further examination of Hu et al. (2010b)’s values reveals 32% of steel being used in urban residential buildings, with structural steel intensities of 35–45 kg/m² quoted. Such values are high for a typical residential structure — they would be more usual for commercial buildings. The results from the analysis

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with Turkish data show that developing nations may have different breakdowns by product but that these partially negate each other on aggregate, i.e. proportionately more rebar is used in buildings in Turkey than the world, but less sections and sheet are, giving a small net deviation from the global results.

Interestingly, the building:infrastructure ratios quoted for steel stocks are similar to those found for current consumption, shown in table 3.11. In one sense this confirms that the ratios found are of the correct magnitude, indeed the New Haven and Connecticut studies provide further detail to show reasonable agreement at sector-level. However, because infrastructure is longer-lasting than buildings (Hatayama et al., 2010), it is expected that the stocks ratios would be less than the consumption ones, but this is not the case. The reasons for this discrepancy are not understood; preliminary data shows that buildings:infrastructure consumption ratios in the 1970s and 1980s are similar to today's (Daigo, 2012), but an analysis of consumption for the past 30 years would be required to investigate this phenomenon further.

As data for steel use globally were scarcer than UK data, broad assumptions had to be made to enable an estimate of the global steel use: data from Europe and the USA was assumed representative of global values; rebar was assumed used in the same proportion with cement as calculated for the UK; sheet and tube use were assumed to follow sections' use. It is unlikely these assumptions are true, which has implications for the robustness of the results. The analysis of Turkish steel data, used to test the first assumption, found that sector values had an error margin of 30 Mt (or 6% of global construction steel use); sector values would change by up to 90 Mt (18%) if Turkish cement data is used to calculate global rebar use assuming directly proportional use (this is unlikely to be true but places a bound on the error); if UK sheet or tube allocations are used in the global results then this will alter sector values by up to 30 Mt (6%). Based on these three tests, it is estimated that the global results are within approximately 50 Mt (or 10%) of true values.

Discussion of typical building results

The results reveal that the proportions of steel by category do not change significantly with frame material, i.e. the steel-framed and reinforced-concrete-framed buildings studied have similar percentages of steel in their substructure/ superstructure/ non-structural categories, despite having markedly different absolute steel tonnages. This reflects the predominance of superstructure: regardless of frame material, it contains most steel, so changes elsewhere are small by comparison.

The distribution of steel within a typical building illustrates that non-structural steel can amount to one-third of a building's total. However such elements are often omitted from published analyses, possibly because their steel contents are difficult to calculate, or because they are installed by many different tradesmen reporting to different clients. A major difference between the non-structural and structural categories is that the former is frequently replaced (Treloar et al., 1999), meaning that over a building's lifespan the cumulative sum of non-structural steel is even greater — by using replacement rates from Scheuer (2003), non-structural components amount to almost half of a building's total lifetime steel use. This effect is not currently captured in some life-cycle analyses (LCAs) of buildings, e.g. Target Zero (2010), where the carbon emitted to operate a building over its life is calculated, but only the embodied carbon of the 'shell' is computed, neglecting fittings, furniture and their replacements. Including this material would give more accurate and comparable results for LCAs, correcting the current understatement of embodied impacts.

Though data for the typical building results are taken primarily from UK sources, this distribution is applicable worldwide because structural engineering principles are universal and local preferences will generally only affect non-structural components. Significant exceptions to this would be highly-seismic zones, such as Japan, where extra steel is required to ensure safe structures (Müller et al., 2011).

Future forecasts

Having an enhanced 'snapshot' of current steel use improves the accuracy of future forecasts. This is particularly true for the UK proportional results as they are largely time-invariant (global results are probably time-variant in the longer term as described in section 3.3.2). This will allow policy-makers and industry professionals to input more granular and accurate data into models of future steel flows. For example, should UK industry to decline, it would be expected that sections manufacturers would see a decline in that market, whilst rebar demand would not diminish to the same extent.

3.3.2 Time variance

The results are based primarily on data for 2006, during which the world economy and construction industry were experiencing strong growth. The economic recession

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since then has caused steel production to diminish (worldsteel, 2011a); hence absolute magnitudes quoted in the results are not valid for a more recent year. Do the proportional results, i.e. percentage of total steel use in a given sector, also vary with time? Analysis of proportional consumption of sections by sector in the UK since 1979 (BCSA, 2010) reveals that most sectors stay within $\pm 3\%$ of their average for the period (the two sectors that exceed this, industrial and public, display long-term decline and increase respectively). While a comparable worldwide study was not possible, analysis of Japanese steel construction data from Japan Iron and Steel Federation (2011) for the years 2007–2010 shows only small changes between building and infrastructure use, as does one for cement (a proxy for rebar) for 2005–2006 in the USA Portland Cement Association (2011). Therefore, the proportional steel use between sectors is unlikely to change significantly year-on-year for developed countries, regardless of economic recessions, or other time-variant effects.

As discussed in section 3.3.1, it is expected that developing countries will use less steel in infrastructure, once these networks are built up sufficiently, and hence the global buildings:infrastructure ratio will move closer to the UK one, with corresponding changes in sector proportions. Because development takes decades rather than years, the global sector proportions are probably time-invariant in the short-term, but variant in the longer term.

3.3.3 Future work on steel flow into construction

Given the prohibitive time and cost of gathering reliable data on steel use, the results represent the best possible estimate with currently available sources. Future work in this area would be to refine the results by acquiring more data. Three areas in particular would benefit from additional sources: rebar use; sheet/plate applications; developing nations. As stated in section 3.1.1, rebar allocation is based on cement use information — however the assumption that rebar and cement are always used proportionately is not true, hence further data would provide a more accurate result. As also indicated in section 3.1.1, sheet/plate has many and varied uses within construction, so allocation is based on disparate sources — improved information on the use of sheet would enhance estimates. The global results are based mainly on European and USA data. While the analysis of Turkey data suggests these sources might be representative of the world additional data from large developing nations, such as China and India, would test this and ensure results are truly global.

3.3.4 Implications for material efficiency

The findings of this chapter suggest that research on material efficiency potential in construction should focus on a typical building superstructure and on structural steel sections. For both the UK and the world, the results show that more steel is used in buildings than in infrastructure. For both boundaries industrial, commercial, office and residential buildings all are significant end-uses of steel. The typical building identified in section 3.2.3 is representative of the latter three categories, while superstructure is confirmed as containing the most steel — therefore material efficiency strategies will have significant impacts if applied to a typical superstructure. Utility applications are the largest infrastructure end-uses of steel, but table 3.1 notes that infrastructure has a range of structural types, tailored to each use, making findings for material efficiency in these products more difficult to apply generally.

While rebar and sheet are the largest uses of steel in construction, there is less information available on their use and therefore more uncertainty in their results (as noted in section 3.3.1). Additionally these products are usually used with other materials (concrete for rebar; sheet is used compositely with many materials such as insulation (cladding), concrete (decking) and glass (façades)) so examining opportunities for material efficiency in steel is more complicated as trade-offs must be examined. By contrast there is more information available about sections due to an international network of trade associations (through which findings can be dispersed), sections are discrete elements that are explicitly quantified on construction projects (by comparison sheet tonnage is seldom calculated), and sections can be more easily examined in isolation from the other structural elements. Therefore sections were chosen as the primary product for material efficiency studies in the following chapters.

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Chapter 4

Utilisation of structural steel in buildings

Of the three material efficiency strategies introduced in chapter 1, the first to be examined is ‘using less material by design’ — i.e. how much less steel could be used if structures were designed differently. Following the findings of chapter 3, the superstructure of multi-storey braced-frame buildings was selected for study. The review of literature in section 2.2 concludes that such research has not been previously published; although structural engineers are aware that they do not produce designs with a minimum of material, it is not known how much extra is actually added. Section 4.1 describes how ‘utilisation ratios’ were used to quantify the excess mass of steel in each beam and column within 23 commercially-designed buildings. Section 4.2 presents the results of this analysis, culminating with an estimate of the steel tonnage that is surplus to design requirement. The implications of these findings are discussed in section 4.3. This chapter is based on the journal article “Utilisation of structural steel in buildings” (Moynihan and Allwood, 2014b).

4.1 Methodology to analyse utilisation of structural steel elements

A methodology is proposed to analyse steel superstructure elements in building designs to infer the potential to reduce steel mass without adversely affecting performance. It is based on the concept of a ‘utilisation ratio’ for structural elements, already used in the industry, as described in section 4.1.1. The methodology is

applied to the beams (section 4.1.2) and columns (section 4.1.3) of 23 commercially-designed, steel-framed buildings supplied by three leading UK design consultancies. All factors influencing these designs were not known; therefore a verification process, explained in section 4.1.4, was used to ensure the results were reflective of the reality of each building's design.

4.1.1 Utilisation of structural steel

A 'utilisation ratio' (abbreviated to U/R; also called 'utilisation factor' or 'unity factor') is defined in equation 4.1 as the ratio of the actual to maximum allowable performance values, with examples of its use given for moment and deflection.

$$Utilisation\ Ratio = \frac{Actual\ performance\ value}{Maximum\ allowable\ performance\ value} \quad (4.1)$$

For example:

$$\begin{aligned} Moment\ U/R &= \frac{Maximum\ moment\ along\ element}{Section\ moment\ capacity} \\ Deflection\ U/R &= \frac{Maximum\ deflection\ along\ element}{Deflection\ limit} \end{aligned}$$

This ratio can be calculated for a range of performance requirements for steel elements (beams or columns). For any element, engineers are concerned with the highest U/R across all performance requirements, as this defines the most likely failure; for example, for bending moments, the numerator of equation 4.1 is the largest applied moment along the element (at ultimate limit state) and the denominator is the element's moment capacity. Design standards such as Eurocode 3 contain equations of this type for all performance requirements of interest, with specific calculation instructions and specify the maximum value of the ratio (usually 1) (BSI, 2005).

By determining the maximum utilisation of an element, a U/R also indicates its excess capacity — i.e. the material that is unnecessary. By analysing U/Rs for an entire building its total potential for steel saving can be estimated. For simplicity of calculation it was assumed that material requirements were directly proportional to U/R, whereas actually a section that is twice as stiff (hence has twice as much moment capacity or deflects half as much) does not necessarily have twice as much mass. To ascertain an average level of savings potential, a substantial dataset was assembled by collecting structural design data for steel-framed, commercially-designed buildings, supplied by three leading UK design consultancies. It was requested that

4.1 Methodology to analyse utilisation of structural steel elements

the buildings be reflective of ‘typical’ UK steel-framed buildings — ideally an office as identified in section 3.2, though schools and other building types were also accepted — excluding those with unusual geometries, very long spans or particularly onerous design conditions. In total, information on 12,787 beams and 2,347 columns of 23 building designs was obtained and analysed using 12 man-months of labour over a two-year period. As building designs are bespoke, this large dataset was necessary to facilitate statistical analysis to determine average values and prevent results being skewed by individual buildings. Other methods to estimate steel over-provision would have been to redesign each building entirely with a minimum-weight design criterion, or to do so incorporating variable-section elements (as described in section 2.2.1), however both of these options would have been impractically labour-intensive.

Six different design criteria were examined for each element: moment, shear, axial force, buckling, combined axial and moment buckling, deflection (exact details of checks done are supplied in appendix B, along with a sample calculation). These criteria were chosen because design guidance mandates that they are checked and because they can be meaningfully expressed as a U/R with maximum value 1.0. For each element, the U/R for each criterion was calculated at the most onerous point according to the design standard originally used for the parent building (Eurocode 3 (BSI, 2005) or British Standard 5950 (BSI, 2000b)). The highest U/R was then selected as the single, governing U/R for the entire element, and this used in subsequent analysis. In the majority of cases U/Rs were calculated by the software programmes used during design; for the remaining cases the completed calculations (supplied by the design engineers) were converted into U/Rs.

Data were assumed valid and accurate unless they indicated overload or bracing, or a U/R could not be calculated. Elements were omitted from the analysis if they had U/Rs in excess of 1.00 (i.e. over-loaded and thus failing) because it was assumed that such elements were later corrected to non-failing sections, but without knowledge of the new sections it is impossible to calculate a U/R. Elements designated as bracing (or those with U/Rs equal to zero, assumed to be bracing) were omitted because U/Rs are not meaningful for such elements. Elements for which there was not enough information to calculate a U/R were also omitted from the analysis. In total 2,657 beam and 510 column data were omitted leaving a valid dataset of 10,130 beams and 1,837 columns. Table 4.1 summarises the dataset assembled.

4. UTILISATION OF STRUCTURAL STEEL IN BUILDINGS

Building number	Building type	Floors analysed	Beams		Columns	
			total	omitted	total	omitted
1	office	3	186	39	52	2
2	hospital	2	802	23	156	9
3	school	3	106	3	30	0
4	school	2	62	0	21	0
5	office	1	21	0	15	0
6	office & education	3	1194	494	75	0
7	school	5	908	142	113	10
8	office	5	519	144	40	2
9	office	4	606	94	59	3
10	office	1	48	13	0	0
11	school	3	503	124	109	55
12	school	2	578	52	108	8
13	school	1	372	61	74	10
14	school	3	760	9	168	2
15	residential & retail	14	2230	783	215	147
16	mixed-use residential	6	536	172	215	154
17	mixed-use residential	8	947	316	164	99
18	office	2	316	116	57	0
19	school	3	527	28	151	1
20	school	2	322	8	96	1
21	residential	1	73	2	213	0
22	school	3	613	8	118	7
23	school	2	558	26	98	0
Totals		79	12,787	2,657	2,347	510

Table 4.1: Summary of building steel dataset

4.1.2 Utilisation of beams

A statistical analysis of the beam data was undertaken to draw conclusions about the utilisation of steel in each building, and also across the entire dataset to determine what ‘typical’ utilisation was. The valid U/R data for the beams in each building were averaged to produce an estimate of how much spare capacity the building had. U/Rs were then averaged by beam length and mass to determine variance with these characteristics. The frequency of occurrence of each U/R within the building was calculated and graphed, grouping U/Rs in 10% bands for clarity. It was assumed that beams which were explicitly designed would have U/R greater than 0.8 (i.e. a competent engineer would not design below this level), so this proportion was calculated for each building. Beams with U/R below 0.2 are unlikely to have been

4.1 Methodology to analyse utilisation of structural steel elements

designed, so this proportion was also found. As buildings are designed floor-by-floor, U/Rs were also averaged and graphed by frequency for floors with large numbers of beams — beams from small floors or otherwise miscellaneous were classified as ‘other’ when reported and are included in the building ‘total’ graph line, always marked in black. It was noticed that many floorplates used a small number of the sections available in Steel Construction Institute’s (SCI) standard catalogue (SCI, 2009) so the five most common section sizes were identified for each floor and their proportion of the total number of beams on that floor calculated.

To understand how U/Rs varied across the layout of each floor of each building, plots were produced showing the U/R of each beam, an example of which is shown in figure 4.1. This plot is a plan-view of a floor, just showing the beams and column locations. The beams are coloured according to their U/R as indicated in the legend of the figure. The thickness of each line is proportional to the beam’s linear weight (kg/m). Where a beam’s U/R was not available or omitted, its line was coloured grey. For many buildings the beam location dataset was incomplete; in these circumstances beams were manually added in thin grey lines to convey the floor geometry and beam layout. An indicative dimension shows the scale of each floor. Sufficient location data were available to produce 43 such plots for floors of 17 buildings, a selection of which are contained in appendix C.

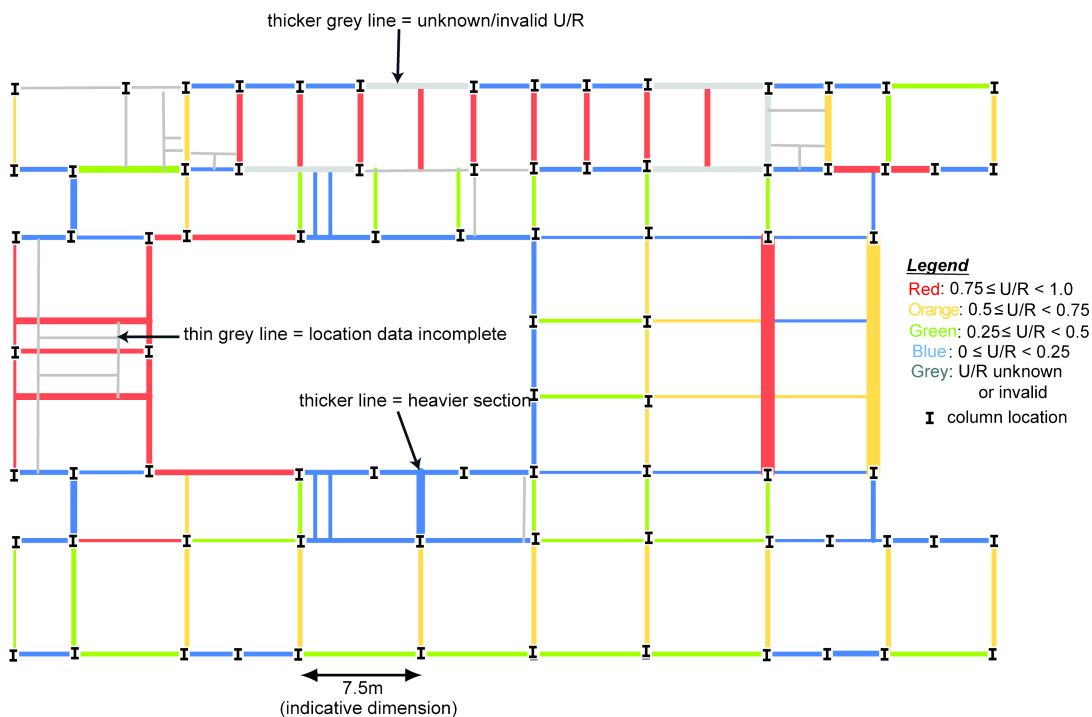


Figure 4.1: Example plot of floor showing U/R and section weight

4.1.3 Utilisation of columns

U/R data were also available on the design of columns in 22 of the buildings. Average U/Rs were calculated and graphs showing the frequency of U/Rs plotted. However no data were available on column mass, length or location which prevented further analysis of these elements.

4.1.4 Verification processes

The six design criteria analysed do not form an exhaustive list; there are other criteria which influence the size of element chosen. To determine the reasoning behind the designs and hence understand these omitted criteria, a semi-structured interview was held with a structural engineer for each building. Each engineer was asked the questions shown on the interview template in figure 4.2, derived from discussions with experienced industry professionals.

4.1 Methodology to analyse utilisation of structural steel elements

Utilisation study: questions for interviews	
Interviewee:	Building name (#):
Email:	Emailed in advance on: / /
1. What do you think the average U/R for the building is? 2. At what stage was job handed over? I.e. what stage was model at? a. Was model handed to fabricator? Any idea if tonnage up or down for construction? 3. This study was based on the Fastrak model, which included moment, shear, axial, buckling, combined cases and deflection; a. Are there any other design cases that governed? If so where? i. Vibration ii. Construction loads iii. Section depth – if so how onerous? iv. Connections b. Were any design limits more onerous than included in model? i. deflection limits ii. other? 4. How was robustness (disproportionate collapse) accounted for in design and how do you think this would impact on utilisation? a. Were any beam sizes changed for robustness specifically? 5. Anything else that will impact my study?	

Figure 4.2: List of questions used when interviewing building designer

The UK standard catalogue of hot-rolled structural steel sections (SCI, 2009) was analysed to determine the reduction in capacity between consecutive sections — a practical limitation on the U/R achievable in design. Moment, shear and axial capacity were calculated using Eurocode 3-1-1 (BSI, 2005) for each Universal Beam and Universal Column section; bending stiffnesses were also calculated. These four properties were compared with those for the beam one size larger and the reductions in each calculated; the average reduction was calculated using the largest of these reductions.

4.2 Results from analysis of steel utilisation

The methodology described in section 4.1 was applied to the data from the 23 steel-framed buildings. A selection of these results are detailed in appendix C; the full results are available in the Supporting Information document, containing 88 figures and 23 tables, that accompanies the journal article. A summary of the results is presented in section 4.2.1, revealing that steelwork is not utilised as expected. The reasons for this are explored in section 4.2.2, where a practically-achievable average U/R for buildings is also proposed.

4.2.1 Utilisation results

Table 4.2 gives a summary of the results of the analysis of U/Rs of beams in 23 buildings, with results averaged across the buildings also. The average U/R across all projects is 0.40, with a range of 0.15 to 0.90. The average rises when weighted by length — to 0.48 — and again by mass, to 0.54, with a range of 0.25 to 0.96. On average, twice as many beams have U/Rs less than 0.20 as do greater than 0.80, while the 5 most common beam sections in a building account for three in every four beams typically. Results for the average U/R of columns in each building are also listed in table 4.2, showing an overall average of 0.49 with a range of 0.12 to 0.72. The large ranges found for each result set are caused by outliers; most results are close to the mean, with standard deviations less than 15% for all datasets in the table.

Frequency graphs were plotted for all buildings. Figure 4.3 displays four graphs that exhibit the following patterns found across the entire dataset: a large ‘spike’ at low U/R with a smaller spike between U/Rs of 0.6 and 0.9 (figure 4.3a); this spike pattern does not vary significantly depending on floor, although roofs more frequently have larger low U/R spike (hence lower average U/R) despite often containing the largest number of beams (figure 4.3b); the building ‘total’ line is often skewed towards low U/Rs because this line includes the ‘other’ beams from small floors and miscellaneous areas which have consistently lower U/Rs (figure 4.3c). Graphs of frequency against U/R for columns have a flatter frequency distribution (i.e. less prominent spikes) than for beams typically (figure 4.3d).

4.2 Results from analysis of steel utilisation

Building number	Beams analysed	Avg. U/R for beams			% of beams with U/R		5 most common beams %	Columns avg. U/R
		by number	by mass	by weight	≤ 0.2	> 0.8		
1	147	0.36	0.47	0.43	39%	7%	66%	0.31
2	779	0.58	0.62	0.68	16%	30%	68%	0.60
3	103	0.25	0.32	0.47	65%	4%	90%	0.12
4	62	0.17	0.37	0.62	75%	5%	66%	0.13
5	21	0.44	0.41	0.41	0%	0%	100%	0.64
6	700	0.15	0.21	0.25	69%	0%	77%	0.42
7	766	0.33	0.39	0.45	42%	9%	62%	0.47
8	375	0.31	0.39	0.39	43%	7%	81%	0.72
9	512	0.37	0.49	0.50	38%	16%	49%	0.60
10	35	0.90	0.93	0.96	0%	83%	100%	-
11	379	0.64	0.70	0.68	12%	45%	71%	0.69
12	526	0.47	0.56	0.60	32%	17%	64%	0.49
13	311	0.39	0.48	0.49	38%	17%	75%	0.52
14	751	0.26	0.32	0.38	56%	3%	65%	0.54
15	1447	0.18	0.27	0.37	75%	4%	74%	0.62
16	364	0.23	0.37	0.46	51%	9%	95%	0.57
17	631	0.52	0.65	0.70	26%	35%	81%	0.60
18	200	0.54	0.68	0.66	27%	40%	97%	0.12
19	499	0.36	0.39	0.43	50%	15%	71%	0.49
20	314	0.33	0.47	0.66	58%	25%	74%	0.35
21	71	0.55	0.54	0.61	0%	17%	99%	0.65
22	605	0.47	0.58	0.63	28%	20%	78%	0.55
23	532	0.35	0.44	0.50	50%	11%	87%	0.60
Average	440	0.40	0.48	0.54	39%	18%	78%	0.49

Table 4.2: Results of the analysis of utilisation ratios for beams and columns in buildings

4. UTILISATION OF STRUCTURAL STEEL IN BUILDINGS

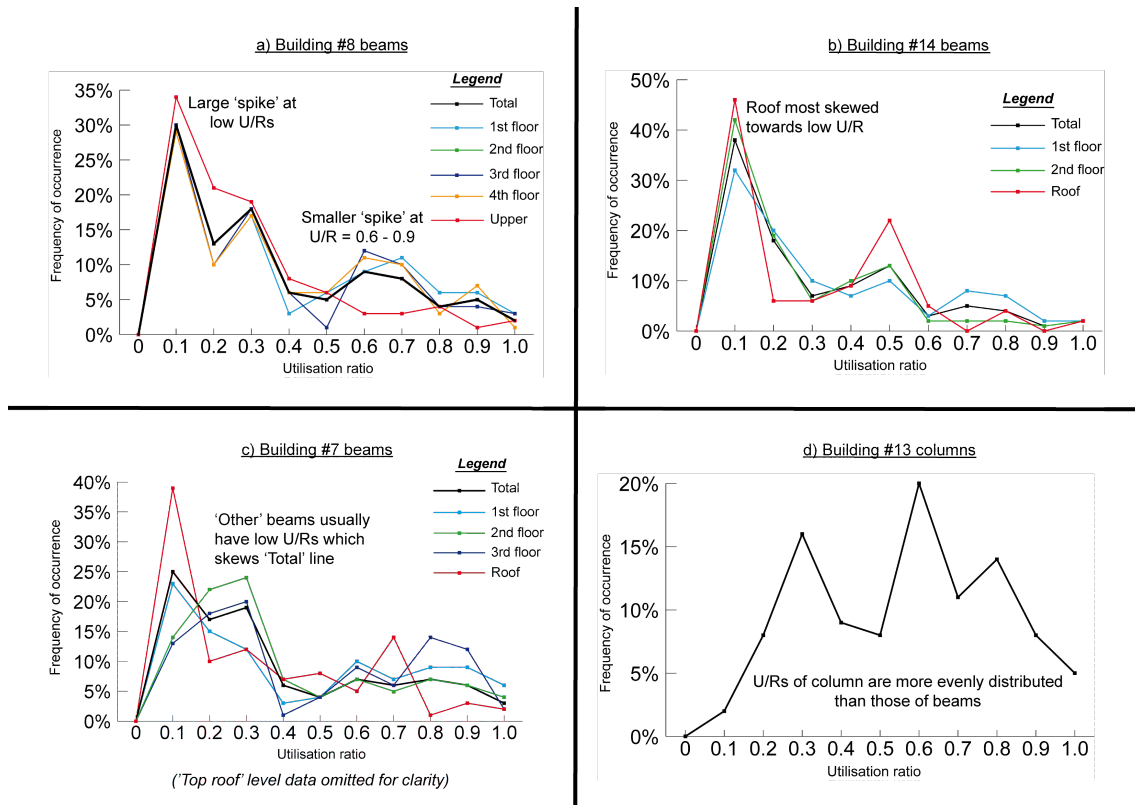


Figure 4.3: Four of 45 graphs of frequency occurrence against utilisation ratio, three for beams (by floor and overall) and one for columns, displaying patterns found across all buildings

Figure 4.4 shows four floor plots of U/R and beam mass which exemplify the patterns found across the 43 plots produced: beams with high U/R (coloured in red) are generally located towards the middle of buildings and tend to have larger section sizes (i.e. thicker line weights), while beams with low U/R (coloured in blue) are lighter (thinner lines) and situated more often around the periphery (figure 4.4a & b respectively); beams of the same section size (i.e. line thickness) near to one another often display a range of colours (figure 4.4c); whether a beam was loaded by other beams (i.e. whether it was a primary beam or not) did not appear to correlate strongly with its U/R (figure 4.4d).

4.2 Results from analysis of steel utilisation

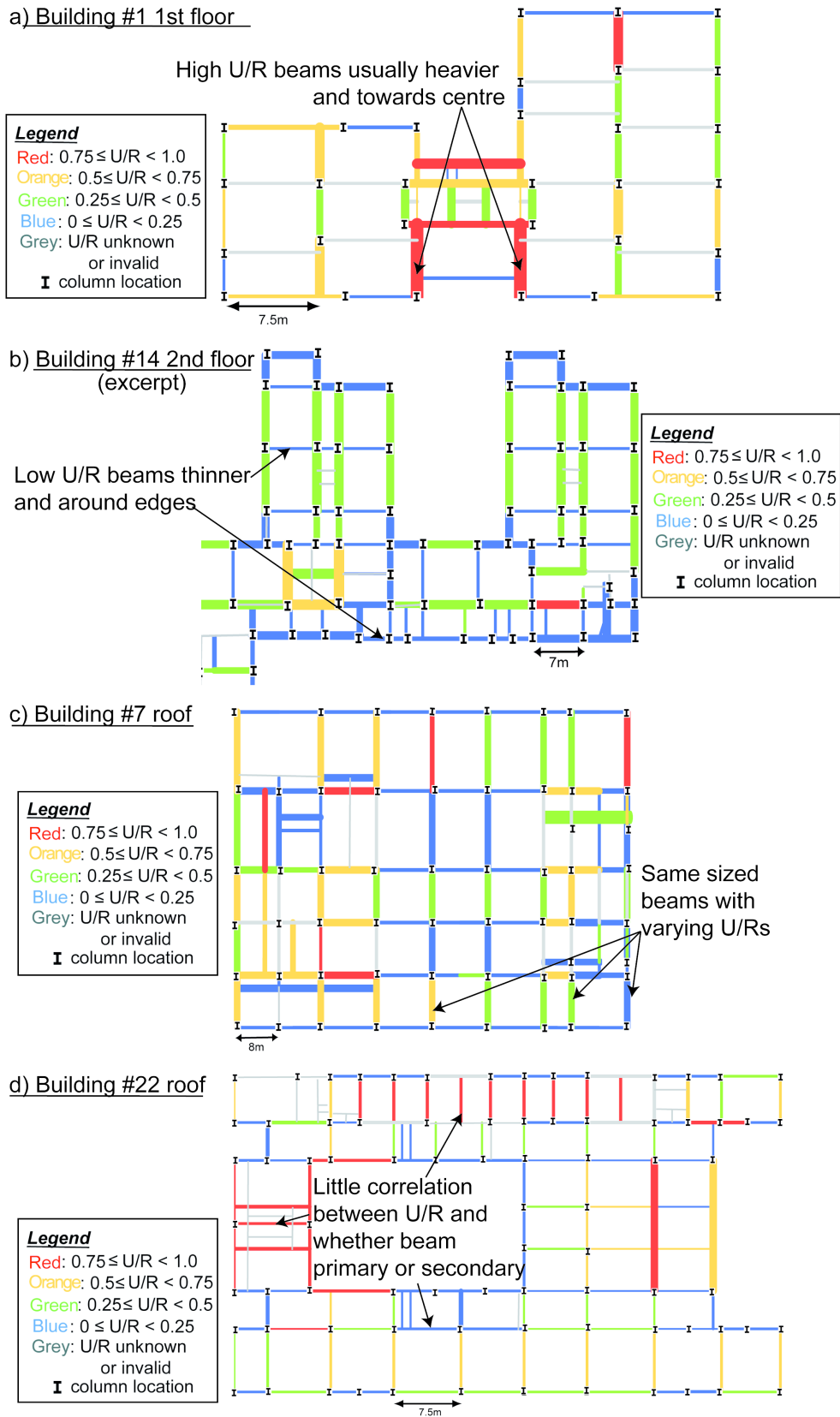


Figure 4.4: Four of 43 plots that indicate beams' U/R and section weight. These examples show typical patterns found across entire dataset.

4.2.2 Verification of results

Table 4.3 summarises the responses from the interviews with designers; further details are noted alongside the relevant building results as shown for the examples in appendix C. All building information was at or close to the construction stage, and nowhere did robustness significantly impact element sizes. Each building was designed to a specific client brief for a unique site at a precise time, but nonetheless recurring themes were present in the responses. As can be seen in the table, engineers reported that floor vibration requirements (a criterion not included in the analysis) governed beam design in large areas of 3 buildings and small areas of a further 6 design, thus affected elements' true U/Rs will be higher. However removing the former three buildings from the analysis actually *decreases* average U/R to 0.39 and average U/R by mass to 0.52, suggesting that vibration does not directly lead to low U/Rs.

Cost concerns featured prominently in the interview responses: designers deliberately repeated section sizes for economies of scale during procurement and to reduce mix-ups on site, and used larger sizes than necessary to facilitate easier connection and hence reduce labour requirements during construction. Despite this, most designers were surprised that average U/Rs were so low: a number of them suggested that maximum depth limits on beams lead to low U/Rs however one engineer noted that a shallow section should still have a high U/R despite being heavier than a deeper section.

The results from analysis of the catalogue of steel sections revealed that on average a Universal Beam section has 85% of the capacity of the beam one size larger, whereas for a Universal Column section the corresponding value is 81%. Assuming that U/Rs are uniformly distributed between these values and 1.00, an average U/R of 0.9 is therefore possible using this catalogue.

4.3 Implications of results

The surprising results of section 4.2 reveal that buildings could be designed with around half the steel used at present and still deliver specified safety and service levels to occupants. The results point towards rationalisation as the main cause of this over-specification of steel beams in construction, and this has implications for three groups within the construction industry: designers; contractors and fabricators; clients, standards committees and policy-makers.

Building number	Criteria that governed steel design:			Further detail and comments
	vibration?	construction?	section depth?	
1	No	Yes	No	Torsion during construction; larger beam sizes the cheapest solution
2	No	No	No	No special criteria
3	No	No	No	Deflection governed mainly
4	No	No	No	Deflection governed mainly
5	No	No	No	Applied loads reduced but too late to redesign
6	Small areas	Yes	Small areas	Perimeter beams governed by vibration or connection depth
7	Small areas	No	No	Mainly stress & deflection governed
8	No	No	No	Many omitted beams were fabricated bespoke
9	Small areas	No	No	Many omitted beams were fabricated bespoke
10	No	No	No	Had time to design thoroughly and no late changes
11	No	No	No	Steelwork rationalised to reduce procurement costs
12	No	No	Yes	Building geometry optimised to reduce façade and heating costs
13	No	No	No	Steelwork rationalised to reduce procurement costs
14	Small areas	Yes	Small area	Torsion during construction; larger beam sizes the cheapest solution
15	No	No	No	Complex procurement procedure increased steel tonnage
16	No	No	No	Complex procurement procedure increased steel tonnage
17	Large areas	No	Yes	Shallow beams used to minimise cladding costs
18	Large areas	No	Yes	Shallow beams used to minimise cladding costs
19	No	No	No	Deflection governed most beam designs
20	Large areas	No	No	Vibration governed in many areas
21	Small areas	No	Yes	Standardised beam depths for architectural reasons
22	No	Yes	No	Steelwork rationalised to allow faster construction
23	One area	Yes	No	Sizes repeated to allow faster construction

Table 4.3: Summary of responses from interviews with designers

4.3.1 Evidence for rationalisation as the cause of low utilisation

The result that the average utilisation of beams weighted by mass is 0.54 implies that up to 46% of the buildings' combined beam mass is not necessary (relative to the design standards). An average U/R of 0.9 is possible with the discrete catalogue of section sizes, thus 36% of a building's beam mass could be removed with no loss in safety or service. Comparing the results from section 4.2 with the descriptions of rationalisation in section 2.2, the following observations are made:

- low variation of section sizes in a floor or building — the five most common sections make up almost three-quarters of the beams in a building, and usually over 80% of the beams on a floor, regardless of area; this suggests repetition in design;
- low frequency of high U/Rs — on average fewer than one beam in five has a U/R greater than 0.8 (the assumed lower bound for elements explicitly designed by an engineer) which suggests that most beams are not explicitly designed;
- sections being copied across a floor — the floor plots (e.g. figure 4.4c) showed high and low U/Rs for beams with the same section size near each other, supporting the previous point that beams are copied across from the most onerous scenario (explicitly designed) to less onerous ones (which then have low U/R);
- recognition by designers that cost concerns cause repetition of section sizes and use of larger section sizes than necessary.

These four points indicate that rationalisation is occurring and is the most likely cause of U/Rs being lower than expected. Interestingly there appears to be no correlation between the number of beams in a building and its average U/R — rationalisation affects big buildings as much as small ones.

The average U/R by piece for columns is 0.49 compared with 0.40 for beams — it is not clear why columns are more highly utilised by piece than beams. It may be they are simpler to design, being mainly axially loaded, or that there are fewer of them, or that this analysis included all design criteria for columns and that beams' average U/R would appear to be higher if omitted criteria such as vibration were included.

4.3.2 Implication of results for designers

The interviews confirmed that engineers are responding to clients' requests for a low cost building by following the rationalisation guidelines outlined in section 2.2. However designers are not aware that rationalisation is adding 40% extra steel mass to the building, four times what Needham (1971) suggests and double what Gibbons (1995) recommends. While it is not clear whether this level of rationalisation is economically beneficial, it is certainly contributing unnecessarily to carbon dioxide emissions through production of the over-provided steel.

Rationalisation can be reduced by at least two methods: by increasing the time engineers have to design buildings, or by greater use of existing steelwork design and optimisation software. Both strategies involve extra cost but reductions in steel mass may offset these, particularly as weight savings compound — i.e. a lighter roof requires less column material to support it and thus smaller foundations. Benefit can be maximised from both strategies by: targeting areas of low U/R such as roofs, half-levels and 'miscellaneous' elements; focusing on beams before columns (as beams contain three times as much steel mass as columns as found in section 3.2); checking light or short beams (that average U/R by length and mass are higher than by piece indicates these beams have low U/Rs). Simply calculating the average U/R for a building may spur designers to increase it in an economic way — Needham (1971) notes the satisfaction engineers derive from finding an optimum design. It is noteworthy that building #10 achieved an average U/R in excess of 0.9 within existing commercial constraints.

4.3.3 Implications for contractors and fabricators

Literature on economic design, reviewed in section 2.2, reveals that many rationalisation measures are motivated by fabrication and construction considerations. Thus increased use of technology in the fabrication factory and on site could reduce the incentive to rationalise. Increased automation and flexibility of fabrication could reduce the amount, hence cost, of labour to fabricate sections and allow many different section sizes to be processed with little additional cost. Increased use by contractors of information technology on site (such as radio tags described by Ikonen et al. (2013)) could reduce the motivation to standardise beam sizes if there is less risk of pieces being placed or installed incorrectly. Cost structures within the industry may need adjustment to ensure profit and material reduction motives are aligned — for instance fabricators have no incentive to reduce steel mass when paid per tonne,

but might be rewarded with a proportion of the material cost saved. Rationalisation by fabricators and contractors (after that by the designer) is not included in this study; a number of interviewees remarked this rationalisation is anecdotally larger than that done by designers, but without ‘as built’ information for each building it was not possible to verify this.

4.3.4 Implications for clients, standards committees and policy-makers

As noted in section 4.3.2, designers respond to the instructions of their clients so environmentally-minded clients, or those who simply do not like waste, could reduce excess material in the buildings they commission by specifying a minimum average U/R. A value in the region 0.70–0.90 will give environmental benefit whilst being practically achievable. Clients can be motivated by sustainable building schemes, so these should consider adding credits for achievement of minimum average U/Rs as incentives to reduce rationalisation. Designers usually work within design standards, so those who set them should consider not just having a maximum U/R for elements (as currently is the case) but also a minimum U/R, or a target U/R range below which justification is required; local authorities might also make planning permission contingent on similar metrics being achieved. An initial step for the latter three parties may be to mandate reporting of average U/R.

4.3.5 Potential benefits and future research

The 23 buildings studied cumulatively contain 2,823 tonnes of steel. Presuming the average U/R by mass for this steel could be raised from 0.54 to 0.90, this would have avoided use of 1,027 tonnes of steel. This level of excess steel is double what Gibbons (1995) estimated the upper threshold for net saving, therefore further research is required to understand the economics of rationalisation. Specifically it is not known how much more design time is required to achieve a percentage increase in average U/R, nor what the extra cost of fabricating and constructing this design would be, nor how these extra costs compare with the saving in material cost and with the overall project cost. An estimate of the financial rewards from higher U/R can be obtained by assuming a steel price of £400/tonne (Allwood et al., 2010a), meaning the buildings studied could have saved an average of £18,000 each. This is a substantial amount, enough to pay for approximately 45 days of an engineer’s

time, and therefore potentially enough to merit the additional design, fabrication and construction costs.

As outlined in section 2.2, further steel savings are possible, for instance by using variable depth sections as described by Carruth (2012). This would reduce steel mass by another 30% per element, implying that 55% ($=1-(1-0.36)*(1-0.3)$) of structural steel in buildings could be removed. Therefore efforts are also required to manufacture sections using Carruth's method and install them in structures.

4. UTILISATION OF STRUCTURAL STEEL IN BUILDINGS

Chapter 5

Strategies for long-life and reusable structures

The second material efficiency strategy applied to construction is ‘using products for longer’. To extend the life of a product, it first must be understood why that life is currently curtailed — the review of literature in section 2.3 duly identifies four general causes of failure that apply to all products. Design strategies can avert or mitigate these causes but a related set of strategies has not previously been published. Section 5.1 proposes a methodology to identify a corresponding framework of strategies to extend product life, with the results presented in section 5.2. This research is based on the published journal article “Component level strategies for exploiting the lifespan of steel in products” (Cooper et al., 2013). *(This article was co-written with Daniel R. Cooper and Alexandra C. H. Skelton, both of the Low Carbon and Materials Processing Group at the Department of Engineering, University of Cambridge. Contributions to this chapter that are the outcome of their work, and not my own, are clearly referenced in the text; specifically these are seven of the twelve case study interviews detailed in appendix D).*

Where it is not possible to extend the life of the whole structure it may be possible to extend the life of individual components by reusing them — the third and final material efficiency strategy for construction presented in chapter 1. The review of literature in section 2.3 finds that design for deconstruction strategies are a key enabler of reuse, and that research is needed to identify commercial advantages (‘co-benefits’) of these strategies that might encourage their use. Section 5.3 presents a methodology to investigate co-benefits of design for deconstruction, with the results presented in section 5.4 and implications discussed in section 5.5. The work presented in these latter three sections is based on the conference paper “Deconstruction

and reuse: realities of design, commerce and logistics for portal frames” (Moynihan and Allwood, 2012b).

5.1 Methodology to identify strategies to address failure modes

The literature reviewed in section 2.3 revealed four types of failure in products, but a corresponding set of design strategies to avert these failures has not yet been published. In order to identify strategies to address product and component failure, a set of twelve ‘case study interviews’ with industry and academic experts (who are experienced in extending the lifespan of products and components) was conducted. Each interview covered the causes of failure and the technical strategies that had been, or could be, applied to extend product or component life. Table 5.2 provides a summary of the interviews conducted across all steel-intensive industries — not just construction — to find the widest possible group of solutions. Transferable lessons were gleaned from each case study and the resulting strategies were tailored to the four types of failure selected in section 2.3: degraded, inferior, unsuitable and worthless. Definitions of these failure types are given in table 5.1 — a repeat of table 2.1 from section 2.3 — with examples specific to construction added.

	Degraded	Inferior
The performance of the product has declined...	... relative to when it was bought	... relative to what is currently available
	e.g. dilapidated building; unsafe bridge	e.g. bridge with piers where engineering knowledge /technology now enables single-span
	Unsuitable	Worthless
The desire for the product has changed...	... in the eyes of its current user	... in the eyes of all users
	e.g. building with enclosed offices where open-plan now desired; traffic loads in excess of bridge capacity	e.g. oil rig in unproductive oil field; building in derelict area

Table 5.1: Product failure framework, from Cooper et al. (2013) (information repeated from table 2.1 in section 2.3)

5.1 Methodology to identify strategies to address failure modes

Case study	Sector	Interviewee/source	Interviewer ^a
1. Refurbishing modular buildings	Construction	Technical Manager, Foreman's Relocatable Building System	MCM
2. Steel rolling mills: replaceable work roll sleeves	Industrial equipment	Technology Manager, Siemens VAI	DRC
3. Adaptable foundations	Construction	Director, Arup	MCM
4. Adaptable, robotic packaging equipment	Industrial equipment	A fast moving consumer goods manufacturer	ACHS
5. Durable infrastructure	Construction	Professor, Cambridge University	MCM
6. Hard-wearing rails, replacing rails & resurfacing tram rails	Construction	Programme Manager, Network Rail	ACHS
7. Carbon-fibre aircraft body	Transport	Technical Fellow, Boeing	MCM
8. Restoring supermarket equipment	Metal goods	Development Manager, Tesco	DRC
9. Office block refurbishment	Construction	Associate, Expedition Engineering	MCM
10. Steel mill upgrade	Industrial equipment	Senior academic, Manchester Business School	ACHS
11. Upgradable washing machine	Metal goods	Director, ISE appliances	DRC
12. Component reuse of oil rigs	Construction/ Industrial equipment	Project Director, Able UK	ACHS

Table 5.2: Interviews undertaken to investigate lifespan extension strategies

Note:

a. Key to interviewers:

ACHS — Alexandra C.H. Skelton

DRC — Daniel R. Cooper

MCM — Muiris C. Moynihan

5.2 Strategies to prolong structure life

Details of each case study interview can be found in appendix D. From the interviews it became apparent that knowledge of the anticipated failure mode determines the type of life extension strategy that can occur. When the cause of failure can be foreseen, measures can be taken to ‘design-out’ the features that cause failure. For example water ingress at joints is a cause of reinforcement corrosion, a cause of bridge failure; therefore reducing the number of joints in a bridge and/or using stainless steel (i.e. corrosion-resistant) reinforcement in susceptible locations can prevent this failure (described in case study 5). When the exact failure is less certain, or when design-out solutions do not exist, features can be incorporated into the design that prevent product failure by providing sufficient flexibility to adapt or replace elements. These strategies are referred to as ‘design-in’ strategies; examples are modular systems that allow parts to be upgraded (case study 1) or open architectures that allow portions to be changed without overly affecting the whole structure (case study 10). Design-in strategies can be enhanced by monitoring the condition of structure in use to inform the rate, location and types of interventions. The interviews revealed that maintenance strategies are the same as design strategies but applied during the product’s life — i.e. maintenance operations aim to restore (case study 6), upgrade (case study 11) or adapt components (case study 3). Figure 5.1 shows the strategies and their relevance to each of the four failure modes, with examples chosen from the construction industry for the major buildings and infrastructure sectors identified in chapter 3. As noted in section 2.3, buildings usually fail because they become ‘unsuitable’, therefore should be designed with flexibility or adaptability strategies. Infrastructure usually fails when it is ‘degraded’ therefore engineers should examine ways to include durability or restore-ability in their designs.

5.3 Methodology to investigate co-benefits to deconstructable designs

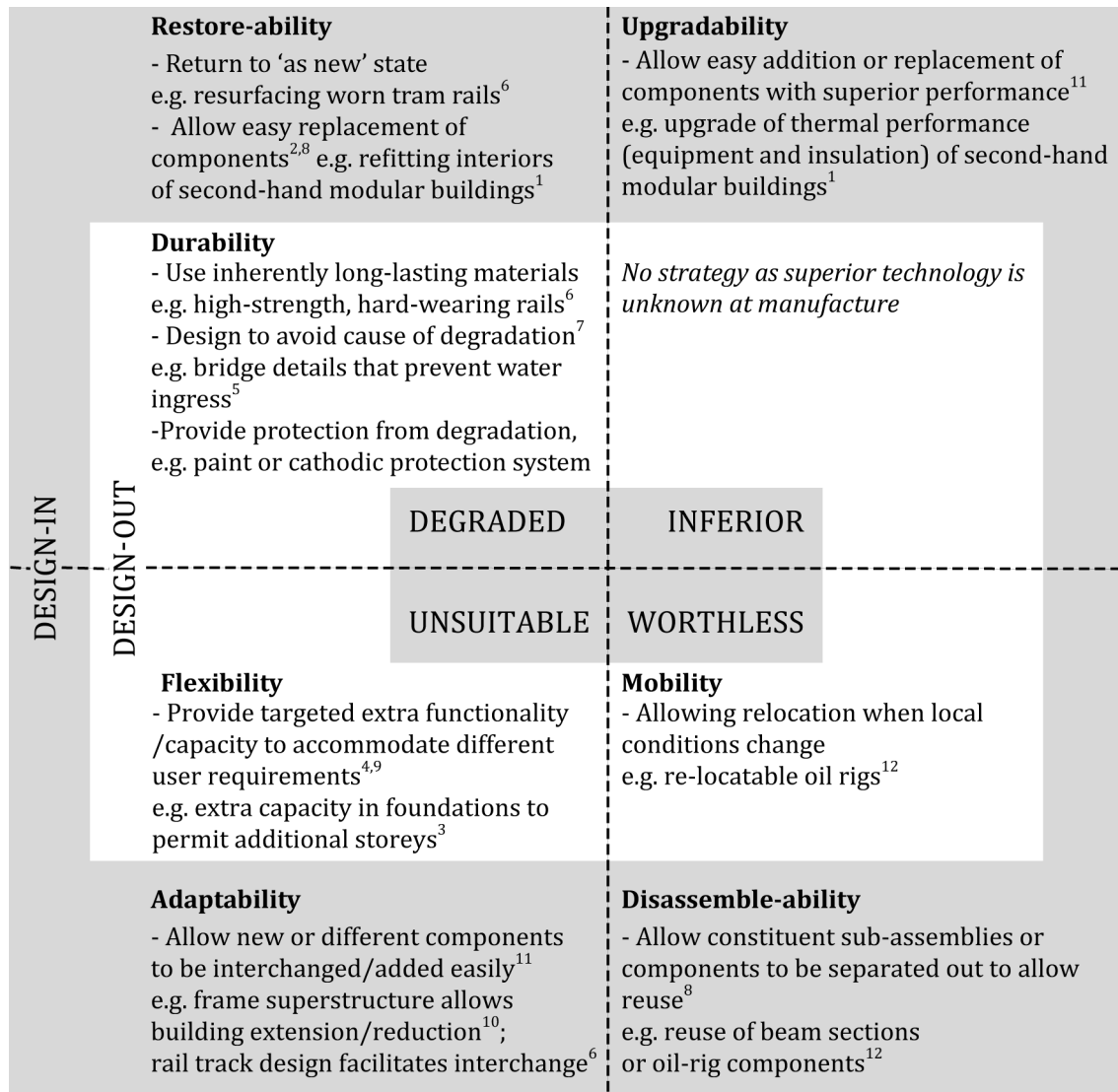


Figure 5.1: Targeted strategies to address product and component failure, adapted from Cooper et al. (2013)
 Superscript numbers refer to relevant case studies in appendix D.

5.3 Methodology to investigate co-benefits to deconstructable designs

As found in section 2.3, there is potential to reuse a much greater proportion of steel sections exiting the building stock than currently happens. A lack of supply is the main barrier, caused by a preference for demolition (which damages sections, making reuse impossible) over deconstruction (which removes elements intact) because demolition is quicker and less risky. One method to overcome these challenges is to design for deconstruction (DfD) but this is not practised as it is more costly. Two

5. STRATEGIES FOR LONG-LIFE AND REUSABLE STRUCTURES

studies were undertaken to ascertain which aspects of DfD also give commercial advantage (‘co-benefit’) to the client: a series of interviews (described in section 5.3.1) and a working group developing a deconstructable concept (section 5.3.2). Both studies addressed a single-storey supermarket building because such portal frame structures are large uses of steel, as found in chapter 3. Additionally, this type of building already incorporates some of the DfD strategies identified in table 2.5 of section 2.3 — separated systems, accessible structure and mainly reversible (bolted) connections — so would require fewer changes to make deconstructable.

5.3.1 Interviews to investigate co-benefits

To ascertain which designs offer both initial and deconstruction benefit, a series of semi-structured interviews was conducted with nine construction industry professionals with knowledge or experience of DfD. Table 5.3 lists the nine professionals, identified by their role, company and sector within construction. Although the reference building in each case is a single-storey supermarket, built project examples were discussed for all building types.

Job Title	Company	Sector
Associate Director	Longcross	Contractor
Structural engineer	Construction Howarths	Timber designer & fabricator
Chairman	Buildoffsite	Trade association
Managing Director	B&K Structures	Designer & fabricator
Structural Engineering	Longcross	Contractor
Researcher	Construction	
Managing Director	Accio Group	Temporary/modular building fabricator
Business Development & Technical Manager	Powerwall	Façade and modular fabricator
Technical Director	Kingspan	Façade and modular fabricator
Project Manager	Tesco	Retail client

Table 5.3: List of interviewees on design for deconstruction principles and limits

Each interviewee was asked relevant questions from the list given in table 5.4 and supplementary questions asked extemporarily following the interviewee’s answers. Recurring ideas were identified in the responses and then grouped into themes.

5.3 Methodology to investigate co-benefits to deconstructable designs

Have you built a deconstructable building before?
What made it deconstructable?
What was the client's motivation?
Would you want to do again? Under what conditions?
What was different to a 'normal' project?
Any issues with regulations/standards/certification? Robustness?
What techniques/features made it easier/harder?
Was there any element of standardisation? What was this governed by?
What were the joints/connections?
How was the cladding attached? What were the foundation connections?
Were there any wet trades?
How long was building up for? Could have lasted longer? What is the limit?
Has the building since been deconstructed? Was this entirely successful?
What was different?
If you were commissioned to design a deconstructable building, what features would it have?
What emerging/innovative/unusual technologies could be used to deliver a deconstructable building?
What co-benefits are there to deconstructable design?
Any other comments?

Table 5.4: List of questions for design for deconstruction interviews

5.3.2 Working group to investigate deconstructable concept supermarket

Subsequent to the interviews, a working group was set up with a major UK retail client to formulate a deconstructable single-storey supermarket building. The group consisted of a structural engineer, architect, cost consultant, contractor and the client. It met six times in nine months with the aim of developing a technically feasible and commercially viable concept design for a deconstructable supermarket. All structural components and related activities on site were examined, from preparatory works through to floor finishes, and potential suppliers interviewed. The findings from the interview series were input to this process, and all existing technologies were considered at the outset before converging on a single design. This final design was costed in comparison to a traditionally-built supermarket and presented to the client with the aim of securing funding to develop the design further, leading to its construction. A substantial dataset was assembled from this process consist-

ing of the minutes of meetings, the reports, presentations and designs prepared by different members and suppliers, and the notes kept on informal conversations in between meetings. The final design concept was analysed along with this dataset to determine the major co-benefits of DfD.

5.4 Results: benefits and barriers to deconstructable designs

Section 5.4.1 outlines the three major co-benefits for DfD identified from analysis of the interview and working group data. Further detail on barriers to DfD was obtained, given in section 5.4.2.

5.4.1 Designs that give initial benefits and also aid deconstruction

Analysis of the interview and working group data found the same three major co-benefits of DfD for clients: adaptability, programme savings and reduced risk. However the working group concluded that adaptability was the largest co-benefit, while interviewees frequently asserted that programme and risk savings would be more beneficial to clients. The interviews and working group proposed similar solutions for a deconstructable supermarket: flexible designs with prefabricated and modular elements. The working group's design concept is described below before each of the three co-benefits is discussed.

Deconstructable supermarket concept

The working group output was the concept design shown in figure 5.2. Figure 5.2a shows a plan view of the column-free retail space (entirely serviced from above) with modules along the sides and back containing prefabricated retail (e.g. bakery, deli, seasonal), facilities (canteen, toilets) and storage areas. Figure 5.2b shows an elevation identifying each of the systems for the fabric. The walls and roof consist of prefabricated panels bolted to a structural steel frame (itself entirely bolted together) which permits the store to be extended/reduced as necessary; steel was selected as the most efficient material to span the distances required. Glazing is provided in a prefabricated, unitised system bolted to the frame. The floor systems consists of prefabricated (although with finishes installed on site) planks spanning between

5.4 Results: benefits and barriers to deconstructable designs

pretensioned, precast concrete ground beams. Using block-paving for the carpark would allow the entire site to be cleared once the supermarket reaches end-of-life.

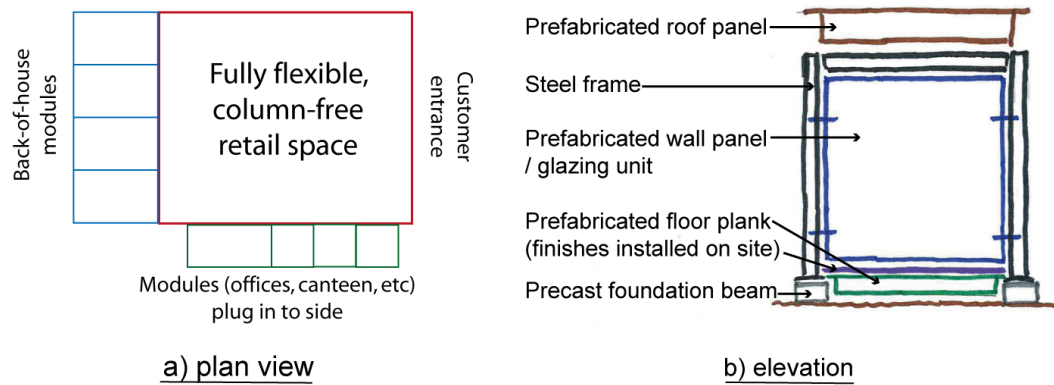


Figure 5.2: Deconstructable supermarket concept a) plan view; b) elevation showing different systems

Adaptability

Adaptability is the ability to alter building layout, either during construction or during use; adaptable designs reduce the cost, time and risk in doing so. Deconstructable components are inherently adaptable as components can be removed and upgraded; modular systems are particularly suited to rapid interchange. The working group noted that supermarkets change their layout regularly to gain competitive advantage and expect these changes to become more frequent. Additionally, late changes to the layout during construction is often the cause of extra work and delays on site. Therefore a building allowing quicker, easier and cheaper changes would give a commercial advantage.

The working group concluded adaptability was the biggest co-benefit of DfD and was central to the group's building concept shown in figure 5.2: fully-flexible retail space largely removes retail considerations from the construction programme, while servicing the space from above, particularly by suction drainage (also called 'vacuum drainage') negates the need to break up the floor when changing retail cabinet locations; modules along the back and sides can be changed to bring in new retail zones or facilities; wall, glazing and roof panels can be quickly changed to 'refresh' the store aesthetic or alter thermal performance.

Interview responses characterised adaptability as a lesser co-benefit to DfD. Three interviewees noted that structural frames can be extended with less interference to the existing building than load-bearing façades and suggested that 'clip' joints exist

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that would make panel interchange particularly quick. One respondent recognised that traditional construction can incorporate many adaptable features however the presence of non-reversible connections (especially in concrete) and non-standardised interfaces or designs inhibits this.

Programme savings

Programme savings (i.e. reduced construction time) can be achieved by using prefabricated components, either 2D ('flatpack') or 3D ('volumetric') modules. The modules are fabricated in a factory, transported to site and assembled in less time than traditional construction takes. The modular nature of the design, discrete components combined in sub-assemblies, and the prevalence of reversible connections (usually bolts), means these systems are deconstructable; Foremans Relocatable Building Systems is a company which successfully relocates and reuses modular buildings in the UK (BuildOffsite, 2006).

The contractor on the working group estimated that the DfD construction time would be approximately two-thirds that of traditional construction through the use of prefabrication (which allows components to be manufactured in parallel and more quickly installed) and the removal of in-situ concrete (and associated curing time) from the programme. This saving is shown indicatively in a Gantt chart in figure 5.3, which also lists the construction options that enable it. The chart is based on information provided by the working group contractor; while absolute values have been changed, the relative lengths of DfD to traditional construction for each task have been preserved.

The 33% saving estimated by the working group is in excess of the 20% time saving estimated by one interviewee, although another reported a project that was made watertight in 50% less time, with reduced weather delays subsequently giving further programme savings. The manufacturers interviewed stated that design time is condensed when standardised modules are used, avoiding duplication of effort. As noted by two respondents, a programme saving is worth most when on the project 'critical path' and thus decreases time on site and allows income to be generated from the completed building sooner.

Risk reduction

Almost all interviewees emphasised the reduction in health & safety risks when performing work in a controlled factory environment as compared with a site, an

5.4 Results: benefits and barriers to deconstructable designs

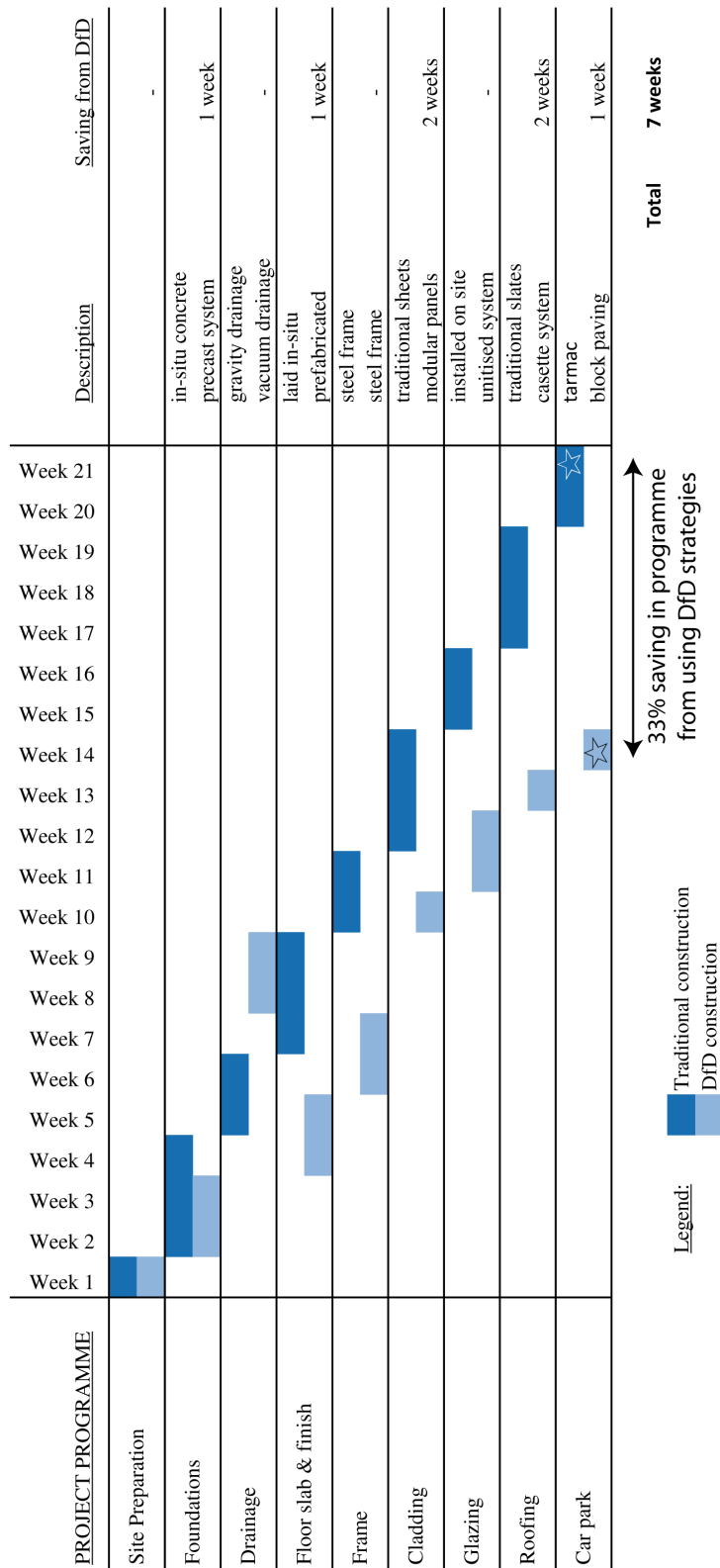


Figure 5.3: Gantt chart showing indicative programme savings use of DfD strategies

advantage also acknowledged by the working team. Five interviewees also noted that prefabricating products in factories reduced programme risks due to weather, waiting for equipment/materials/personnel to arrive, and overruns from one trade impacting another, all of which would give financial gain, though the working group observed that proper management on site can reduce these risks also. The working group client postulated that future governments might legislate to enforce reuse or deconstruction; building deconstructable supermarkets would reduce the impact of this risk.

Risk on a construction project derives “primarily from ground conditions and delays” according to two interviewees. Ground conditions are unique to every site, meaning time and cost contingencies must be set aside for adverse outcomes of ground investigations. All interviewees confirmed that the vast majority of foundations are in-situ concrete ground slabs, redesigned for each site. These foundations are difficult to recover and almost impossible to reuse. Additionally, they displace after installation (called ‘settlement’), and can do so unevenly, causing tolerance problems for the superstructure. Many interviewees recommended steel ‘screw piles’ and steel footings as deconstructable foundation systems, however only one interviewee had experience of these. In contrast to the interview responses, the consensus of the working group was that risk of unforeseen ground conditions was not a major concern.

Other benefits to design for deconstruction

The working group client indicated that building a ‘sustainable’ supermarket could enhance their public standing, so accruing them value. Similarly the client opined that “leading the construction industry” in this area would attract positive publicity. The working group’s final output assigned less importance to these benefits than the three described above however.

5.4.2 Remaining barriers to deconstruction

As found in section 2.3, published literature concludes that the main barrier to deconstructable designs is their extra initial cost compared with traditional construction. The working group results support this conclusion but find that extra risk is another important factor. There is potential to overcome both of these barriers.

The cost consultant of the working group produced an estimate comparing the concept design with that of a traditionally-constructed supermarket with the same

5.4 Results: benefits and barriers to deconstructable designs

specification. The resulting costs have been made relative (for confidentiality purposes) to arrive at the values given in table 5.5, which show a cost premium of 16% for the deconstructable option — within the range predicted by the interviewees (5–20%). The costing includes the financial savings of the reduced programme: savings on site facilities and other preliminaries, and increased revenues due to earlier store opening — the effect of the latter is twice as large as the former for this project.

Item	% extra cost ^a
Foundation	16%
Floor slab & flooring	43%
Drainage	175%
Car Park	-15%
Steel Frame	0%
Façade	88%
Cladding	14%
Roofing	-8%
Total	16%

Table 5.5: Costs of deconstructable supermarket relative to traditional
Notes:

a. Costs include financial impacts of programme savings

Although the biggest relative cost premium is for drainage, the biggest absolute premiums are for façade and for floor slab & flooring. As can be seen the deconstructable design of the car park and roofing actually result in net cost reduction once their programme savings are included.

When formally presented with the concept store, the client decided not to pursue the project further due to the extra cost. Interestingly, vacuum drainage was selected by the client for implementation on new projects — despite costing almost three times that of traditional, gravity drainage — because it was deemed to offer sufficient value in adaptability.

The client singled out risk in the floor slab and flooring as contributory reasons for not progressing the project. The working group had been concerned that unforeseen problems would arise in the installation and behaviour of the demountable foundation and flooring systems because such products are not used widely. (Although there is more certainty in the performance of screw piles — suggested by interviewees — the cost premium was too large for this product to be commercially viable).

In particular the client was wary of the risk that the floor-planks would crack the floor finish when deflecting, as such planks have not been demonstrated in this way previously.

While cost is currently a barrier to DfD, it could become a powerful co-benefit. One interviewee suggested that a net cost saving of 10% might be achievable on large developments with economies of scale. The working group tried to assess this claim by soliciting bids from suppliers based on a large number of orders. This approach was unsuccessful because suppliers did not tender significantly lower prices for a larger order. The working group estimated that such suppliers currently offer a ‘premium product’ with a small market, thus have high operating margins which they are unwilling to reveal, but would be forced to reduce prices if a larger market encouraged competition (e.g. if a large number of deconstructable buildings were commissioned).

One fabricator interviewee observed that design teams assign a larger risk contingency to designs or construction methods they are not familiar with, so compounding any cost comparison, even though the actual risks may be lower. The working group agreed with this, asserting that contingencies would be larger on this project as many new technologies were being trialled. Further knowledge and experience of deconstructable technologies will reduce both risk perception and risk contingencies for these products.

5.5 Implications of results

The barriers to DfD indicate opportunities for future research (discussed in section 5.5.1), while the identified co-benefits suggest certain client sectors that are more amenable to DfD than others, outlined in section 5.5.2. The results have two major implications for the construction industry (explored in section 5.5.3) and also suggest a link between DfD and long-life strategies (explained in section 5.5.4).

5.5.1 Remaining barriers to DfD and research opportunities

Cost and risk are the main barriers to DfD identified in section 5.4.2. Cost can be overcome by economies of scale as described in that section. While risk contingencies and aversion can be overcome by more experience with, and examples of, DfD, research is needed to develop deconstructable foundation and floor technologies to fundamentally reduce their associated uncertainty and hence risk.

Research is needed to develop a deconstructable, ground-bearing foundation. As identified by one interviewee, traditional construction methods are often not devoid of DfD features — steel frames are usually bolted together and prefabricated cladding/roof panels are not uncommon. However foundations are overwhelmingly poured concrete and columns are usually cast into these, while grouts are frequently used to join elements or seal up gaps; these irreversible connections make deconstruction impossible, or at least commercially unviable. All the deconstructable foundation options examined required floors to span between support points/lines, rather than being ground-bearing. Research should address this gap because a suspended floor system has higher stresses and deflections than a ground-bearing one, thus is inherently more massive and complex. Challenges to a novel foundation system will be accommodating small differences in ground level initially and tolerating ground movements during the life of the structure.

A related challenge, highlighted by the working group client, is achieving finishes of sufficient quality and robustness on prefabricated floor planks. In the case of a supermarket the preferred finish is tiles, which are brittle and therefore crack if relative movement occurs — i.e. between two planks. Precast concrete planks can be grouted together to avoid this but this connection is then irreversible. Three potential research areas to solve this are: (reversible) lateral connectors to ensure planks do not deflect relative to one another; a demountable system between the plank and finish that prevents differential movements from reaching the brittle finish; flexible tiles that meet supermarkets' requirements while tolerating movements without cracking.

5.5.2 Clients to target for DfD

Assuming that research can develop a deconstructable foundation and flooring system to a satisfactory performance, then fully deconstructable buildings could be built with the co-benefits found in section 5.4.1. As described in section 5.4.2 it is possible that simply increased prevalence of deconstructable buildings will surmount barriers of cost and risk by utilising economies of scale and giving more experience to industry professionals. The construction industry is conservative so examples of successful deconstructable designs will show prospective clients that they can be achieved and encourage them to do likewise, so spurring further cost and risk reductions. This would create a ‘virtuous circle’ of DfD as illustrated in figure 5.4.

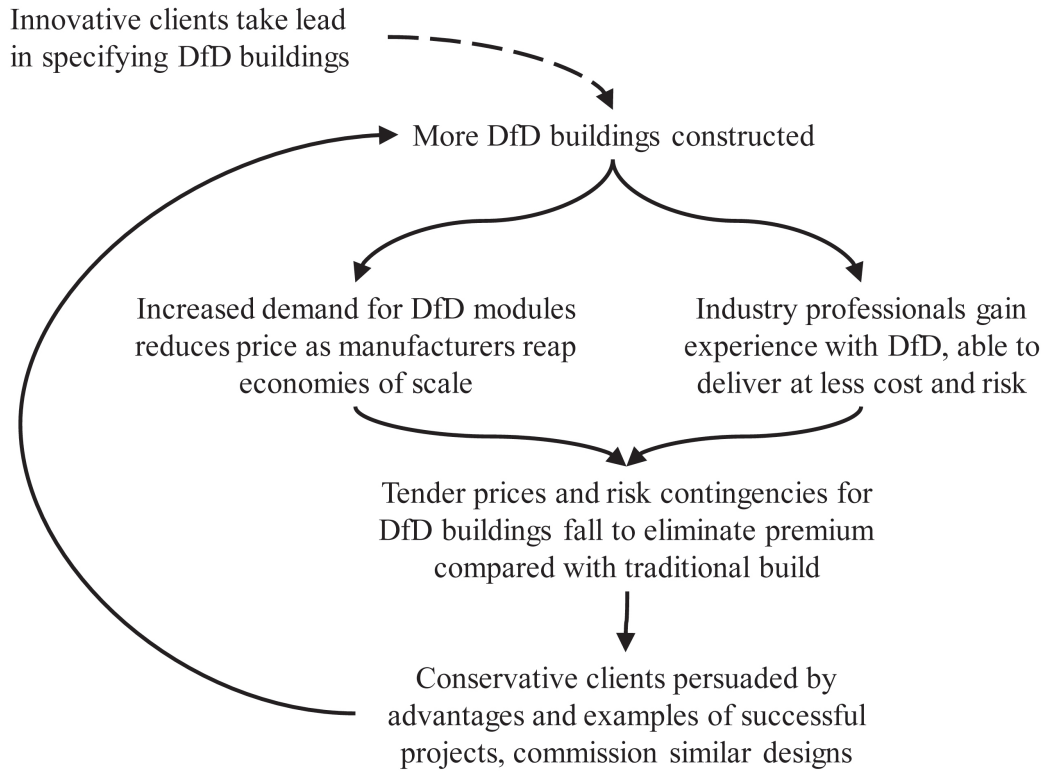


Figure 5.4: Increased construction of DfD buildings leading to the removal of barriers and creation of a ‘virtuous circle’

The challenge therefore is to get a first cohort of clients to design for deconstruction, ones who are enticed by the benefits outlined in section 5.4.1. Such a client will prize adaptability and will value faster construction time. Clients that will own the building throughout the life-cycle or that build large amounts of similar buildings will also reap benefits at deconstruction. Industrial and retail clients (the main users of portal frames) are most likely to fit these criteria, as they may value first-to-market advantages and rapid reconfiguration, and may wish to move sites in

20–40 year cycles. Retail clients often have portfolios of similar buildings and are constantly adding and removing buildings, therefore having the potential to move elements directly from a deconstruction site to a building under construction.

5.5.3 Implications for the construction industry and policy-makers

The results have two implications each for the construction industry and for government policy. Within industry, design team members should make clients aware of the initial benefits designs for deconstruction can have, in particular for industrial or retail clients. Demolition contractors, fabricators and stockists should be aware of the potential for business in the reuse of elements once deconstructed, and develop deconstruction, storage and reclaimed market capabilities. To encourage the formation of a sophisticated reclaimed-materials supply chain, government procurement specifications should require a set fraction of reclaimed materials, and all designs to be deconstructable. Building sustainability rating schemes such as the Building Research Establishment Environmental Assessment Method (more commonly known as BREEAM) should reward deconstruction instead of treating reused and recycled waste streams the same. Legislation penalising/prohibiting demolition or requiring reuse of structure would provide an immediate spur to deconstructable technologies and skills, as well as forcing formation of a marketplace for reclaimed elements.

One remaining barrier to deconstruction could be its future cost or risk, relative to demolition, at end-of-life, rendering any DfD features defunct. As found in section 2.3.4, risk of overrun or damage is currently a deciding factor in the choice of demolition over deconstruction, however this could change in future as deconstruction skills change — therefore efforts should be made by the industry and government to ensure such skills improve rather than deteriorate. Planning authorities could reduce risk of deconstruction overrun by specifying a minimum period between start of demolition/deconstruction works and construction works to negate demolition’s advantage in this case. As Allwood et al. (2010a) find, buildings are often vacant for long periods before demolition, during which deconstruction could take place with reduced risk of impacting the overall project programme.

5.5.4 Link between long-life design and DfD

As noted in section 2.3, most buildings fail because they become ‘unsuitable’. The findings of section 5.2 suggest this can be averted by designing-in ‘adaptability’,

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which is also one of the co-benefits of design for deconstruction reported in section 5.4.1. Therefore the same design strategy can be used to allow the steel in buildings to last longer and for it to be reusable. This can be demonstrated in the case of a supermarket which is designed for deconstruction, thus is adaptable and can be changed more readily to suit the retailer's requirements, thus allowing it to avert 'unsuitable' failure and last for longer. This is not surprising as both 'using for longer' and 'reusing components' strategies aim to extend the life of material, with the distinction that 'using for longer' applies to entire products while 'reusing' applies to components.

Chapter 6

Deconstructable composite connectors

One of few technical barriers to deconstruction, and therefore reuse, identified in section 2.3 is composite construction — joining a steel beam and concrete slab together using a welded stud. This type of construction constitutes about 40% of floor area built each year in the UK, mainly in office buildings (BCSA, 2011) (akin to the typical building established in chapter 3). Composite beams are used because they are cost-efficient (partially by reducing the mass of steel required (Lam and Dai, 2013)) — can the connector be changed to permit reuse whilst still allowing this efficient first use of material?

Recent efforts to develop a demountable connector, as reviewed in section 2.4, have consisted of push tests on machined studs or pretensioned bolts. However beam tests better mimic the loading connectors experience in practice, while non-preloaded bolts are easier to install. Section 6.1 describes three beam tests that were performed using a non-preloaded bolt as a demountable connector, examining its behaviour absolutely and relative to welded studs. The results from these experiments are presented in section 6.2 with their implications discussed in section 6.3. This chapter is based on the journal article “Viability and performance of demountable composite connectors” (Moynihan and Allwood, 2014a).

6.1 Methodology to test a demountable connector design

Three composite beam specimens, of lengths 2 m, 10 m and 5 m, were laboratory tested to investigate the behaviour of steel bolts as demountable composite connectors. The 2 m specimen was used to test the concept. To compare connector performance with that of welded studs, the larger specimens were constructed to the same specifications as Hicks (2007), who undertook tests on 10 m and 5 m specimens with welded studs in the same laboratory in 2005.

UK practice is to use profiled steel decking, so the same commercially available decking (Multideck 60–V2, 0.9 mm thick (Kingspan Structural Products UK, 2013)) as used by Hicks was chosen for all three specimens. The decking was laid on top of the steel beams and connected by 20 mm diameter (M20) grade 8.8 bolts through 24 mm diameter holes predrilled through the decking and top flanges, then fastened by washers and nuts on either side as shown in figure 6.1. The nuts were tightened to 100 Nm torque to ensure the decking was clamped to the beam; a higher value of pre-load would not have permitted the effect of bolt slip on the overall behaviour of the beam to be investigated. (Also preloading bolts would be more laborious in practice.) Following the procedure of Hicks, fewer than the optimal number of connectors were installed to ensure each was fully loaded at failure.

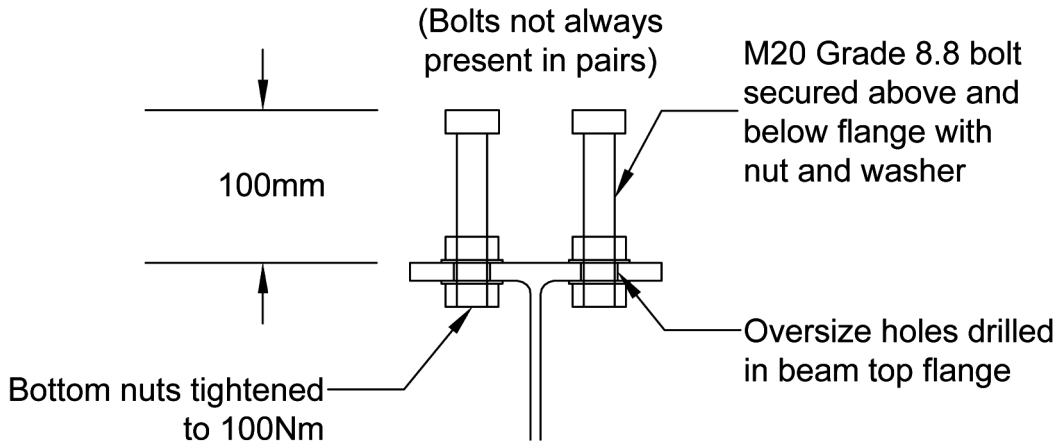


Figure 6.1: Demountable, bolted connector

Data were recorded from displacement and strain gauges along the specimens, and from a loadcell attached to each jack. These were analysed and maximum applied moments compared with predictions calculated using Eurocode 4-1-1 (BSI, 2004), informed by results from concrete cube and steel tensile tests performed to obtain

6.1 Methodology to test a demountable connector design

the materials' properties. For the 2 m specimen an elastic analysis was used to back-calculate the failure moment that would cause crushing strains in the concrete. The results from the larger two specimens were compared with Hicks (2007)'s previously published results.

6.1.1 Laboratory testing of demountable connector design in 2 m specimen

A 2m long specimen was constructed as shown in figures 6.2 and 6.3, with C16/20 concrete poured to form a 140 mm thick slab 0.5 m wide on top of a UB 254x102x28 S355 steel beam. Two demountable connectors were placed in each half-span in the 'favourable' trough position, staggered either side of the beam web.

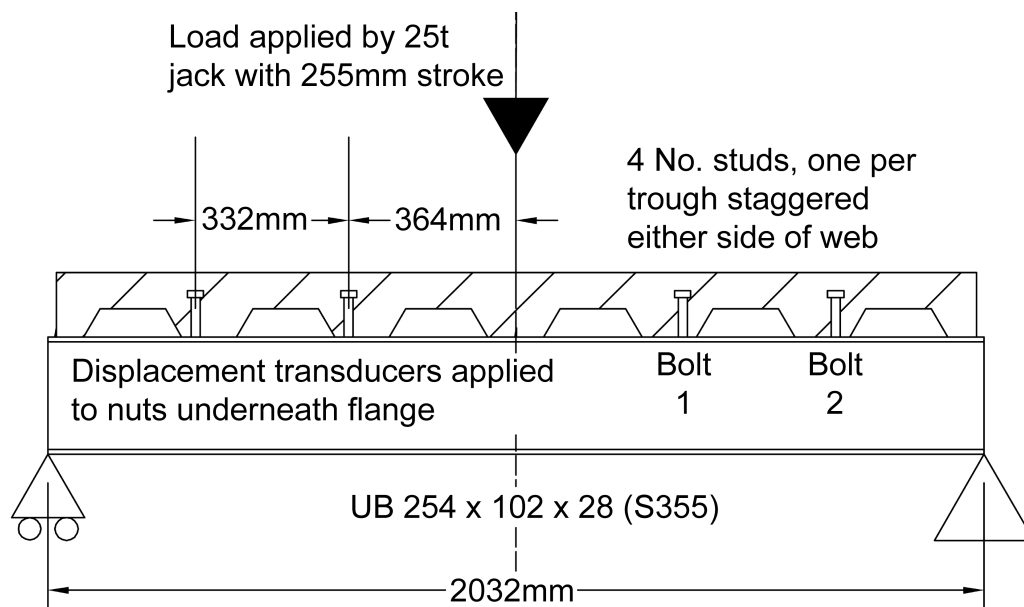


Figure 6.2: Geometry and loading setup for 2m specimen

6. DECONSTRUCTABLE COMPOSITE CONNECTORS

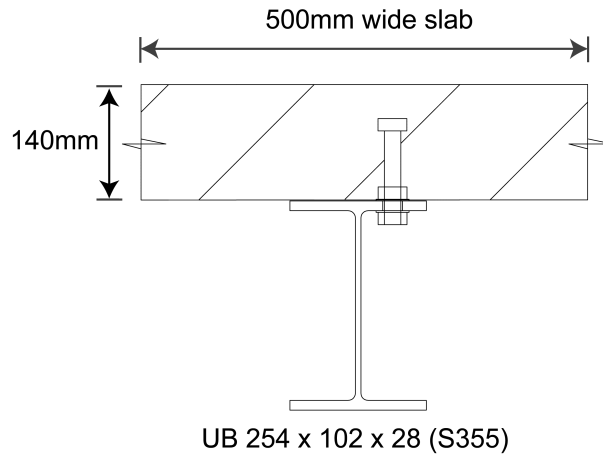


Figure 6.3: Section through 2m composite beam and slab at bolt 1

Displacement gauges were placed against the lower nut of each connector and fixed to the underside of the flange, as shown in figure 6.4, to measure relative slip. Displacement gauges were also placed at the loading point and the beam midpoint and third points to measure deflection. Loading was imposed at a rate of approx. 2mm/minute via a 25t hydraulic jack mounted on a rig to subject the specimen to 3-point bending. The beam was initially loaded to a service moment of 4 kNm, equivalent to a uniform distributed load of 6.5 kN/m² (a typical office loading as specified by Eurocode 1 (BSI, 2002)). It was then unloaded and demounted — the bottom nuts released and the beam lowered clear of the slab — to test that the bolted connector design did facilitate reuse. The beam was then reattached and reloaded in cycles to increasingly higher loads until failure occurred.



Figure 6.4: Displacement gauge attached to the underside of beam top flange measuring relative slip of nut

6.1.2 Laboratory testing of demountable connector design in 10 m specimen

Figures 6.5 and 6.6 show the arrangement of the 10 m specimen, mimicking Hicks (2007)'s setup: 7 pairs of bolts in one half-span and 15 single bolts (staggered either side of the web to ensure a balanced application of force) in the other; 2.5 m wide slab cast from C16/20 concrete, 140 mm thick on the decking. Following Hicks, the beam was propped at the third-points until testing so the full self-weight was applied to the connectors once the props were struck. Displacement gauges were placed at each nut along one side of the beam and at the nuts closest the support and the middle on the other side. Displacement gauges were also attached to the slab midpoint and third points. Strain gauges were affixed longitudinally at the centre of the flanges and at 45° to the vertical at the mid-height of the web at 15 locations indicated in figure 6.5.

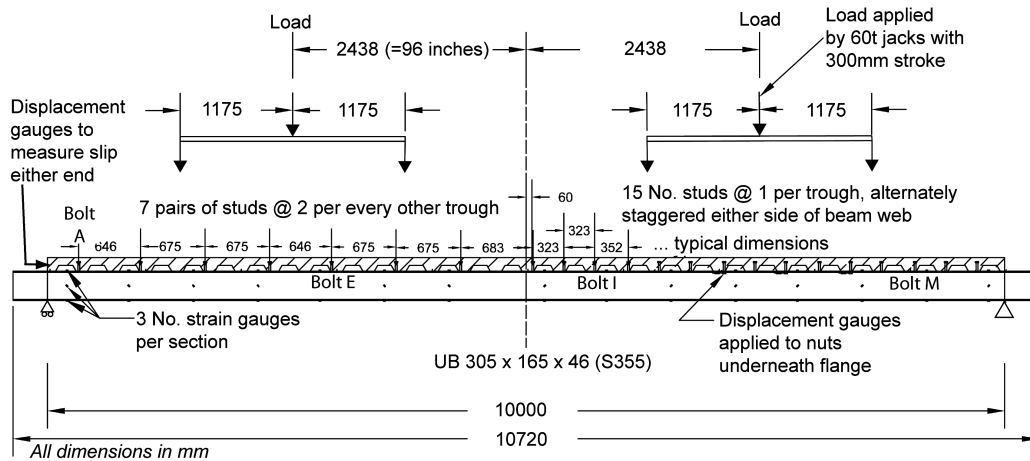


Figure 6.5: Geometry and loading setup for 10m specimen

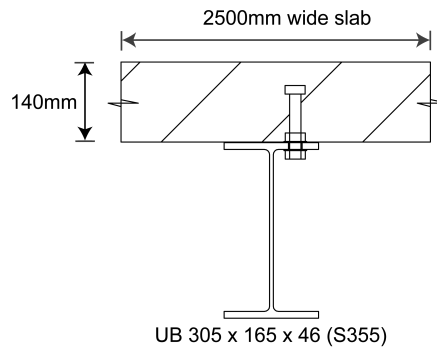


Figure 6.6: Section through 10 m composite beam and slab at bolt I

6. DECONSTRUCTABLE COMPOSITE CONNECTORS

Following the approach of Hicks (2007), the beam was loaded in six-point bending using two hydraulic jacks mounted on rigs, each loading two spreader bars. The rate of imposed displacement was approx. 5 mm/minute (as measured at midspan), continued until an imposed service moment of 81 kNm was reached, equivalent to a uniformly distributed load of 6.5 kN/m² (again chosen as a typical office loading from Eurocode 1 (BSI, 2002)) and then unloaded. After twice repeating this, the bottom nuts were loosened and the slab jacked up approx. 10 mm clear of the beam. The slab was then lowered and beam reattached. The specimen was reloaded to service three times and gauges affixed to either end of the beam to measure relative displacement of the slab. Loading was increased in cycles until failure occurred in one half. To try to force failure in the other half, an end-stop (shown in figure 6.7) was welded at the left-hand end of figure 6.5 to prevent the left half-span from moving further.

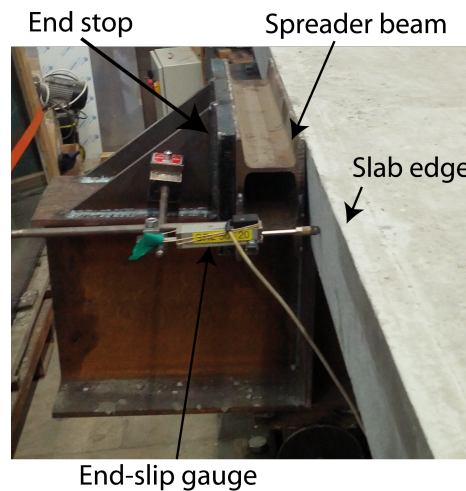


Figure 6.7: End-stop welded on to left-hand end of 10 m specimen

6.1.3 Laboratory testing of demountable connector design in 5 m specimen

After the procedure described in section 6.1.2 was applied, half of the composite beam appeared not to have failed. Following Hicks (2007)'s methodology, and to gather further data, the beam was then cut in half and the unfailed portion tested as shown in figure 6.8. Clearly this 5 m specimen had the same slab geometry and sensors attached as the parent 10 m specimen. However eight bolts were now in the 'unfavourable' location of the trough. A spreader bar was used to load the beam in 4-point bending using a hydraulic jack mounted on a rig, imposing a cyclic displacement until failure, at a rate of approximately 5 mm/minute, as measured at midspan.

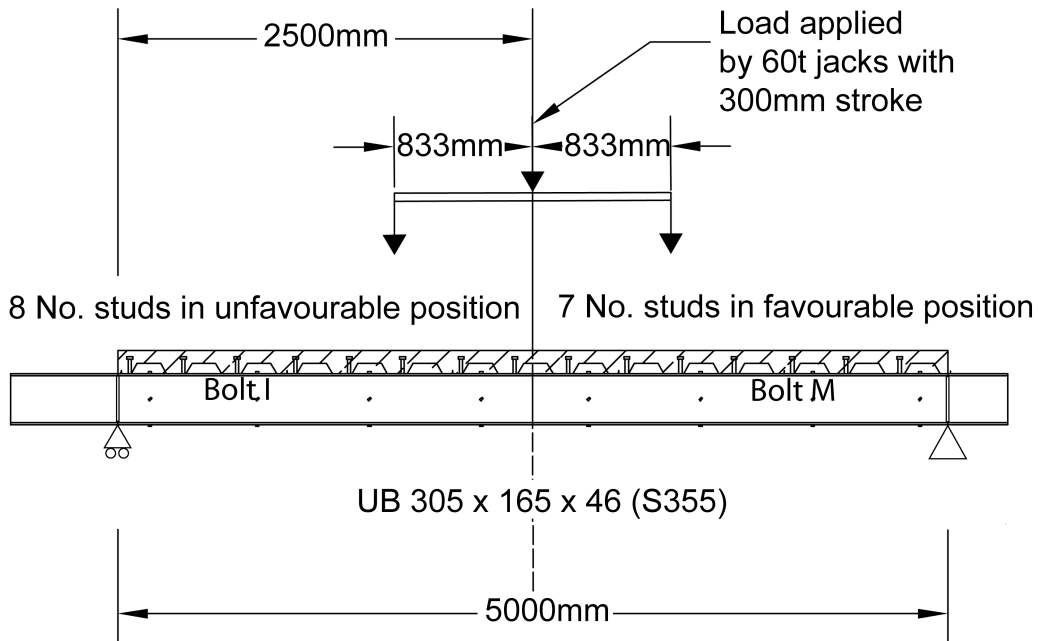


Figure 6.8: Geometry and loading setup for 5m specimen

6.2 Results from demountable connector tests

The experimental results for all three beams are reported in turn below, and compared with the predictions. The results of the larger two beams are compared with the previously published results for welded studs.

6. DECONSTRUCTABLE COMPOSITE CONNECTORS

6.2.1 2 m specimen results

The 2 m specimen was successfully demounted and reassembled: figure 6.9b shows the suspended slab with the beam removed entirely, contrasted with the initial configuration in figure 6.9a.

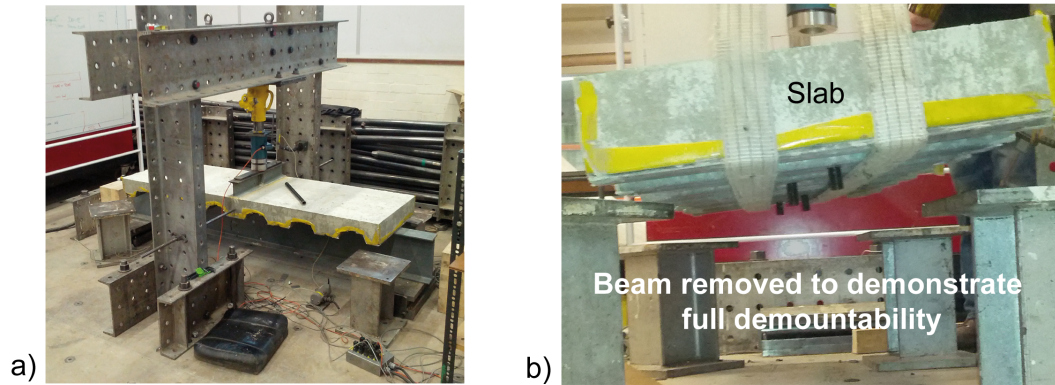


Figure 6.9: a) initial, assembled 2m specimen and loading rig; b) demounted slab after loading to service and unloading

Results from the material tests for the 2m specimen are given in table 6.1. Eurocode 4-1-1 (BSI, 2004) calculations with these values predict failure in the concrete at the connector at a moment of 185 kNm.

UB 254x102x28 (S355) steel beam	
Mean flange yield strength	420 MPa
Mean web yield strength	480 MPa
Beam dimensions assumed same as from standard UK catalogue (SCI, 2009)	
C16/20 concrete slab	
Age at testing	14 days
Mean compressive cube strength ($f_{cm, cube}$)	21.1 MPa
Characteristic compressive cube strength ($f_{ck, cube}$)	20.6 MPa
Characteristic compressive strength (f_{ck})	16.5 MPa ^a

Table 6.1: Measured material properties for 2 m specimen

Notes:

- a. Calculated from BS EN 1992-1-1 (BSI, 2008); other concrete properties taken from typical values from this source.

Failure actually occurred at a moment of 246 kNm (32% greater than predicted using Eurocode) due to compression, shown by a shear-plane in the slab at midspan as seen in figure 6.10 (a plastic hinge had already started to form in the steel beam).

6.2 Results from demountable connector tests

A failure moment of 248 kNm was calculated from the back-analysis of concrete crushing strains (assuming crushing strain $\varepsilon_c = 0.0035$); this is within 1% of the experimental value. That the concrete crushed indicates fully composite action, as assumed in the calculation.



Figure 6.10: Crack indicating shear failure of 2 m specimen

Figure 6.11 shows the moment-displacement profile at the midspan of the 2 m specimen, displaying elastic and plastic regions as expected.

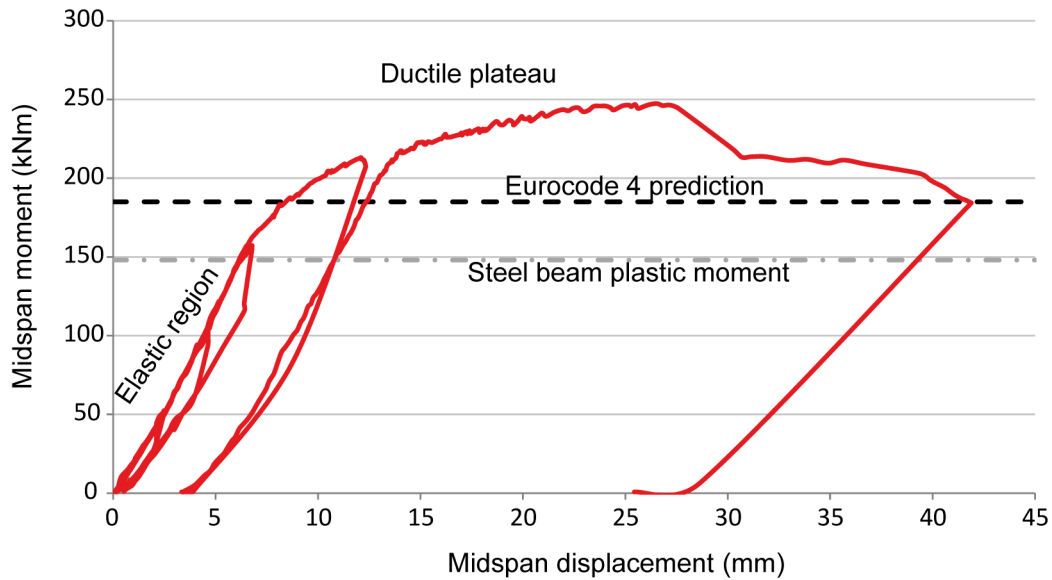


Figure 6.11: Moment vs. displacement for 2 m specimen at midspan

6. DECONSTRUCTABLE COMPOSITE CONNECTORS

Bolt slips, as measured at the underside of the flange, are shown in figure 6.12 for the two bolts in the right half-span (locations indicated in figure 6.2). The initial steep gradient to each plot may be explained by the approx. 5 kN of shear force needed to overcome the friction induced by the torque on each bolt. The slip profile in each appears to be tri-linear after this — both changing gradient near 100 kNm and again near 150 kNm. The reasons for this are not understood, although one explanation could be that the first change in gradient is due to the bolts bearing on the side of the hole, and the second due to the bolts themselves yielding.

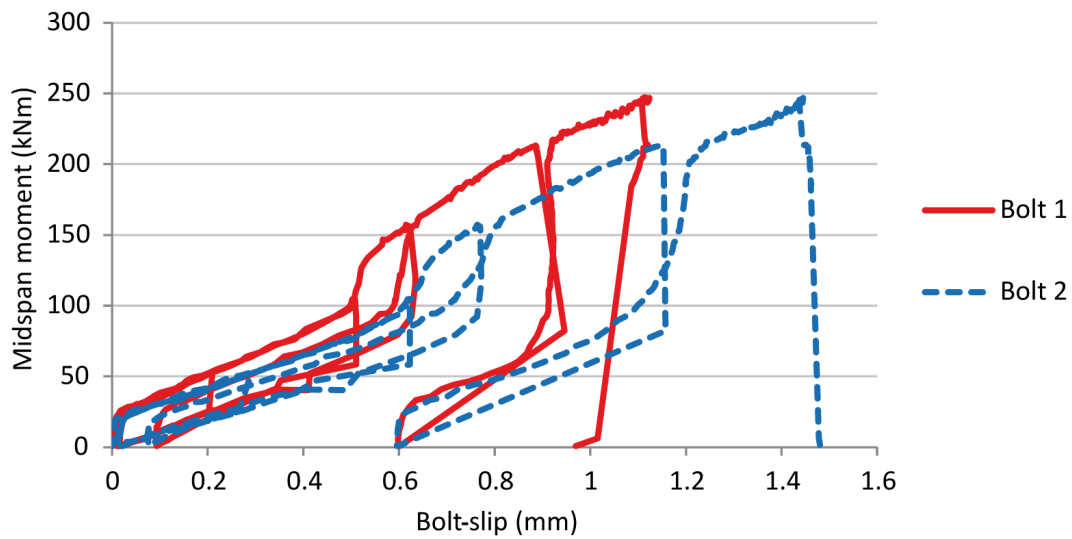


Figure 6.12: Moment vs. slip for two bolts from right half-span of 2 m specimen

6.2.2 10 m specimen results

The 10 m specimen was successfully loaded to service, demounted and reassembled; the latter two processes were achieved more easily and quickly than had been anticipated. Figure 6.13 shows the test specimen in initial and disassembled states.

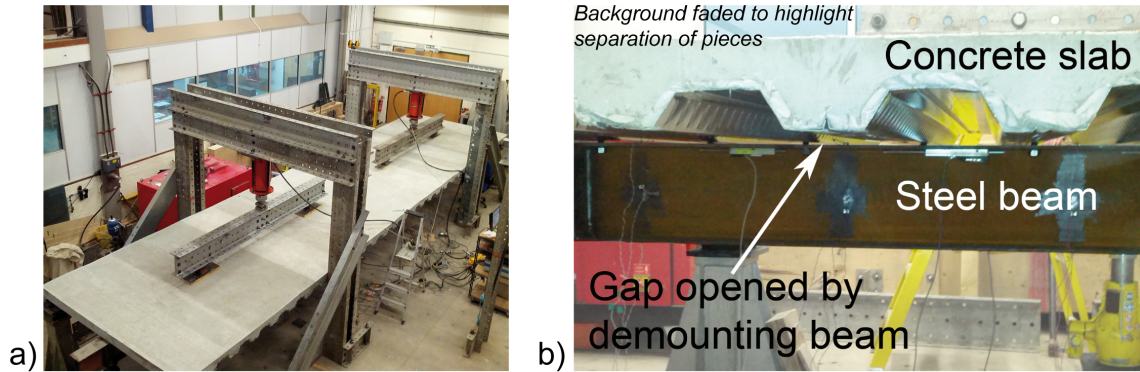


Figure 6.13: a) Initial, assembled 10 m specimen and loading rig; b) Demounted beam after loading to service and unloading

The reassembled beam was then loaded until the decking had delaminated from the slab in the left half-span at a midspan deflection of 280 mm. This was confirmed as pull-out failure in a cone shape around the bolts, shown in figure 6.14, once the decking was removed. After testing was complete longitudinal cracks were noticed along the centreline of the slab, further indicating concrete failure initiated at the bolt locations.



Figure 6.14: Cone failure surface indicative of pull-out failure in left half-span of 10m specimen

6. DECONSTRUCTABLE COMPOSITE CONNECTORS

The results of the cube and coupon tests for the specimen are given in table 6.2. Eurocode 4-1-1 (BSI, 2004) calculations with these values predict failure of the concrete at the bolt pairs at a moment of 357 kNm, 5% less than the maximum moment (including self-weight) recorded experimentally: 378 kNm (achieved before delamination).

UB 305x165x46 (S355) steel beam	
Mean flange yield strength	376 MPa
Mean web yield strength	395 MPa
Depth of section	303 mm
Width of flange	167 mm
Flange thickness	10.9 mm
Web thickness	6.6 mm
C16/20 concrete slab	
Age at testing	18 days
Mean compressive cube strength ($f_{cm, \text{cube}}$)	13.8 MPa
Characteristic compressive cube strength ($f_{ck, \text{cube}}$)	13.3 MPa
Characteristic compressive strength (f_{ck})	10.7 MPa ^a
Secant modulus of elasticity (E_{cm})	24.6 GPa ^b

Table 6.2: Beam and slab properties for 10m specimen

Notes:

- a. Calculated from BS EN 1992-1-1 Section 3
- b. Derived from beam bending stiffness

After the end-stop was welded, failure was predicted (using the values in table 6.2) in the concrete around the single bolts at a moment of 375 kNm. Midspan deflection was increased to 490 mm, causing a moment of 434 kNm (14% higher than the predicted maximum) but without causing failure in the right half-span — at this point it was noticed that the end-stop itself had failed and the experiment halted. The final deformation of the beam is shown in figure 6.15.



Figure 6.15: Final state of 10 m specimen, showing plastic deformation

Figure 6.16 shows the moment-displacement graph at the midspan of the specimen, with self-weight moment and predicted failure moments from Eurocode 4-1-1 (BSI, 2004) indicated. Also plotted are results from Hicks (2007), whose displacement values were measured relative to the propped mid-span height and therefore have been uniformly reduced to facilitate comparison (Hicks' predictions are not shown).

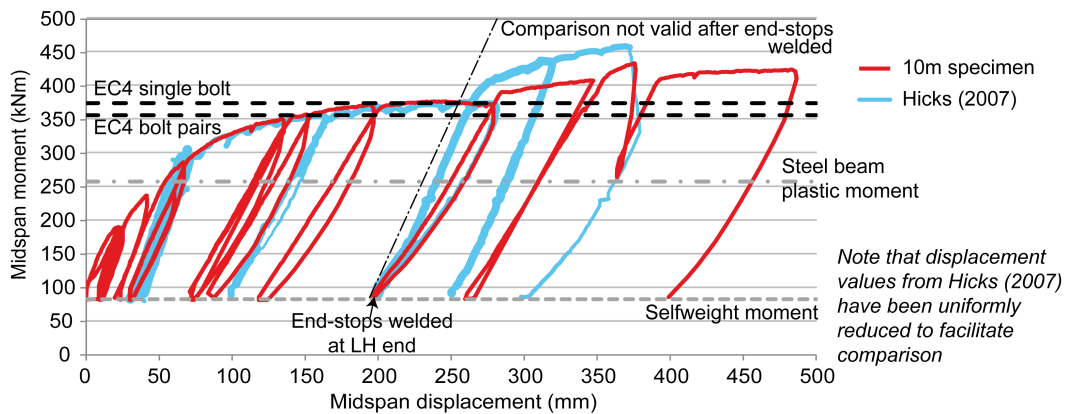


Figure 6.16: Moment vs. displacement for 10m specimen and comparison with published values from Hicks (2007)

Both curves in figure 6.16 exhibit elastic behaviour initially followed by a ductile plateau, caused by formation of a plastic hinge in the beam approximately under the point-load immediately left of midspan, revealed by Lüder's wedges visible in the web and confirmed by strain gauge readings. Both beams fail due to concrete

pull-out in the left half-span (i.e. with stud pairs) at similar moment values. This is surprising as Hicks reports a concrete characteristic strength 14% higher, a stud capacity 14% higher (as calculated using Eurocode 4-1-1) and a beam axial capacity 3% higher, so the expected difference in maximum moment is 8%. Once end-stops are welded results cannot be compared as Hicks' end-stop was designed differently and did not fail prematurely.

Figure 6.17 is an enlargement of the initial portion of figure 6.16 (and omitting Hicks (2007)'s values) to compare moment-deflection curves just for the service loading cycles before and after demounting. As can be seen the curves are almost identical once initial 'bedding in' occurs after remounting.

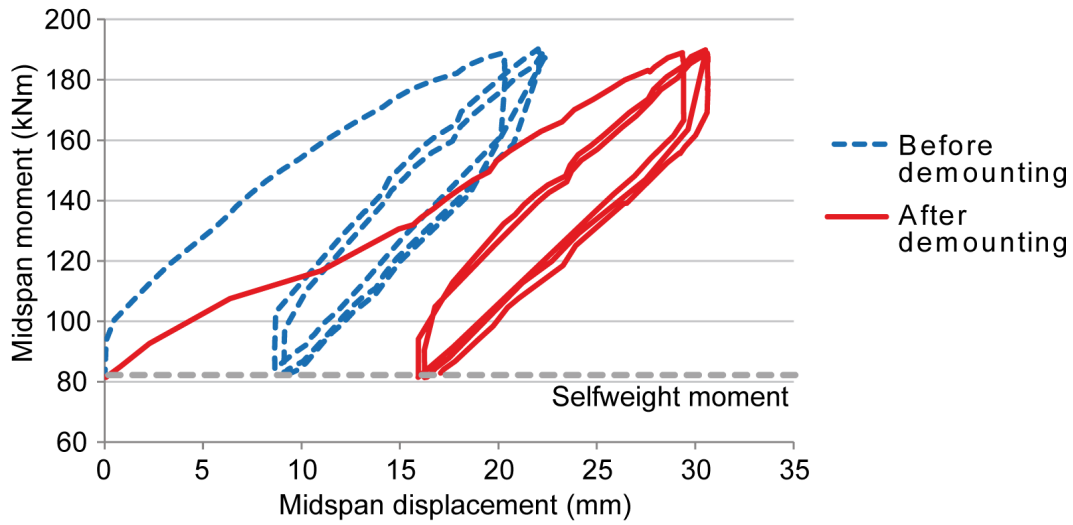


Figure 6.17: Comparison of midspan moment vs. displacement of 10m specimen before and after demounting

Plots of end-slip with moment are given in figure 6.18, omitting the left slip after the end-stop is welded. Ductile behaviour is seen in both sides, but magnitudes are greater on the left side: maximum left slip is 19.8 mm, while Hicks (2007) reports a corresponding value of 26.5 mm.

6.2 Results from demountable connector tests

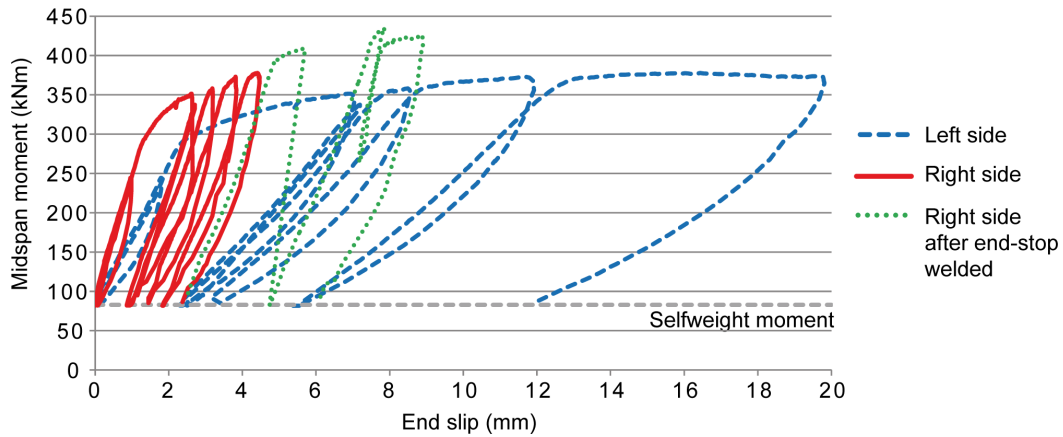


Figure 6.18: Midspan moment vs. end-slips for 10m specimen

Note: left side slip not shown after end-stop welded

Figure 6.19 displays a plot of midspan moment against the slip of four bolts (as measured at the lower nut) taken from different locations along the beam (labelled on figure 6.5). The shapes of the slip curves are similar to those for the 2 m beam in figure 6.12 but continued further as more deflection and slip occurred. The 10 m slips show the same initial stiffness while the torque is overcome but only bolt E shows the same ‘tri-linear’ behaviour seen in the 2 m slip plot (figure 6.12).

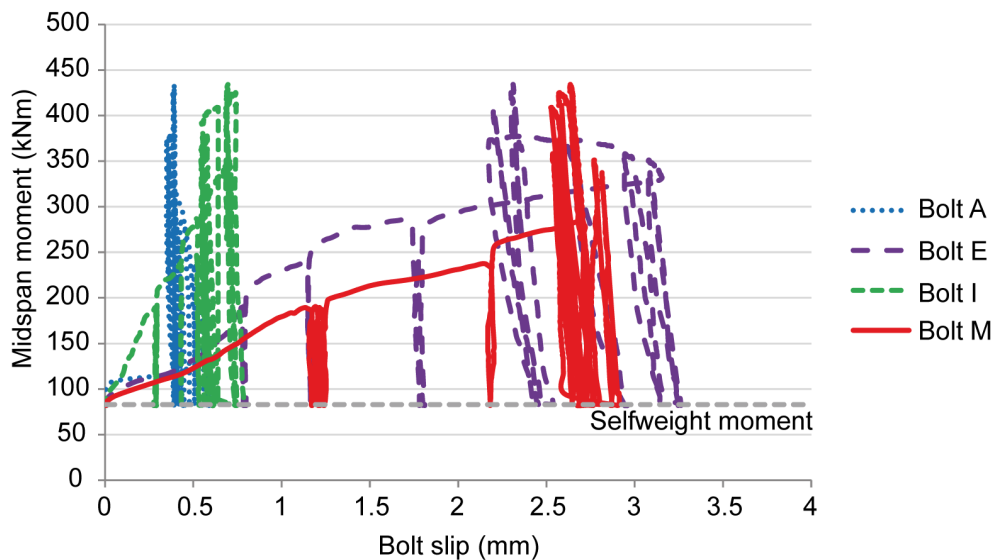


Figure 6.19: Moment vs. slip for four bolts from different locations along 10 m specimen

The different maximum slip values for each bolt can be explained as follows: each bolt has nominally 4 mm of clearance in the oversized holes, thus potentially 4 mm

6. DECONSTRUCTABLE COMPOSITE CONNECTORS

of slip can occur before the bolt must bear on the side of the hole (i.e the beam flange). Assuming that the bolts are initially randomly positioned in the holes, it is then not surprising that some bolts (e.g. A and I) slip less than 1 mm whereas others slip almost 3.5 mm (e.g. bolt E) — none however slip more than 4 mm. Once the bolt bears directly on the flange little further slip occurs as would be expected; some reverse slip of the nut is seen, for example bolt E, potentially caused by rotation of the bolt as the slab continues to move away from the centre. Concrete pull-out prevents the left half-span bolt slips being correlated with the left end-slip, however right end-slip was 4.5 mm before the end-stop was welded (i.e. while this half-span was unfailed), which is a similar magnitude of slip to the right half-span bolts (e.g. bolt I).

Strain profiles at midspan are plotted in figure 6.20 for different values of moment. As expected the neutral axis position falls as the slab slips under increasing load. The maximum net axial force in the beam is 771 kN, or 55.1 kN per bolt, 54% greater than the 35.8 kN capacity predicted by Eurocode 4-1-1 (BSI, 2004).

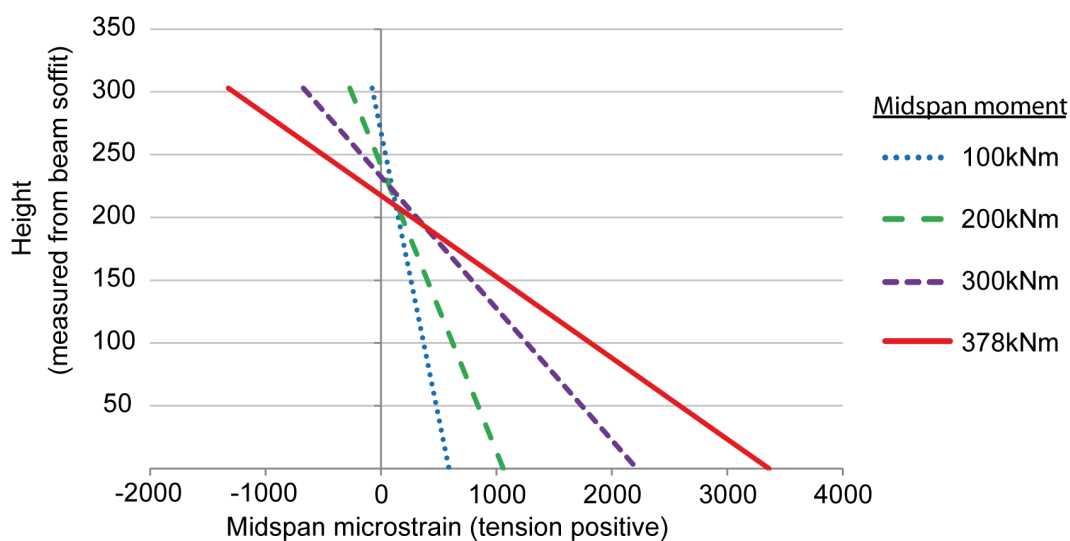


Figure 6.20: Midspan strain profiles in 10m specimen for different moment values

6.2.3 5 m specimen results

The 5 m specimen (shown in figure 6.21a) was loaded in cycles until deck delamination occurred (shown in figure 6.21b) in the right half-span of the beam — where bolts were in the ‘favourable’ position (as indicated in figure 6.8) — at a maximum moment of 376 kNm (achieved before delamination) and midspan deflection of 145 mm.

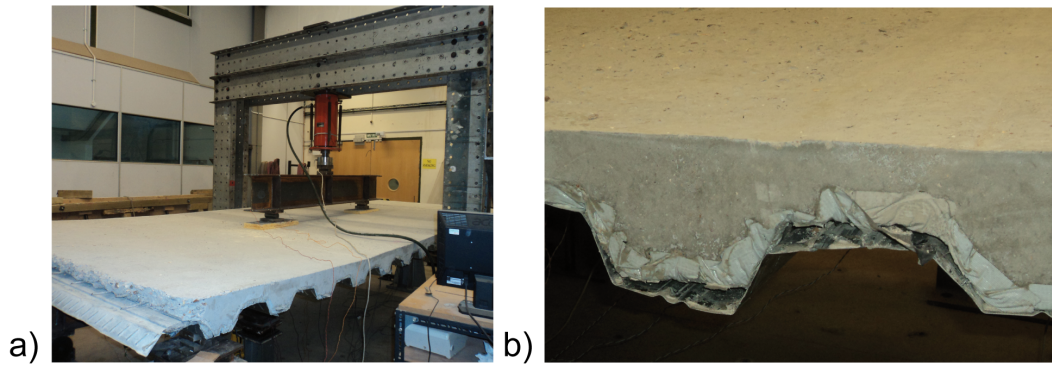


Figure 6.21: 5m specimen a) initially, with loading rig; b) showing decking delaminating from slab

Concrete cube and cylinder tests undertaken the same day the specimen failed (68 days after casting) resulted in a characteristic cylinder compression strength of 11.1 MPa. Eurocode 4-1-1 (BSI, 2004) calculations performed using this value and other properties taken from table 6.1 predicted failure of the concrete at the studs at a moment of 328 kNm; 13% lower than that found experimentally. Inspection of the slab once the decking had been removed confirmed concrete pull-out failure around the bolts. Figure 6.22 shows a moment-displacement graph for the specimen at midspan, with self-weight moment and predicted failure moment indicated. Also plotted are the results of Hicks (2007)’s 5m test (though not his predictions).

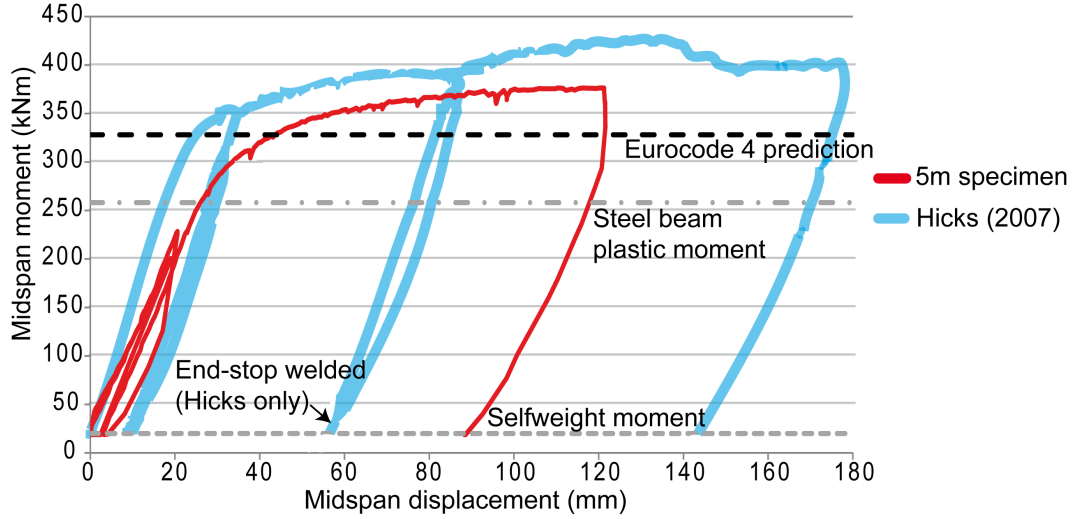


Figure 6.22: Moment vs. displacement for 5m specimen and comparison with published values from Hicks (2007)

Both curves show elastic then ductile behaviour, and both witnessed plastic hinges forming in the beam near the left load point. Unlike this experiment, Hicks observed failure in the half-span with ‘unfavourable’ stud locations first, then welded an end-stop and failed the other half-span. Hicks reported a concrete strength 10% higher, which Eurocode 4-1-1 (BSI, 2004) calculations suggest should give a maximum moment 7% higher, however the actual value is approximately 12% higher.

Figure 6.23 shows the variation of end-slip with midspan moment, displaying ductile behaviour after initial elasticity. Maximum end-slips of 13.3 mm (left side) and 12.0 mm (right side) were recorded — similar to the 12.9 mm of slip Hicks (2007) reports for first failure.

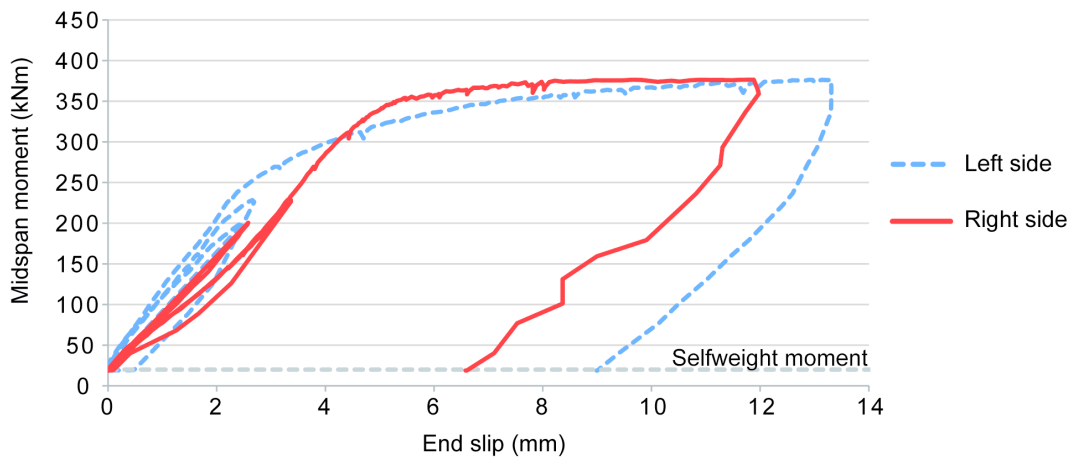


Figure 6.23: Midspan moment vs. end-slips for 5m specimen

6.2 Results from demountable connector tests

The slips of bolts I and M (labelled on figure 6.8) are shown in figure 6.24; the slip of these two bolts are also shown on figure 6.19 for the 10 m specimen. The direction of slip is consistent between the two slip figures, so it can be seen that Bolt I slips 3 mm in the opposite direction as it did during the 10 m experiment — expected as the bolts are now loaded in the opposite sense. Also Bolt I slipped less than 1 mm in the 10 m experiment but now slips 3 mm, consistent with the 4 mm of clearance in the hole. Bolt M is loaded in the same sense as the 10 m experiment and appears to slip backwards a little, probably because the bolt is now rotating in the hole.

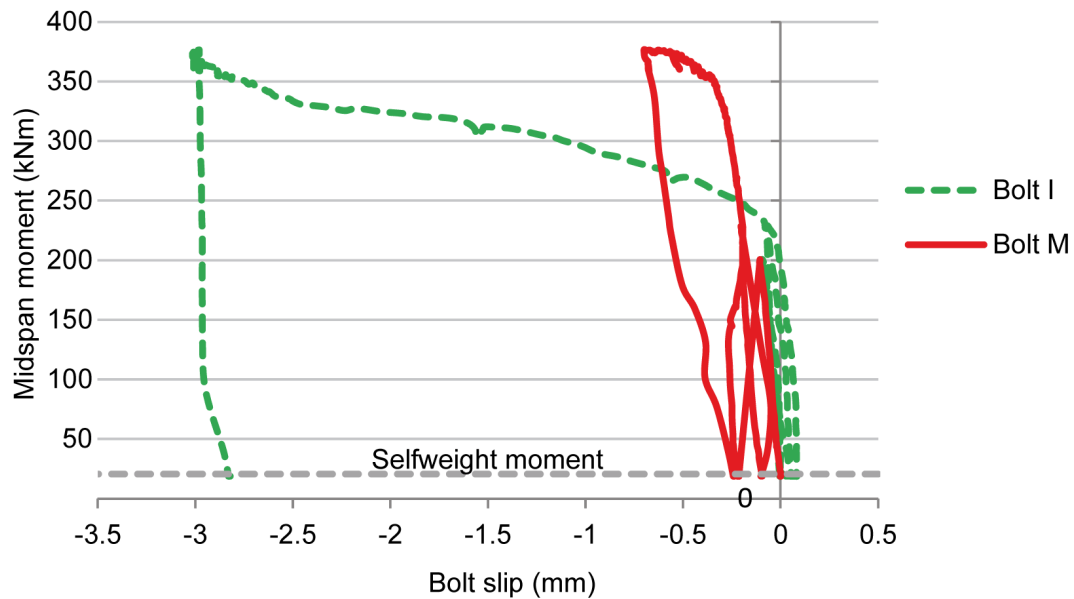


Figure 6.24: Moment vs. slip for two bolts from different locations along 5 m specimen

Readings from the strain gauges on the 5 m specimen suggested that many no longer gave consistent output. This was attributed to damage to the gauges caused by overstraining from the large imposed deformation on the 10 m specimen. Therefore the strain gauge data were not analysed for the 5 m specimen.

6.3 Discussion of results

The experimental results demonstrate that a composite beam with bolted connectors performs similarly to such beams with welded studs in the tested scenarios — but with the added benefit of permitting deconstruction. However discrepancies exist between the results and those found by Hicks (2007), reasons for these are explored in section 6.3.1. The results also show that bolted connectors give greater strength than predicted by the design standards, indicating they can be safely used. Further research described in section 6.3.2 would optimise both bolt design and design guidance, permitting more material- and cost-efficient solutions. Two challenges of using bolted connectors on commercial projects are identified in section 6.3.3, but two potential solutions are also proposed. Policy recommendations are made in section 6.3.4 to encourage adaptation of demountable and reusable systems in construction.

6.3.1 Comparison of results with predictions and with welded specimens

Table 6.3 summarises the salient results from sections 6.2.1, 6.2.2 and 6.2.3. As can be seen, the maximum moment resistances are all above the values predicted by Eurocode 4-1-1 (BSI, 2004). This is expected because design standards such as Eurocode deliberately predict conservatively to allow for uncertainties. The low level of shear connection (20%) may explain the significant under-prediction for the 2 m specimen, as this is below the minimum level strictly required to use Eurocode equations. That all specimens saw failure in the concrete indicates full composite action was achieved as expected.

	Max moment	EC4 ^a prediction	Hicks' max ^b moment	Concrete f_{ck}	Hicks' f_{ck}
2m specimen	246 kNm	185 kNm	-	16.5 MPa	-
10m specimen ^c	378 kNm	357 kNm	385 kNm	10.7 MPa	12.4 MPa
5m specimen	376 kNm	328 kNm	420 kNm	11.1 MPa	12.4 MPa

Table 6.3: Summary of demountable connector experiment results, predictions and comparisons from sections 6.2.1, 6.2.2 and 6.2.3

Note:

- a. Calculations done to Eurocode 4-1-1 (BSI, 2004)
- b. Hicks' experimental results read off graphs provided in Hicks (2007)
- c. Values for pairs of bolts used as comparison with Hicks not valid for single bolts

Table 6.3 shows that the moment capacities of the 10 m and 5 m specimens are 2% and 12% lower than those from Hicks (2007)’s specimens using welded studs. Different material properties and holes drilled in the flange are potential causes of these discrepancies. Despite using an identical mix from the same commercial supplier, a lower concrete strength than Hicks was recorded for both specimens, which causes expected failure moments to be 8% and 7% lower respectively. The holes drilled in the top flange of the beam reduced the plastic moment capacity by 2–3%. Accounting for these two effects, the 5 m specimen’s moment capacity is still 3% lower than Hicks’ value; however the 10 m specimen’s capacity is 8% higher.

If material properties and holes are not the causes of the discrepancies, what are? The divergence for the 5 m specimens can potentially be explained by the larger strains imposed during the 10 m testing — Hicks (2007)’s 5 m specimen saw 100 mm less midspan deflection when still part of the 10 m specimen. These larger strains probably invalidated the ‘unfailed’ assumption about the 5 m specimen, as indicated by the strain gauge failures and the different failure sequence than that reported by Hicks (2007). The latter occurred because the ‘favourable’ half-span of the 5 m specimen had been more highly stressed (probably causing some failure at the shear connectors) under the large shears in the 10 m experiment, while the ‘unfavourable’ half-span experienced lower shear, being closer to the middle of the span. The 5 m specimen’s ultimate moment capacity remains above predicted values (and almost 50% greater than the plastic moment capacity of the steel beam alone) despite the initial damage, indicating that sufficient shear connection remained to enable composite action.

Although both 10 m specimens failed in similar ways, the results from section 6.2.2 exceed predictions whereas Hicks (2007)’s moment capacity was lower than expected. It is not clear why this divergence occurred although Hicks attributes the low result to uplift of the slab between troughs which was not witnessed in the bolted connector experiments — it is possible the use of nuts and washers more effectively clamped the decking to the beam flange, preventing this phenomenon.

6.3.2 Avenues for further research

Knowledge about demountable connectors could be increased in four ways: creating an analytical model of internal interaction, performing further beam tests, producing tailored design guidance, and undertaking push tests.

- The finding that bolts slip different amounts before bearing on the beam has implications for the forces in the beam and how these change as bolts slip. An **analytical model** could be developed to predict these internal forces to compare with experimental results — Lee and Bradford (2013)’s work could potentially be extended to include this. Such a model might also explain the reasons for, and impacts of, the tri-linear behaviour in the bolt-slip curves. Bolt slippage may also have an impact on beam stiffness — although results in sections 6.2.2 and 6.2.3 indicate stiffness similar to Hicks (2007)’s specimens.
- As only three tests were undertaken **further beam tests** are required to investigate the performance of demountable connectors in other scenarios, for example where the beam neutral axis is at the steel-concrete interface or above the connectors, exposing the bolts to higher shear or tension than already investigated. Together with an analytical model, a more extensive testing programme would provide the research support necessary to include demountable connectors in design guidance.
- To give confidence to designers when considering bolted connectors, **tailored design guidance** is required to provide formulae and empirical values suited for demountable connectors because formulae and empirical factors in current guidance, e.g. Eurocode 4-1-1 (BSI, 2004), assume welded studs. Further laboratory testing will be required to calibrate these.
- Eurocode 4-1-1 (BSI, 2004) mandates **push-tests** to verify that ductility requirements are met. These should be undertaken for any bolts used, noting their limitations as discussed in section 2.4. However push tests by Lam and Saveri (2012) and Lee and Bradford (2013) indicate that demountable connectors perform better than welded studs in such tests anyway.

Performance of demountable connectors could be improved by research in two areas: optimising connector material and geometry, and reducing hole size.

- Research is needed to inform the **optimal material properties and geometry** for connectors, accounting for ductility as well as strength, and considering that standard practice uses higher-strength concrete. Grade 8.8 bolts (with a nominal ultimate strength of 800 MPa) were used, unlike those used by Hicks (2007)’s whose studs had an ultimate strength of 513 MPa. Size M20 was chosen as geometrically similar to 19 mm diameter welded studs that Hicks used.

- 24 mm holes were chosen to facilitate demounting but it is possible that demounting could occur with **standard 22 mm holes**. Commercially-designed composite beams would typically have higher shear connection resulting in the neutral axis being closer to the flange and reducing the loss in moment capacity, in which case the benefit of having smaller holes may be negligible.

6.3.3 Implementation of demountable composite beams in industry

The experiments demonstrated that the proposed bolted connector design allows demounting, and therefore reuse, and that the moment capacities can be reliably estimated by Eurocode 4-1-1 (BSI, 2004) and are similar to results from beams with welded studs. Thus the proposed, demountable connector system could potentially be used in practice. However, in practice there may be a cost premium when implementing bolted connectors on site: the unit cost of grade 8.8 bolts is estimated at three times that of similarly-sized welded studs; additional labour is required to install bolts as one person must be (at height) holding the nut underneath the decking whilst another is tightening it from above, while drilling holes in the flange would add further labour. Solutions are suggested to negate these extra costs. Two advantages of this system may justify any cost premium.

Further research can address the extra unit cost — Lam and Saveri (2012) machined a traditional stud into a demountable version, so it is likely that a demountable, cost-efficient (when mass produced) solution can be found. Increased use of prefabrication and ‘smart’ construction technology can address the extra labour requirement: the concrete slab could be manufactured off-site with the bolts cast in required locations protruding from the soffit, and then transported to site (a leading UK construction firm already prefabricates concrete units for use on site, giving a programme and cost saving). The steel beam can be predrilled with holes for the bolts as part of the automated fabrication process to ensure a good fit (provided sufficient manufacturing accuracy can be achieved), requiring only one person to tighten the nuts from below — assuming this task can be performed as quickly as stud welding then this would yield a labour-neutral solution. Optimising the bolt design for installation would aid this process, and may reduce the cost of alternative installation methods.

A demountable system would have two advantages over traditional connectors: no welding and increased flexibility. Welding studs alters their material properties, whereas bolts’ material properties are unaffected by installation. Welds are susceptible to fatigue under cyclic or seismic loading, so bolts may be preferred in

these circumstances — Kwon et al. (2010)’s findings support this hypothesis. Site welding also involves extra health & safety risks that are avoided when bolting; additionally special equipment (usually bespoke to welding studs) is no longer required. Using demountable connectors could allow extra flexibility in the finished building as the steel beam can be replaced if the concrete were propped. This would allow a stronger/stiffer beam to be added if extra capacity/damping were required. The results of chapter 5 found that certain clients value adaptability, so it may be possible that landlords with shorter-term tenants would be willing to pay a premium to be able to adapt composite floors, for example if it facilitates faster installation/removal of stairways during fit-out between tenants. Contractors could generate additional revenue from this longer-term partnership with such clients, as the firm that installed the elements would be best placed to remove or alter them; over time this could more-than-compensate for the cost of adopting the technology. That the beam specimens demounted easily suggests that the concept could work in commercial buildings. The nuts may become difficult to remove after 20 years in place or may damage their bond with the concrete in doing so — so further research is required to understand changes in bolt condition over time.

6.3.4 Further challenges and policy recommendations

While the technology now exists to demount and reuse steel beams, hence reducing carbon dioxide emissions associated with new material production, there is as yet no demand for this option. Policy makers should consider measures to incentivise reuse of construction materials, potentially through schemes that increase the value of materials at the end of structure life (e.g. refunding a deposit if materials are reused) or that provide tax benefits for firms that commission demountable structures. Following the suggestions of section 5.5.3, legislation could penalise demolition or require reuse, thus encouraging use of demountable technologies. The use of demountable connectors to allow steel re-use points also to the potential to reuse concrete slabs, giving further emissions savings. However there are additional challenges in handling and verifying such re-used slabs, and further examination of this opportunity is required.

Chapter 7

Implications and future research

This research has identified the largest uses of steel by the construction industry and discovered significant potential for adoption of the three material efficiency strategies most applicable to construction. This chapter summarises the key contributions from each chapter and their implications, and makes suggestions for future work in each of the research areas.

7.1 Contributions to knowledge

The mapping of steel flows into construction in chapter 3 revealed that industrial buildings and utility networks are the two largest end-users of steel in construction, that prominent structures such as bridges and stadia are small end-uses of steel, and that rebar is the most common construction product, followed by sheet. It identified a multi-storey braced-frame office building as ‘typical’, with most steel in its superstructure but also found that non-structural applications are a non-negligible proportion and should be included in life-cycle analyses.

An analysis of 23 commercially-designed steel-framed buildings in chapter 4 uncovered the potential to use almost 50% less steel in such buildings with no loss in performance. Rationalisation is identified as the probable cause for this over-provision of material.

Chapter 5 presented a set of design strategies for long-life tailored to a framework of product failure causes for the first time. The case-study and interviews also in chapter 5 discovered that deconstructable designs give commercial advantages of adaptability, programme savings and risk reduction, and identified methods to overcome cost and risk barriers.

The demountable connector proposed in chapter 6 would allow composite floors — previously un-reuseable — to be reused while giving predictable performance similar to that of traditional, welded connectors.

7.2 Wider implications of this research

The finding that material efficiency can significantly reduce steel demand in construction implies actions for different parts of the industry; these are outlined in section 7.2.1. Potential reductions in steel production and carbon dioxide emissions by using these strategies are estimated in section 7.2.2.

7.2.1 Implications for the construction industry

Implementing material efficiency strategies does not require a large shift in construction practice, but rather small changes by individuals and companies. Suggestions are given below for each set of actors.

Designers start work at the outset of a project and so can introduce material efficiency early, producing designs that inherently use less material, achieve high utilisation ratios, last for longer and are deconstructable. **Contractors** can advise how to make these aspects constructable, incorporating new technologies to successfully build structures with less rationalisation (e.g. repetition), more adaptability and fewer irreversible connections (e.g. grout). Both parties can cite the co-benefits of deconstructable designs as reasons to employ such techniques.

Stockists can address the lack of supply of reused steel by purchasing reclaimed sections from demolition contractors. Designers and contractors could aid this by giving stockists advance notice of the section sizes they will require for a project so the stockist has time to source these types. **Manufacturers** or **fabricators** could contribute to a reused steel supply by trialling a leasing model for beams whereby they retain ownership of elements throughout the structure life, then reclaim and relocate them once no longer required.

Environmentally-minded **clients** can instruct their design teams to use material more efficiently by achieving minimum average utilisations, designing for deconstruction and designing for long life. Success in these aims will require the collaboration of the entire design team. By pioneering these new designs and technologies, clients provide examples which give confidence to more risk-adverse clients.

Building rating schemes (for example BREEAM (UK), LEED (USA) or Green-Star (Australia)) can encourage clients and design teams to implement material efficiency strategies by providing incentives for them. A first step toward this would be to increase the proportion of ‘credits’ for materials to be in line with their environmental impact, as suggested by Kaethner and Charlson (2012), so that project teams have greater incentive to consider materials’ impacts. **Standards committees** can follow this example and mandate minimum standards for material efficiency, akin to minimum standards for building energy performance specified by Part L of the UK Building Regulations (UK Government, 2010).

In their roles as clients, **governments** can directly specify minimum utilisations, a minimum amount of reused components and design for deconstruction in the buildings they commission. **Policy-makers** can address the failure of a reused materials market through direct incentives or penalties (as suggested in section 6.3) and they can remove the financial incentive to demolish buildings rather than refurbish them by applying the same level of VAT (as described in section 2.3). Each study found economic impediments to implementing material efficiency and suggested innovation to overcome them; therefore putting a higher value on environmental objectives might motivate the desired changes on a macro scale. One way of achieving this could be to put a ‘carbon price’ on CO₂ emissions; however Skelton and Allwood (2013) investigate exactly this for steel use and conclude that “[r]eliance on a carbon price alone to deliver material efficiency would [...] be misguided and additional policy interventions to support material efficiency should be considered.”

7.2.2 Assessment of research objectives and implications for carbon dioxide emissions

As described in section 1.5, this research aimed to identify and assess opportunities to use less steel, reuse more steel, and use products for longer in the construction industry, with the ultimate goal of reducing demand for steel production, and hence decreasing associated carbon dioxide emissions. This research has found that there are significant opportunities for material efficiency in construction, with substantial savings potential found for each strategy. By extrapolating from the specific studies in chapters 4, 5 and 6 to the global flows of steel quantified in chapter 3, the potential for reduction in CO₂ emissions from using steel more efficiently in construction can be estimated (assuming 2.03 tCO₂ emissions per tonne of steel produced as given in Hammond and Jones (2011)):

- Assuming that all steel sections used to construct office and public buildings globally are as over-provided as those analysed in chapter 4, then 37% of the 20 Mt of sections used in buildings is not necessary. This equates to 7.4 Mt of steel annually, or 15 MtCO₂;
- Assuming that all industrial and retail clients can be convinced to design for deconstruction as described in chapter 5 and this allows all sections used in these sectors to be reused infinitely, then this would reduce demand for steel by 40 Mt annually, equating to 81.2 MtCO₂;
- Assuming that 40% of sections used in office globally are used compositely (the same proportion as UK) and that the demountable connector tested in chapter 6 allows all of them to be reused infinitely, this would reduce demand for steel by 4 Mt annually, and reduce emissions by 8.1 MtCO₂.

Combining these three values to arrive at a single figure requires an assessment of the trade-offs between them — for example some degree of repetition might be desirable when designing for deconstruction and reuse, which hinders efforts for a highly-utilised study. Therefore a single steel- or CO₂-reduction figure is not proposed. However it should be noted that the potential for material reduction in construction appears to be so large that substantial savings may be made initially before needing to consider trade-offs. In particular the strategy of designing for deconstruction (chapter 5) should be prioritised for implementation as this has the potential to reduce steel demand by four times more than either of the other options.

7.3 Future work arising from this research

As a first detailed analysis of the opportunities for material efficiency in construction, this research has found significant scope for saving in each of the areas studied. Further research questions, leading on from findings of each chapter, are proposed below in turn. It is postulated that similar analyses of opportunities for material efficiency in other construction materials will yield comparable emissions reduction potential. Further savings are also likely if material efficiency opportunities are investigated for different applications of steel in construction. As noted in section 7.2, trade-offs between strategies require consideration but this is not yet a priority.

Further research questions by chapter

The global estimates of steel use in construction, presented in chapter 3, would be enhanced by inclusion of data from developing countries, particularly China and India, where large amounts of construction materials are being used. Future research would also examine material flows over the life of a typical building, including effects of repairs and replacements to give a holistic account of embodied environmental impacts.

Chapter 4's finding that average utilisation of beams in buildings is just above 50% prompts the question "how much extra would it cost to achieve higher utilisation?". Future work could examine this question and develop simple rules/tools to enable designers to quickly increase average utilisation. Further rationalisation probably occurs at fabrication and construction stages, so research comparing the 'as built' material mass with that required would determine the total level of excess material provided. This type of analysis could be repeated for different products (e.g. rebar), different materials (e.g. timber) and different sectors (e.g. utility networks) to form an overview of the potential to use less material in construction. As noted in the chapter, yet further material savings could result from producing geometrically-optimised elements which better match stress profile and material provision — e.g. variable-section beams as proposed by Carruth (2012) which require 30% less steel than uniform-section elements.

Chapter 5 identified that no deconstructable, ground-bearing foundation system exists. Future research could examine methods to achieve level ground with uniform stiffness to accommodate a rigid deconstructable system, or research could develop a system that could accommodate unlevel or non-uniform ground conditions while still providing a sufficiently stiff and level platform to build on, or a combination of these two ideas. Additionally it was noted that research is needed to develop deconstructable floor systems that can deflect uniformly — potential solutions are lateral connectors between units that share load, a demountable layer between the structural unit and finish that tolerates movement between units, or a flexible finish that itself can tolerate deflection but also meet other performance requirements. This research could be progressed with assistance from those clients that view adaptability as offering substantial commercial advantage, identified in the chapter.

Chapter 6 investigates demountable steel-concrete connections and concludes that further research could optimise the connector geometry and material, tailor design guidance to such connectors, develop analytical models to better predict beam behaviour (particularly the influence of bolt slip on performance), and investigate

methods to construct beams with less time and cost. A truly demountable composite system would allow reuse of the slab as well as the beam however. One solution for this would be to precast slab units with pre-installed bolted connectors. Precasting would remove the ‘construction case’ from the decking design and allow profiles to be optimised for the permanent design case. Precast units would require a concrete-concrete connection (or at least a unit-unit connection) to ensure they deflect together and potentially to transfer ‘diaphragm’ forces; decking could be extended to the sides of slabs to allow steel-steel connections to be used to permit these load transfers. Research in this area would allow reuse to become a possibility for concrete structures, which are built in greater numbers worldwide than steel ones but are practically impossible to deconstruct and reuse.

A related challenge to reusing construction elements, such as steel sections, is insuring it. A steel manufacturer provides a warranty (a guarantee of performance) for its products, which forms part of the warranty a contractor will give the client for a building, redeemable if the steel is found deficient. For reclaimed steel this chain of warranties does not exist so if the client wants financial protection against the risk of defective steel, another party will have to provide it. A warranty is effectively insurance, so insurers are most likely to be able to provide the financial protection, but it is not known what criteria insurers would (or should) use — the types of testing, the sampling rate and the influence of information on both of these. Research is required to provide a scientific background for a testing regime and hence allow the risk of reusing steel to be mitigated the same as new steel.

Other construction materials

The material efficiency strategies could be applied to cement, the second largest source of industrial CO₂ emissions, informed by an analysis of its use in construction. Allwood et al. (2012) present a concise start to this work but further research is necessary to ascertain the potential savings in this material. A number of the professionals interviewed for chapter 4 remarked that rationalisation of concrete is greater than that for steel due to its lower cost and quality control, e.g. the concrete slab thickness for an entire floor might be governed by a small, highly-stressed area. One solution, as proposed by Orr et al. (2011), would be to use fabric formwork to cast optimised shapes — they report that up to 40% less concrete is required compared with prismatic elements. Concrete poured in-situ is practically impossible to deconstruct and reuse so research is needed to develop concrete units and connections that allow this to occur.

Timber is another widely-used construction material, with a different set of flows and uses, and thus a different set of material efficiency opportunities that merit further investigation.

Different uses of steel in construction

The results of chapter 3 found that large tonnages of rebar and sheet/plate are used annually in construction, probably with proportionately large opportunities for material efficiency, in excess of those found for sections. Future research would examine the uses of these products in greater detail (particularly for sheet) and examine opportunities to use less, reuse more or use them for longer. Rebar design involves more rationalisation than for sections as it is more laborious to design and construct — thus the potential to use less is probably greater. Rebar is also used to reduce crack widths (a non-structural application) — is there another method of achieving this outcome? Sheet is used in cold-formed sections (which may also be rationalised), in decking (which is governed by its performance during construction) and for cladding — are there opportunities to use less material in these applications or for them to be reused?

Trade-offs between strategies

As noted in section 7.2, material efficiency strategies cannot be linearly summed — reducing the amount of material in buildings will reduce the amount available for reuse in future. Also installing highly-optimised or -specialised designs may prevent reuse if elements get damaged in use or are unique to one setting as a result — there is a ‘trade-off’ between strategies. Future research could examine these trade-offs and determine practical limits — i.e. the level of average utilisation beyond which reuse potential is impaired, if such a limit exists at all — and provide guidance to designers. However given that building steel is just over 50% utilised, that buildings are generally not demolished due to physical failures and that only 1.5% of steel sections exiting use are being reused, it appears that material efficiency savings can be made in each of these areas before the impacts of trade-offs merit further investigation.

7.4 Summary

This research has examined opportunities for three material efficiency strategies for steel in construction and found substantial savings potential in each. If achieved, these steel savings could negate tens of millions of tonnes of carbon dioxide emission, and thereby reduce climate change and its negative impact. Wider implications of the findings have been discussed and further opportunities for research have been identified, which will hopefully reveal further potential to reduce emissions. As noted by the IPCC, taking action sooner is likely to result in less cost both economically and environmentally (Edenhofer et al., 2014) if we are to give future generations the opportunities and lifestyles we currently enjoy. It is hoped that this thesis can contribute in a small way towards this action.

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Appendix A

Allocation of steel products to sectors

As described in chapter 3 section 3.1.1, steel products were allocated to the designated building and infrastructure sectors by combining information from a range of sources, using estimates and proxies where no direct data were available. This appendix details the sources used and assumptions made in order to allocate each product to the ten sectors for the UK (section A.1) and globally (section A.2).

A.1 Methodology to allocate products to sectors for UK

The exact reasoning and assumptions used to allocate each product to the end-use sectors for the UK are detailed below. The final percentage allocations are presented also.

Sections: Detailed information on the use of heavy sections by sector is available from the British Constructional Steelwork Association (BCSA). The BCSA’s membership consists of steelwork contractors and producers, from whose data annual steelwork tonnages by sector are reported in BCSA (2010), used as a basis for the sections allocation. These tonnages include fabricated sections in buildings (infrastructure plate girders are not reported through this route), so that tonnage is removed. Light sections, not included in the BCSA tonnages, were assumed to follow this distribution also. Table A.1 shows the values used.

Sector	Heavy sections (kt)	Final sections allocation %
Buildings		
Industrial	622	48%
Commercial	205	16%
Offices	171	13%
Public	82	6%
Residential	46	4%
Other	79	6%
Infrastructure		
Utilities	27	2%
Rail	0	0%
Bridges	25	2%
Other	29	2%

Table A.1: 2006 UK section allocation by sector

A.1 Methodology to allocate products to sectors for UK

Reinforcement: No central database for reinforcement exists, so the distribution was estimated by combining sources, assuming reinforcing bar and wire mesh are used in the same proportions. Celsa UK provide a representative allocation for the UK market in 2010, but it was speculated 2006 values would be different because anecdotal evidence suggests proportionately more infrastructure was built in 2010 than 2006. The Mineral Products Association — Cement, a cement industry trade body, provide a breakdown of cement use by sector for 2005 (MPA - Cement, 2008), which was assumed to be close to 2006 proportions. Comparing this breakdown to the Celsa one shows that rebar is broadly used in proportion to cement, except in the residential sector. It was estimated half of cement in residences is unreinforced concrete blocks (MPA - Cement, 2011), thus the rebar proportion for residential is taken as half that for all other sectors. The final allocation was calculated by multiplying the Celsa data by the average of both the Celsa and cement data buildings:infrastructure ratios, resulting in the values shown in table A.2.

Sector	2005 cement estimate	2010 rebar estimate	Final rebar allocation %
Buildings	72%	46%	56%
Industrial	9%	2%	2%
Commercial	19%	11%	14%
Offices		6%	8%
Public	11%	14%	17%
Residential	32%	13%	15%
Other	-	-	-
Infrastructure	28%	54%	44%
Utilities	-	36%	30%
Rail	-	3%	2%
Bridges	-	1%	0%
Other	-	14%	12%

Table A.2: 2006 UK rebar allocation by sector

A. ALLOCATION OF STEEL PRODUCTS TO SECTORS

Sheet/plate: Steel sheet and plate have many disparate uses and there is no one entity holding data on all uses, therefore industry sources were used to allocate by product. Roofing/cladding is only used in buildings (MCRMA, 2011) and was assumed proportional to sections use. The BCSA stated that floor-decking is specific to multi-storey buildings, and likewise is allocated proportional to sections in non-industrial sectors. Sheet piles are mainly used in infrastructure, with the main UK manufacturer, Arcelor Mittal, informing allocation. The use of cold-formed sections was assumed proportional to hot-rolled ones. The BCSA informed the allocation of girders built up by welding plates. The final allocation between sectors is given in table A.3.

Sector	Roofing/ cladding	Decking	Sheet piles	Cold-formed sections	Plate girders
Buildings					
Industrial	67%	-	-	51%	32%
Commercial	12%	35%	-	17%	12%
Offices	10%	29%	-	14%	10%
Public	5%	14%	-	7%	5%
Residential	3%	8%	-	4%	3%
Other	5%	14%	-	7%	5%
Infrastructure					
Utilities	-	-	33%	-	-
Rail	-	-	33%	-	-
Bridges	-	-	-	-	28%
Other	-	-	33%	-	-

Table A.3: 2006 UK allocation of each sheet/plate product by sector

Rails: Only used in one application in construction — in railroad infrastructure — hence allocation is trivial.

A.1 Methodology to allocate products to sectors for UK

Tubes: Allocated according to product type, based on interview data from the main UK producer, Tata: linepipe (an industry term for pipe products) are all in utility networks (mainly oil and gas); structural tube are allocated the same as structural sections; non-structural tubes are used evenly in handrails, fencing and street furniture — the former two were allocated following sections’ distribution, the latter is allocated to ‘other’ infrastructure; ‘generic’ tubes (comprising ‘gas list’ and ‘pressure’ products) are used only in buildings and assumed proportional to sections. Table A.4 gives the final allocation of each product by sector.

Sector	Linepipe	Structural	Non-structural	Generic
Buildings				
Industrial	-	48%	32%	52%
Commercial	-	16%	11%	17%
Offices	-	13%	9%	14%
Public	-	6%	4%	7%
Residential	-	4%	2%	4%
Other	-	6%	4%	7%
Infrastructure				
Utilities	100%	2%	1%	-
Rail	-	0%	-	-
Bridges	-	2%	1%	-
Other	-	2%	35%	-

Table A.4: Allocation of tube by sector for the UK in 2006

A.2 Methodology to allocate products to sectors globally

The detailed reasoning and assumptions used to allocate each product to the end-use sectors globally are described below. Global data were difficult to find, so regional data (e.g. for EU or USA) were used instead, with proportions from the UK results adding detail as necessary. The final percentage allocations are presented.

Sections: In the absence of world data, it was assumed that Europe is a representative sample of global sections consumption. The European Convention for Constructional Steelwork (ECCS) is a trade association which provides detailed information on the use of heavy sections in 11 European countries, and ECCS (2009) was the basis for the global sections allocation. Light sections were assumed to be used in proportion with heavy sections. Values averaged over three years (2008–2010) are calculated, accounting for non-reporting of sectors by some countries. 2006 proportions were not expected to vary much from these calculated values (as discussed in chapter 3 section 3.1), shown in table A.5.

Sector	Final sections allocation %
Buildings	
Industrial	39%
Commercial	8%
Offices	6%
Public	9%
Residential	5%
Other	9%
Infrastructure	
Utilities	10%
Rail	-
Bridges	5%
Other	8%

Table A.5: 2006 allocation of sections globally

A.2 Methodology to allocate products to sectors globally

Reinforcement: As for the UK, the lack of statistics for reinforcing bars and mesh presents a challenge to allocation. This is more pronounced for the world than for the UK. To generate a global allocation, a weighted average was taken from: UK data; a cement breakdown for the USA (Portland Cement Association, 2011); a cement breakdown for Turkey (Akansa Cement, 2012). Rebar was assumed to be used in proportion with cement apart from in the residential sector, where half as much rebar is used per unit concrete as other sectors. The results are given in table A.6; the rebar breakdown in table A.2 is used to allocate between infrastructure sectors.

Sector	% of cement				Final rebar allocation %
	UK	US	Turkey	Weighted avg.	
Buildings	72	57	83	65	52
Industrial	9	6	-	5	6
Commercial/offices	19	10	13	11	16
Public	11	8	4	7	8
Residential	32	33	66	41	21
Infrastructure	28	43	17	35	48

Table A.6: 2006 global allocation of rebar, including cement proxy values

Sheet/plate: The lack of plate/sheet data, in any form, on a worldwide scale meant that a proxy was employed. As the UK sheet allocation was remarkably similar to the UK sections allocation, the allocation of sheet/plate tonnage globally was assumed to follow the global sections breakdown.

Rail: As for the UK, rails have only one application in construction: ‘rail’; hence allocating is trivial.

A. ALLOCATION OF STEEL PRODUCTS TO SECTORS

Tube: The only data found on tubes globally are for linepipe: Siemens VAI (2011) estimates that linepipe is 35% of global tube. This is approximately the same percentage as for the UK, so the other tube products were allocated following the same method as for UK tube, except that where the UK section breakdown was followed, the global sections breakdown is followed instead. Results are given in table A.7.

Sector	Linepipe	Structural	Non-structural	Generic
Buildings				
Industrial	-	39%	26%	51%
Commercial	-	8%	5%	10%
Offices	-	6%	4%	8%
Public	-	9%	6%	12%
Residential	-	5%	3%	7%
Other	-	9%	6%	12%
Infrastructure				
Utilities	100%	10%	7%	-
Rail	-	0%	-	-
Bridges	-	5%	4%	-
Other	-	8%	39%	-

Table A.7: Allocation of tube products by sector globally in 2006

A.3 Method to allocate products to sectors globally using Turkey data only

To measure the sensitivity of the global results to data from developed nation sources, a concise global analysis was undertaken using information from Turkey (a developing nation) only. The assumptions for this analysis are listed by product and the values obtained are shown in table A.8.

Sections: The use of heavy sections by sector in Turkey is provided by ECCS (2009) for the years 2008–2010. The average of these years is assumed to be the allocation for all sections globally.

Rebar: Cement data from Akcansa Cement (2012) — shown in table A.6 — is used as the basis for global rebar allocation, assuming (as previously) that half of cement used in the residential sector is unreinforced.

Sheet/plate: No sheet/plate data could be found for Turkey, therefore the sheet allocation follows that of sections.

Rail: As previously, all rail is allocated to the ‘rail’ category.

Tube: Data on tube use in Turkey could not be found so allocation follows that of sections.

Each of the sector allocations in table A.8 was multiplied by the corresponding *global* product tonnage. The totals by sector were found by summing these values. The ‘total’ value in the table was found by dividing these sector totals by the total steel tonnages used in construction.

Sector	Sections	Rebar	Sheet/plate	Rail	Tube	Total
Buildings	63%	66%	63%	-	63%	63%
Industrial	32%	-	32%	-	32%	18%
Commercial/offices/public	20%	26%	20%	-	20%	32%
Residential	4%	33%	4%	-	4%	16%
Other	6%	7%	6%	-	6%	6%
Infrastructure	37%	34%	37%	100	37%	37%

Table A.8: Allocation of tube products by sector globally based on Turkey data only

A. ALLOCATION OF STEEL PRODUCTS TO SECTORS

Appendix B

Design criteria and example calculation for evaluating utilisation ratios

B.1 Design criteria

This section lists the details of the design criteria used to evaluate the governing utilisation ratio for each beam and column in each building, as referred to in chapter 4 section 4.1.1.

1. Moment capacity

- (a) About major axis
- (b) About minor axis
- (c) Reduced moment capacity — e.g. at holes, near support
 - i. About major axis
 - ii. About minor axis

2. Shear capacity

- (a) In direction of minor axis
- (b) In direction of major axis

3. Axial capacity

B. DESIGN CRITERIA AND EXAMPLE CALCULATION FOR EVALUATING UTILISATION RATIOS

4. Buckling*

- (a) Lateral torsional buckling
- (b) Strut buckling at various sections

5. Combined axial and moment buckling

- (a) About major axis
- (b) About minor axis

6. Deflection

- (a) Due to dead load
- (b) Due to imposed load
- (c) Due to all loads

Other criteria, such as torsion and combined shear and torsion, were included in U/R calculation when specified as governing by calculations or by designer, but otherwise were omitted.

All checks done to worst loading scenario.

*Shear web buckling was checked on a pass/fail basis – i.e. not used to calculate U/R.

B.2 Example calculation

An example calculation is presented, showing how U/Rs are calculated for each criteria and how the governing criteria is selected.

Scenario

UB 406x178x85 S355 secondary steel beam supporting a non-composite slab in an office building; 9m span @ 3m c/c between beams.

Assumptions:

1. Calculations done following Eurocode 3-1-1 (BSI, 2005);
2. Beam geometrical values from standard catalogue (SCI, 2009)
3. Only vertical loads applied to beam;
4. Pin connections either end (i.e. simply-supported behaviour);
5. Slab provides lateral restraint to top flange of beam along entire length;
6. Deflection limits:
 - (a) Dead load = Span / 250
 - (b) Live load = Span / 360
 - (c) Total load = Span / 200

Loading

All loads uniformly distributed:

- Dead load =
 - Selfweight of beam: 0.85 kN/m
 - Weight of slab: $0.15 * 24 = 3.6 \text{ kN/m}^2$.
- Superimposed dead load = 3.5 kN/m^2 , composed of:
 - False ceiling: 0.25 kN/m^2 ;
 - Services: 0.25 kN/m^2 ;

B. DESIGN CRITERIA AND EXAMPLE CALCULATION FOR EVALUATING UTILISATION RATIOS

- Raised floor: 0.5 kN/m^2 ;
- Blockwork partitions: 2.5 kN/m^2 .
- Imposed loading = 3 kN/m^2 .

Per meter characteristic loading:

- Dead = $0.85 + 3 * (3.6 + 3.5) = 22.2 \text{ kN/m}$
- Live = $3 * 3 = 9 \text{ kN/m}$

Limit state loading:

- Serviceability Limit State, $w_{\text{SLS}} = 22.2 + 9 = 31.2 \text{ kN/m}$
- Ultimate Limit State, $w_{\text{ULS}} = 1.35 * 22.2 + 1.5 * 9 = 43.5 \text{ kN/m}$

Moment U/R

About major axis

Applied moment, $M_{Ed} = w_{\text{ULS}} l^2 / 8 = 43.5 * 9^2 / 8 = 440.4 \text{ kNm}$

Moment capacity, $M_{c,Rd} = W_y f_y / \gamma_{M0} = 345 * 1730 / 1 = 596.9 \text{ kNm}$

U/R = $440.4 / 596.9 = 0.74$

About minor axis

No applied moment about minor axis, therefore U/R = 0

At points of reduced capacity

No holes in beam.

Shear U/R < 0.5 therefore no points of reduced capacity

Therefore U/R = 0

Governing moment U/R = max { 0.74; 0; 0 } = 0.74

Shear U/R**In direction of major axis**

Applied shear force, $V_{Ed} = w_{ULS}l/2 = 43.5 * 9/2 = 195.8 \text{ kN}$

Shear capacity, $V_{Rd,pl} = A_v f_y / (\sqrt{3} \gamma_{M0}) = 4848 * 345 / \sqrt{3} = 965.6 \text{ kN}$

$$U/R = 195.8 / 965.6 = 0.20$$

In direction of minor axis

No loading in minor axis, therefore $U/R = 0$

$$\text{Governing shear } U/R = \max \{ 0.20; 0 \} = 0.20$$

Axial U/R

No axial loads on beam, therefore $U/R = 0$

Buckling U/R

Assumption 5 states that compression flange restrained along length, preventing buckling, therefore $U/R = 0$

Combined axial and moment U/R

No axial force, therefore combined cases not relevant

Deflection U/R**Due to dead load**

$$\text{Deflection, } \delta_{DL} = w_{DL} l^4 / 384 E I_{yy} = 22.2e3 * 9^4 / 384 * 205e9 * 31700e-8 = 5.8 \text{ mm}$$

$$\text{Deflection limit from assumption 6} = L / 250 = 9000 / 250 = 36 \text{ mm}$$

$$U/R = 5.8 / 36 = 0.16$$

B. DESIGN CRITERIA AND EXAMPLE CALCULATION FOR EVALUATING UTILISATION RATIOS

Due to live load

$$\text{Deflection, } \delta_{KL} = w_{LL}l^4/384EI_{yy} = 9e3 * 9^4/384 * 205e9 * 31700e-8 = 2.4 \text{ mm}$$

$$\text{Deflection limit from assumption 6} = L / 360 = 9000 / 360 = 25 \text{ mm}$$

$$U/R = 2.4 / 25 = 0.10$$

Due to total load

$$\text{Deflection, } \delta_{SLs} = w_{SLs}l^4/384EI_{yy} = 31.2e3 * 9^4/384 * 205e9 * 31700e-8 = 8.2 \text{ mm}$$

$$\text{Deflection limit from assumption 6} = L / 200 = 9000 / 250 = 45 \text{ mm}$$

$$U/R = 8.2 / 45 = 0.18$$

$$\text{Governing deflection } U/R = \max \{ 0.16; 0.10; 0.18 \} = 0.18$$

Governing U/R

The governing U/R is the highest across all criteria, i.e. $U/R = \max \{ 0.74; 0.20; 0; 0; 0; 0.18 \} = 0.74$. This value is entered into the analysis for this beam.

Appendix C

Detailed results of utilisation analysis

This appendix gives an abridged set of the results of the 23 buildings analysed, as referenced in chapter 4 section 4.2. It features four examples selected from the 23 sets of results — the buildings with: the highest average U/R , the lowest average U/R , the largest steel tonnage, the smallest steel tonnage. The full set of results, running to 66 pages with 88 figures and 23 tables, is available in the Supporting Information document accompanying the journal article “Utilisation of structural steel in buildings”. As agreed with the providers of the raw data, each building is identified only by a number, with the following information provided:

- Building type;
- Number of beam data obtained and number analysed;
- Table with summary of results by floor and overall;
- Graph of frequency of occurrence against utilisation ratio for each floor and overall;
- Plot of beam layout on each floor analysed showing utilisation ratio of each beam;
- Graph of frequency of occurrence against utilisation ratio for the columns in the building.

For all buildings it was possible to provide the first four items. However limitations in the data resulted in three categories of building for the remaining two items:

C. DETAILED RESULTS OF UTILISATION ANALYSIS

- For 17 buildings over 70% of the beams on each floor could be plotted, and once this level was reached the floor was deemed finished, as patterns were clear. Where necessary to complete the floor geometry, and so aid comprehension of the data, omitted beams were added in manually (coloured grey). Column locations were also added manually for this reason.
- For 6 buildings (#s 8, 9, 11, 16, 17, 21) there was insufficient information on beam layout to produce plots;
- For 1 building (# 10) there was insufficient information to produce a graph of column data.

For graphs, utilisation ratios are groups into bands of 10% to aid clarity; these bands are inclusive of the identifying upper bound, for example the data point at 0.2 includes U/Rs from 0.11 to 0.20. For all plots of beam utilisation ratio per floor the legend in figure C.1 is used.

Legend

Red: $0.75 \leq U/R < 1.0$
Orange: $0.5 \leq U/R < 0.75$
Green: $0.25 \leq U/R < 0.5$
Blue: $0 \leq U/R < 0.25$
Grey: U/R unknown
or invalid
I column location

Figure C.1: Legend for all plots of beam utilisation ratio per floor

Building #10 (highest avg. U/R)

Type: office

35 of 48 beams analysed (73%)

Level	No. beams analysed	% of total steel mass	Avg. U/R	Weighted avg. U/R	Top 5 Beams	
					No.	%
Roof	35	100%	0.90	0.96	3	100

Table C.1: Summary of results by floor for building #10

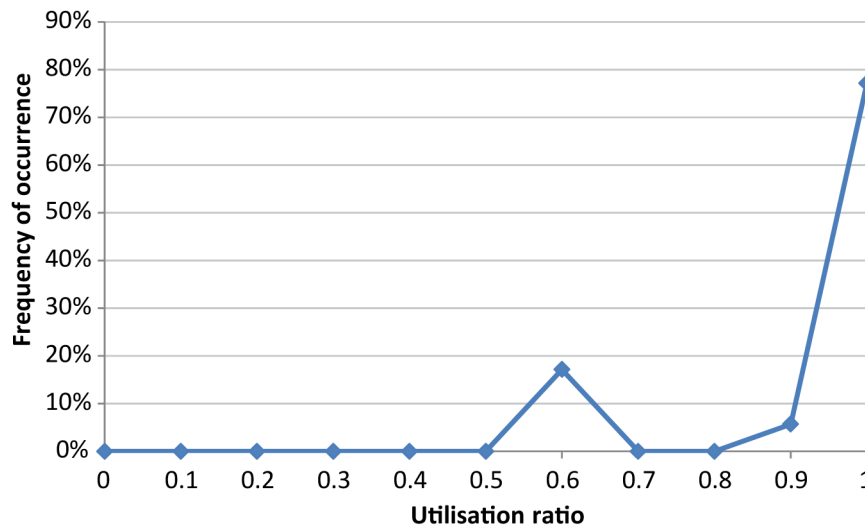


Figure C.2: Graph of frequency of occurrence against utilisation ratio for beams by floor and overall for building #10

Roof

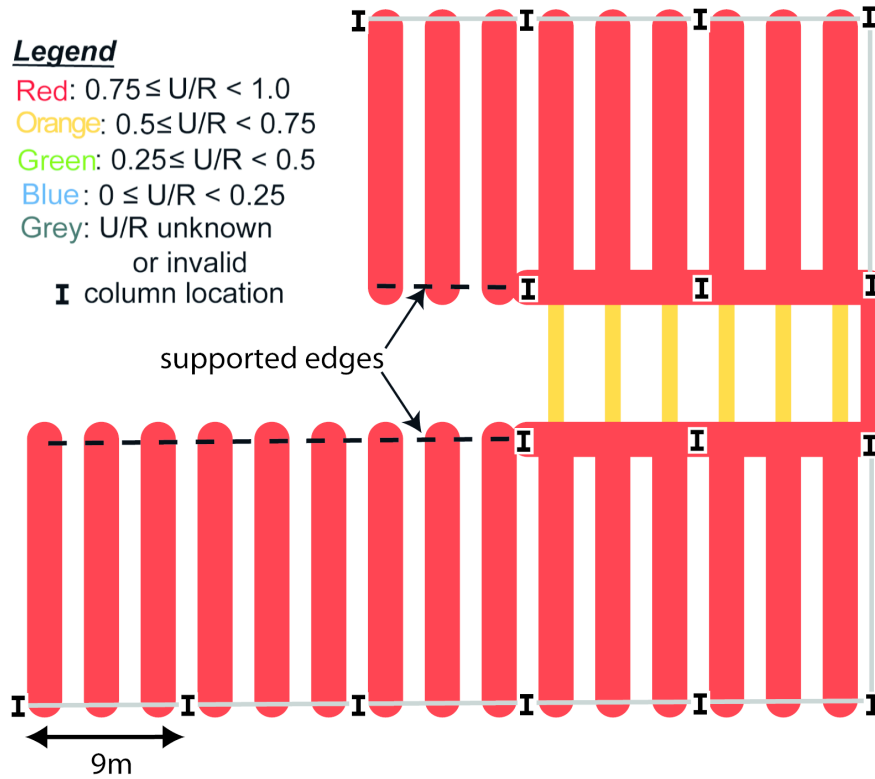


Figure C.3: Plot of floor of buildings #10 showing beams coloured according to utilisation ratio

Engineer's comments

Deflections governed design. Not surprised that had high U/R as had time to design thoroughly and no late changes were made.

Columns

Insufficient information was available about the columns in this building to allow analysis.

Building #6 (lowest avg. U/R)

Type: office & education

700 of 1194 beams analysed (59%)

Level	No. beams analysed	% of total steel mass	Avg. U/R	Weighted avg. U/R	Top 5 Beams	
					No.	%
Roof	197	19%	0.12	0.22	139	71%
2nd floor	229	28%	0.11	0.27	195	85%
1st floor	197	34%	0.20	0.30	160	81%
Other	77	19%	0.17	0.16	-	-
Total	700	100%	0.15	0.25	541	77%

Table C.2: Summary of results by floor for building #6

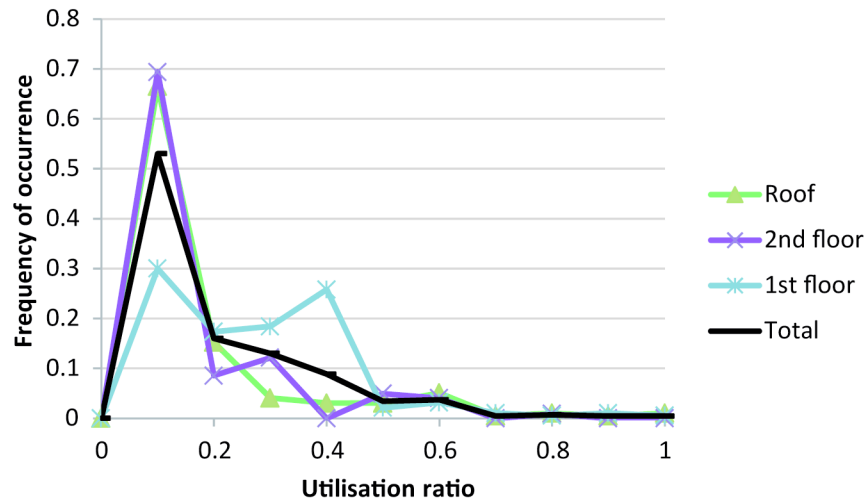


Figure C.4: Graph of frequency of occurrence against utilisation ratio for beams by floor and overall for building #6

C. DETAILED RESULTS OF UTILISATION ANALYSIS

1st floor

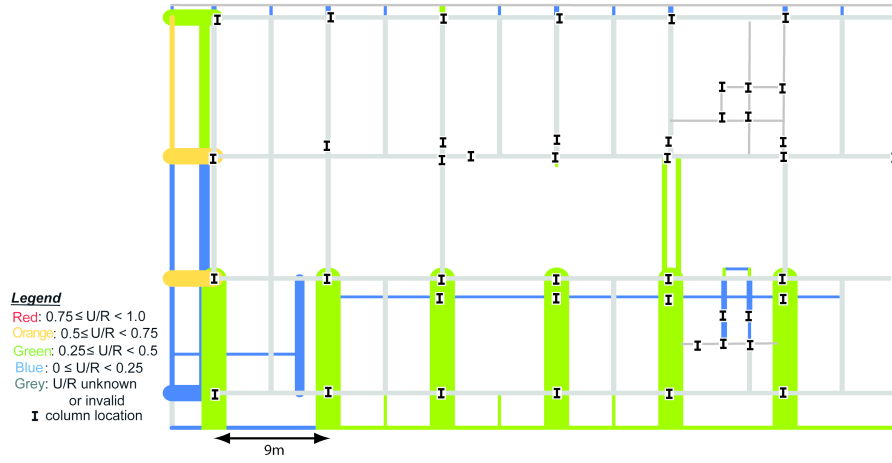


Figure C.5: Plot of first floor of building #6 showing beams coloured according to utilisation ratio

2nd floor

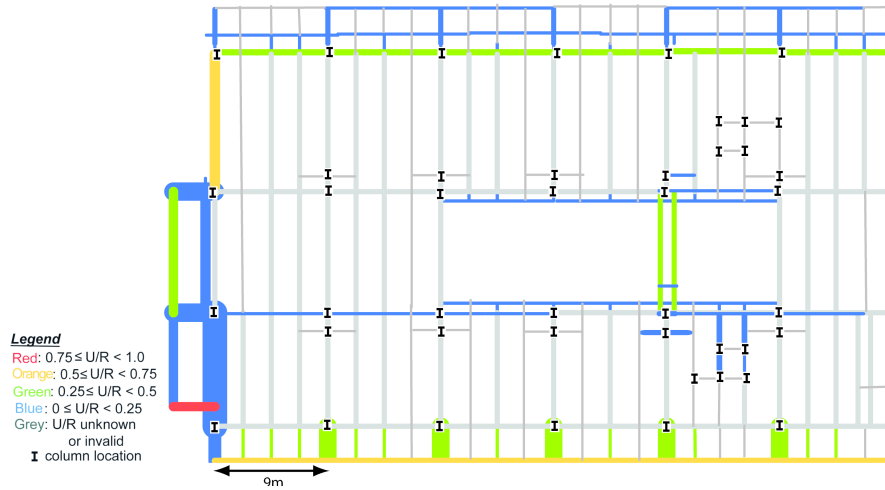


Figure C.6: Plot of second floor of building #6 showing beams coloured according to utilisation ratio

Roof

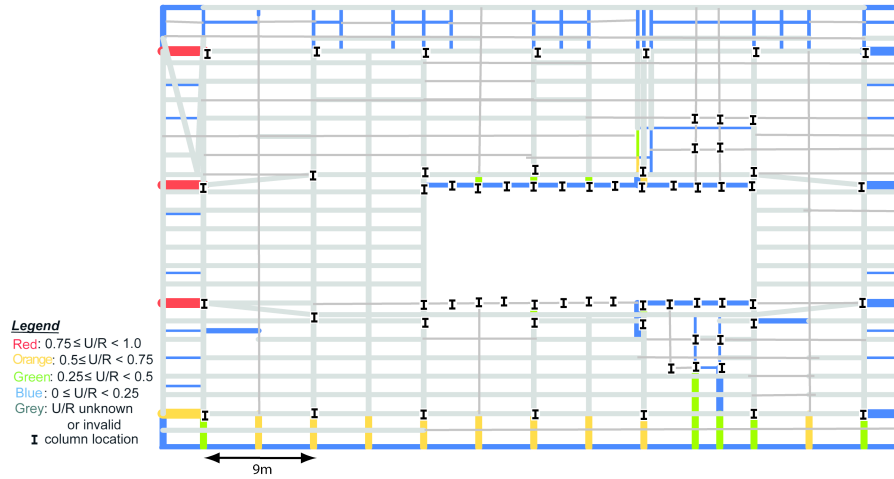


Figure C.7: Plot of roof of building #6 showing beams coloured according to utilisation ratio

Engineer's comments

Computer model used mainly for stability and column design purposes – may explain why so many beams omitted from analysis. Design around edges governed either by vibration or by minimum sizes for façade supporting steelwork (to facilitate faster construction).

Columns

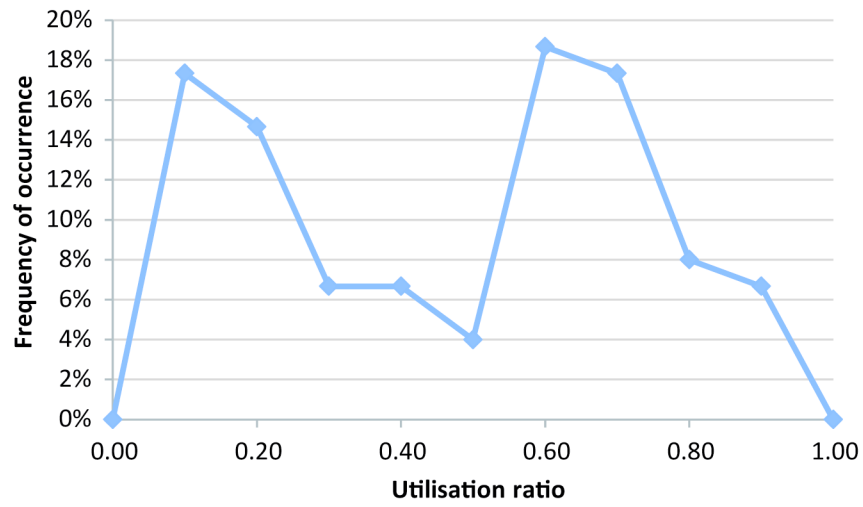


Figure C.8: Graph of frequency of occurrence against utilisation ratio for columns by floor and overall for building #6

Building #7 (highest tonnage)

Type: school

764 of 891 beams analysed (86%)

Level	No. beams analysed	% of total steel mass	Avg. U/R	Weighted avg. U/R	Top 5 Beams	
					No.	%
Top Roof	125	8%	0.17	0.25	125	100%
Roof	196	22%	0.28	0.39	158	81%
3rd floor	114	21%	0.42	0.46	89	78%
2nd floor	129	21%	0.43	0.54	118	91%
1st floor	174	26%	0.39	0.52	150	86%
Other	26	2%	0.14	0.14	-	-
Total	764	100%	0.33	0.45	470	62%

Table C.3: Summary of results by floor for building #7

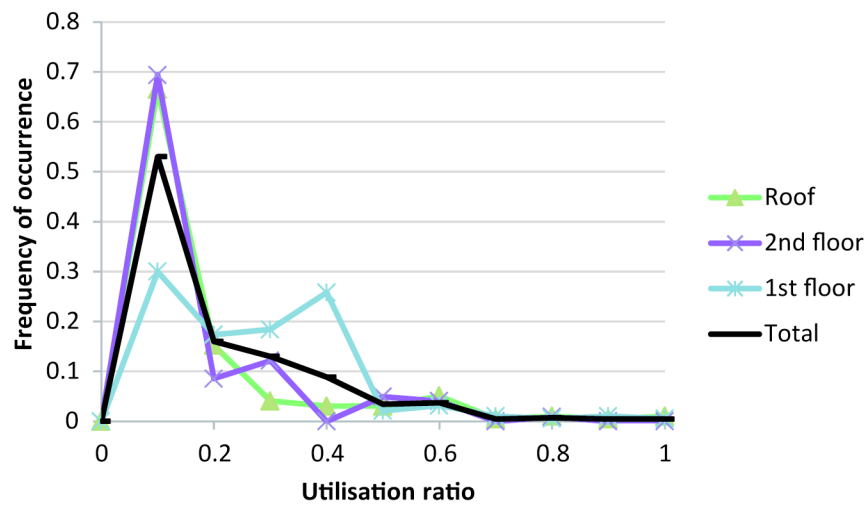


Figure C.9: Graph of frequency of occurrence against utilisation ratio for beams by floor and overall for building #6

C. DETAILED RESULTS OF UTILISATION ANALYSIS

1st floor

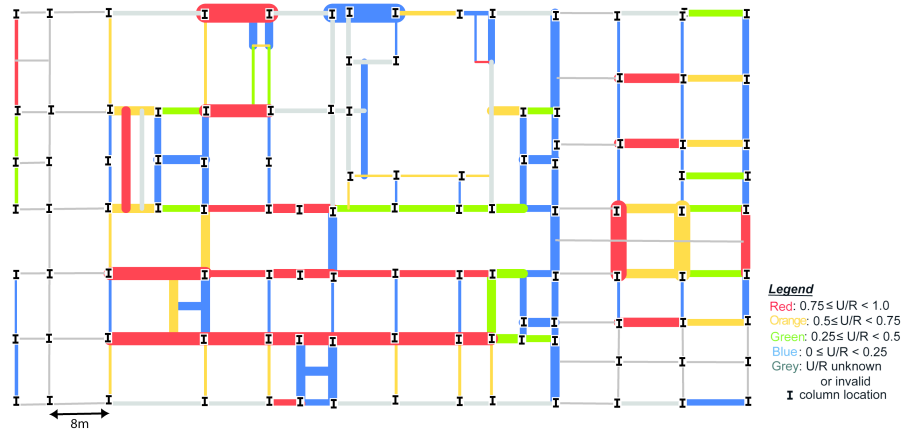


Figure C.10: Plot of first floor of building #7 showing beams coloured according to utilisation ratio

2nd floor

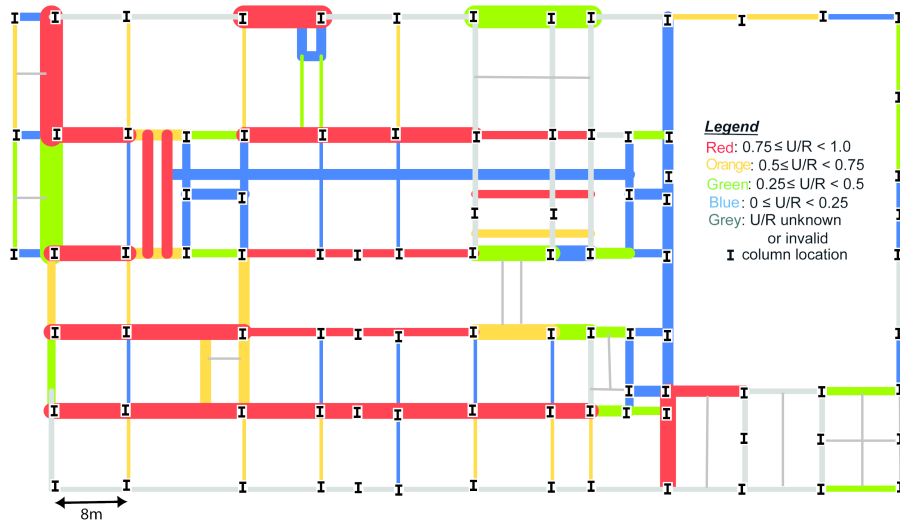


Figure C.11: Plot of second floor of building #7 showing beams coloured according to utilisation ratio

3rd floor

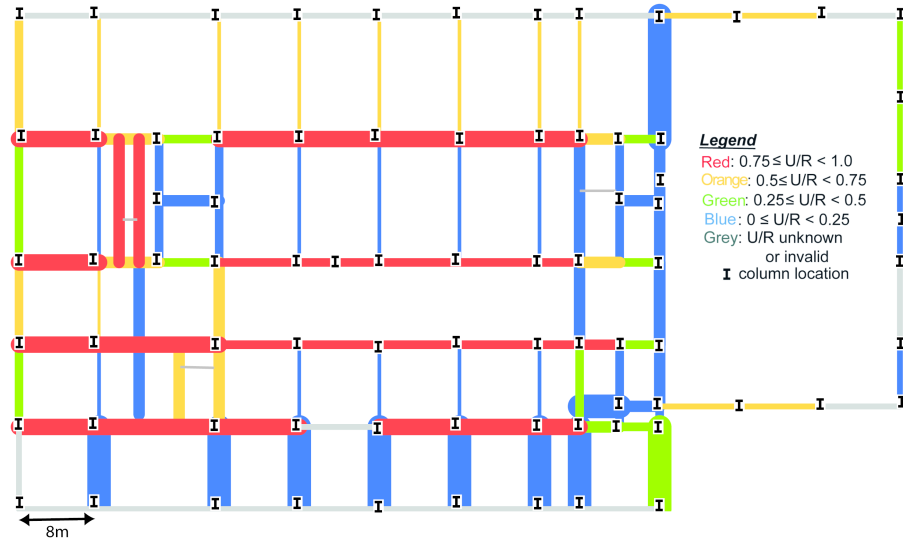


Figure C.12: Plot of third floor of building #7 showing beams coloured according to utilisation ratio

Roof

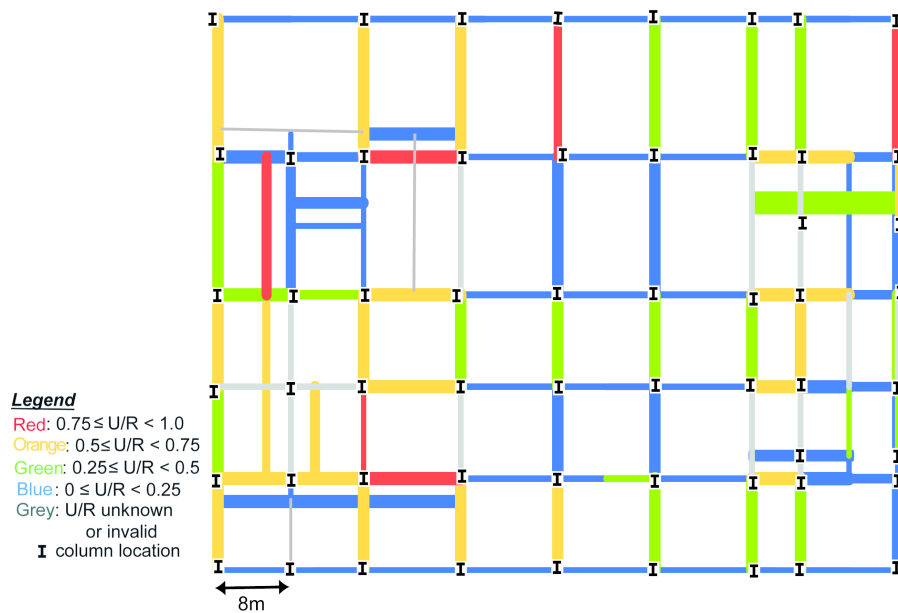


Figure C.13: Plot of roof of building #7 showing beams coloured according to utilisation ratio

C. DETAILED RESULTS OF UTILISATION ANALYSIS

Engineer's comments

Vibration governed in some places but mainly stress and deflection governed.

Columns

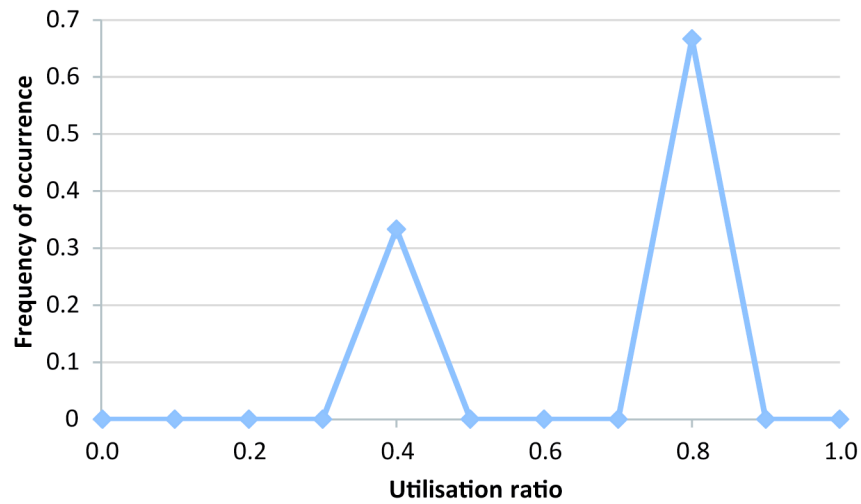


Figure C.14: Graph of frequency of occurrence against utilisation ratio for columns by floor and overall for building #7

Building #5 (lowest tonnage)

Type: office

21 of 21 beams analysed (100%)

Level	No. beams analysed	% of total steel mass	Avg. U/R	Weighted avg. U/R	Top 5 Beams	
					No.	%
Roof	21	100%	0.44	0.41	-	-

Table C.4: Summary of results by floor for building #5

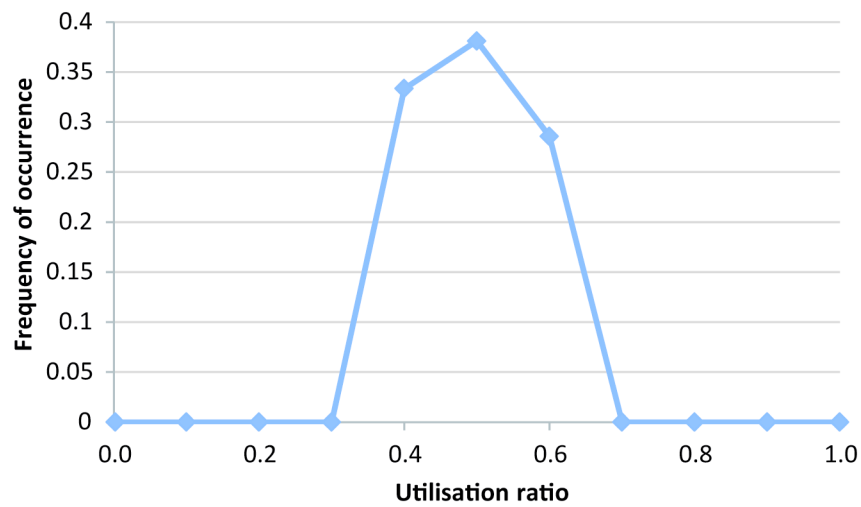


Figure C.15: Graph of frequency of occurrence against utilisation ratio for beams by floor and overall for building #10

Roof

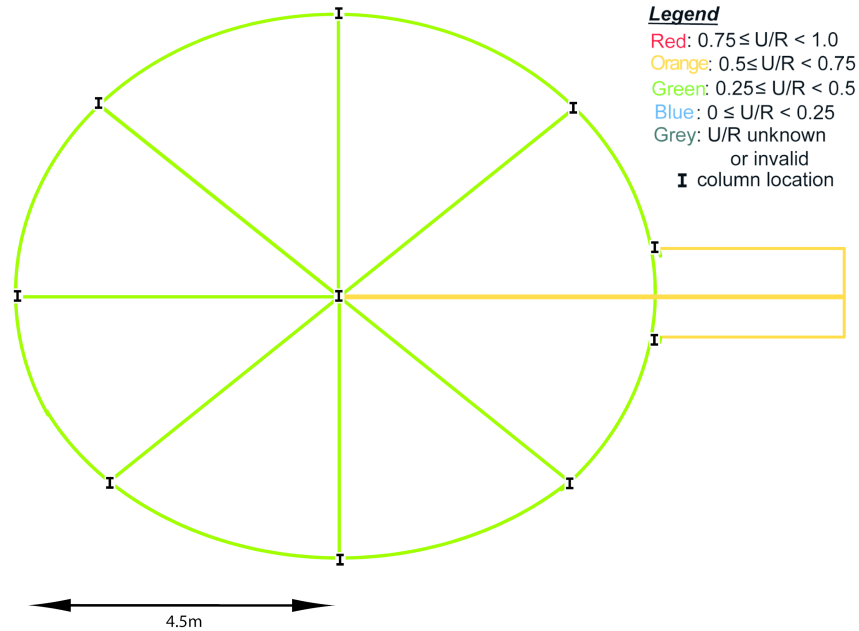


Figure C.16: Plot of roof of building #5 showing beams coloured according to utilisation ratio

Engineer's comments

The applied loads were reduced late in the project programme – too late to redesign, which resulted in spare capacity in places. Deflection governed most elements' design.

Columns

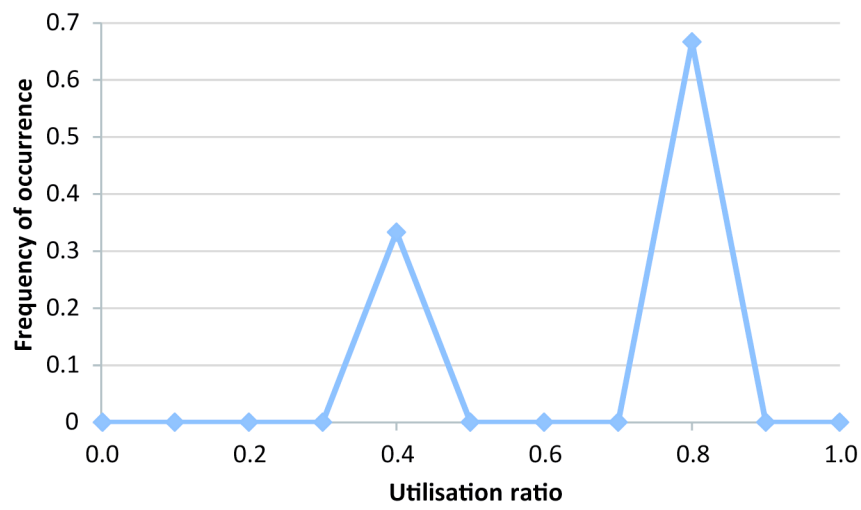


Figure C.17: Graph of frequency of occurrence against utilisation ratio for columns by floor and overall for building #5

Appendix D

Prolonging product life interviews

This appendix details the findings from the twelve case study interviews referred to in chapter 5 section 5.2, conducted in order to target design strategies at failure modes. Four of the interviews were conducted by Alexandra C.H. Skelton and three by Daniel R. Cooper, as clearly labelled. This information is included in the Supporting Information document to Cooper et al. (2013).

Case Study 1: Refurbishing modular building

Interview with Technical Manager, Foreman's Relocatable Building System conducted by Muiris C. Moynihan

Modular buildings are manufactured in a factory, transported to site and erected to form an office, school, hotel or retail unit (these are the most common structures, but almost any are possible). Modular buildings are constructed in a controlled factory environment, resulting in health & safety, quality, cost and time benefits, as well as reduced time on site, minimising disruption.

Foremans Relocatable Buildings Systems are a UK company that specialises in refurbishing modular buildings. Owners of building modules contact them to sell on their units; if the unit has certificates to show it was made by a reputable manufacturer, then Foremans will inspect and potentially buy it. Its team disassembles and removes it to their plant in Yorkshire, where it undergoes refurbishment. Firstly, the module is stripped back to its structure and a thorough check undertaken with any repairs made — this allows the structure to be guaranteed for 10 years, regardless of its age on arrival. It is then held in stock until a client purchases it, at which point modern interior finishes & services are installed in accordance with the client's

specifications and the building regulations. Module components can even be combined in a ‘kit-of-parts’ approach to meet non-standard requirements. The finished modules are then transported to the new site and erected in any feasible geometry. In this way, approximately 80% of the steel each module is retained and kept past the lifetime of its parent building.

Case Study 2: Replaceable roll sleeves

Technology Manager (rolling mills and process lines), Siemens VAI conducted by Daniel R. Cooper

The cylindrical work rolls in steel and aluminium rolling mills weigh up to 90 tonnes (t), and exert loads up to 10,000 t. The typical specification for (5 m) plate mills work rolls is a 1.2 m diameter roll weighing approximately 90 t. The work rolls in a mill weigh up to 450 t (including chocks). The roll is made from forged steel with the inner grey steel surrounded by a thick outer layer of chrome steel. The rolls quickly wear, causing problems with surface quality of the rolled product and a danger of explosive disintegration of the rolls. The rolls are consequently replaced every 8 hours of operation. They are then ground down to remove the damaged outer layer, and returned to the mill. This cycle is repeated over a period of 5 years until the radius has reduced by approximately 100 mm, prompting full replacement with the old roll being scrapped.

Recognising that performance degradation is only applicable to the outer surface of the roll, can the life of the remaining steel be prolonged by modularising the inner and outer core? Sleeved rolls consist of a structural core (the “arbour”) and a sleeve that are joined by a shrink fit. These sleeved rolls facilitate repair of older rolls and multiple use of the arbour. Carefully designed and manufactured sleeved rolls have proved to be equivalent to solid rolls in terms of rolled kilometres and tonnages at rolling mills in the Czech Republic. They have only been used occasionally thus far, due to the problems of induced tensile stress from the shrink fit. However, this problem is being slowly overcome with careful design, better materials and finite element analysis.

According to Hajduk et al. (2010) sleeved rolls are used to repair older rolls and to manufacture large rolls that cannot be made as a solid roll. They offer a cost advantage as the arbour can be reused.

Case Study 3: Adaptable foundations

Interviews with two Directors, Infrastructure Division, Arup conducted by Muiris C. Moynihan

Two aspects of foundations were explored: how to design them such that they are adaptable, and how to adapt them once installed so they can be used beyond the life of their original superstructure. Every structure requires some form of foundation. These were mainly shallow footings until the 1950s, when deep foundations (mainly piles) came into the main stream as taller buildings became more common. Since then concrete has been the material of choice for the industry, as it has a major cost advantage over the main alternative, steel.

Adaptable foundations

A large developer in London usually requires foundations that can take a number of different building layouts, based around the likely core layouts and column grids. Design teams are commissioned to develop concept designs for each one with foundation designs worked out which give a maximum of overlap (a ‘totally’ flexible foundation would be at least twice as expensive, if not more). On one project 15% more piles were added to achieve this flexibility, which added less than 1% to project cost. On another project the individual piles were designed with 10% extra capacity which barely added to project costs; this is possibly going to be used to add extra storeys. At the end of building life these foundations are much more likely to be kept for the next building, as it can be quite different to the previous. Considering a typical pile contains 700–1100 kg of steel, and that there may be hundreds on a site, the potential steel saving is huge, especially given that London’s tall buildings are lasting less than 25 years in places and the ground is slowly filling up with piles. The developer’s motivation for specifying adaptable foundations was that foundation work could start without the superstructure being fully decided, giving programme advantages as well as allowing a greater pool of potential clients.

Adapting foundations

At building end-of-life, the superstructure can be readily removed and potentially reused, however this is very difficult and expensive for piled foundations. Instead there are three options: dig out, leave in place, or use again. Digging out concrete piles is a difficult and expensive task as they cannot be pulled out and go to

great depth, so currently many piles are simply left in place and the next set of foundations merely fit in around them. The Reuse of Foundations on Urban Sites (RuFUS) project highlighted the congestion present underground at certain sites in London, where two or three generations of concrete piles have left almost no space for new foundations to go in (Anderson and Chapman, 2005). Concrete piles are very difficult and costly to remove, as well as damaging to the ground to do so — new foundations must go deeper and deeper to get the same capacity. The RuFUS project championed the reuse in-situ of existing concrete foundations, and listed examples where this has been successfully done, but identified the key barriers of suitability, information and liability.

Suitability: obviously the existing foundation must be able to physically accommodate the new superstructure, i.e. have sufficient capacity in the right locations.

Information: to re-use the foundations safely, engineers must know what they are. If the original design calculations and drawings are available this makes the task much easier, as small investigations and checks can be done to verify these, however if not then much larger investigations and testing must be done to ascertain what is there before the foundations can be used with confidence.

Liability: when a new foundation is installed the contractor provides a warranty for it, however with an existing one the original warranty has expired and no designer or contractor is likely to take responsibility for it as they cannot be certain what is there. The client therefore must shoulder the risk or take out a ‘latent defects’ insurance policy to cover any claims related to the foundation.

Case study 4: Adaptable, robotic packaging equipment

Interview with the Procurement Director European Equipment for a fast-moving consumer goods firm conducted by Alexandra C.H. Skelton

Industrial equipment, predominantly made from steel and stainless steel, is purchased for processing, filling, palletising and packaging food and detergent. Reliability, efficiency and price govern purchasing decisions. The requirements of the equipment changes often as the products and packaging are updated frequently. The cost of ownership is assessed over a 10-year period and the equipment must have a

payback period of 5 years and exceed the internal rate of return of 15%. Increasing the durability of equipment beyond this 10-year mark is not a priority and the potential resale value of equipment has no significant influence on purchasing decisions. This is the case despite the fact that packaging equipment is typically used for 5–7 years in house and has an expected life of 20–30 years. Flexible, robotic packaging equipment could be used to adapt to changing product needs. However robotic packers are 2–3 times more expensive than dedicated packers and require forward planning to ensure a second use in order to be cost effective. The company currently favours dedicated packers given the price differential between the two and given uncertainty over future product lines. However the potential to reduce the time-to-market for future products by using adaptable, robotic packers is under consideration.

Case Study 5: Durable infrastructure

Interview with Professor for Construction Engineering, Cambridge University conducted by Muiris C. Moynihan

Most major transport installations, such as train tunnels or motorway bridges, cannot be allowed to reach end-of-life and get replaced like buildings or other products, as the disruption caused would be too severe — they are, in effect, ‘essential’. Therefore they are undergoing constant maintenance to keep them functional, and a growing part of this is condition monitoring to identify problems and determine best solutions before they become critical. Focusing on UK motorway bridges, the next generation of structures can avoid the degradation being seen currently by implementing novel design features.

Condition monitoring involves the attachment of small wired or wireless sensors which detect changes in strain, inclination, displacement, humidity, etc. and feed this information back to a central hub. Analysing this data points towards likely causes of any problem, and can recommend the best method of addressing it, be it repair or replacement of a section. Before this technology, more manual inspections were necessary, causing more disruption, and components were replaced on a scheduled basis where this was impossible, regardless of actual deterioration, or when a problem was not understood and replacement was the only option. The technology is not at a stage where it should be applied everywhere; it is preferable to target a specific problem and place sensors to best quantify it. As well as allowing efficient

maintenance strategies, monitoring also leads to individuals believing maintenance can be postponed until the situation is almost critical, in order to reduce costs.

UK motorway bridges suffer physical failure, most commonly corrosion of rebar. Left untreated this can cause loss of strength and even collapse, while remedies for it are difficult and expensive to implement. While most bridges projects have a ‘design life’ of 120 years, in fact due to poor design and construction some are requiring major interventions after less than 60 years. More considered design and higher quality construction could, at minimal extra cost, increase the lifespan of a structure by a significant portion.

As water ingress leads to corrosion, design strategies to limit this will obviously improve matters. Examples of this would be minimising joints (increasing internal stresses however) or the specification of stainless steel (or other corrosion resistant) rebar at high risk locations, both of which are done in cases where extended warranties are requested by the client. Poor quality construction, where substandard placing of rebar or concrete lead to insufficient cover depths, or where concrete mixes were not to the specification required, has caused the majority of repairs historically. Especially for installations built in the 1960s, a lack of supervision and checking on site led to these errors, and now the structures are showing more defects than bridges built correctly in the 1920s and before. Proper quality assurance procedures and inspections during construction are required to ensure the current generation of bridges do not suffer the same fate.

Case Study 6: Hard-wearing rails, replacing rails & resurfacing tram rails

Interview with Programme Manager, Network Rail conducted by Alexandra C.H. Skelton

Three strategies to extend the life of rails are documented: engineering harder-wearing rails, cascading rails from main lines to branch lines, and a new technology to replenish worn tram rails.

Harder-wearing rails

Replacing and maintaining rail track is an expensive business, not just because of the cost of materials and the logistics of transporting materials and equipment to

and from the work site, but also because of the economic penalty from lost track time when the line is closed. Therefore, increasing the life of rail track and decreasing the frequency of maintenance are important economic and environmental strategies for the rail industry. Heat-treated or non-heat treated premium grade rail, with a higher wear resistance, can be used in the place of conventional rail to extend the service life and reduce the frequency of maintenance. Table D.1 shows that total emissions for the premium grade rails are less than half of the emissions for conventional rail due to significantly reduced maintenance.

	Conventional rail	High performance rail (non-heat treated)	Heat-treated rail
Material production, processing and recycling	Produced as 108 m length rails at Scunthorpe. At its end-of-life, the rail is collected and recycled. The rail has a mass of 60.21 kg/m.	Produced using the same process route as conventional rail but with additional alloying elements. At its end-of-life, the rail is collected and recycled.	Steel produced in Scunthorpe is transported by rail to France for rolling into rail and heat treatment. The rail is then transported by sea back to the UK before welding to length. At its end-of-life, the rail is collected and recycled.
	52 kgCO ₂ /m rail	53 kgCO ₂ /m rail	56 kgCO ₂ /m rail
Maintenance (over a life of 1000 EMGT ^a for a curve of radius less than 2500 m)	Rail grinding is performed every 15 EMGT ^a of traffic for curves with a radius smaller than 2500 m	High performance rail has a wear rate of about 3% of the value for conventional rail	Heat-treated premium rail has a wear rate of less than 1% of the value for conventional rail
	≈ 1 kgCO ₂ /m rail 66 grinding schedules over rail life	≈ 1 kgCO ₂ /m rail 2 grinding schedules over rail life	≈ 1 kgCO ₂ /m rail <1 grinding schedules over rail life
	66 kgCO ₂ /m rail	2 kgCO ₂ /m rail	0 kgCO ₂ /m rail
Total	118 kgCO ₂ /m rail	55 kgCO ₂ /m rail	56 kgCO ₂ /m rail

Table D.1: Embodied carbon calculations for different rail options

Note:

- a.** EMGT is ‘equivalent million gross tonnes’, a measure of the weight carried by the rail accounting for variations in damage caused by different loads.

D. PROLONGING PRODUCT LIFE INTERVIEWS

Another strategy is to increase the proportion of weight worn away during service by increasing the thickness of the rail head. Assuming the wear rate is identical for both conventional and thicker head rail, extending the rail life would save metal by reducing the need for the manufacture of a completely new rail.

In environments where corrosion may significantly reduce the life of a rail, high purity zinc coated rail can be used to extend its life. In one such environment, a crossing made of conventional rail required replacement every 3 to 6 months due to corrosion. By replacing it with high purity zinc coated rail, the crossing has been in service for the last 16 months without needing to be replaced.

Rail Mainline to Branch

Cascading of rails was performed in the UK until recently, and is still practised on German railways. Worn mainline rails undergo non-destructive, ultrasonic testing to establish integrity. Existing welds are then cut, and the remaining lengths welded into long strings. These are resupplied to the network for use on secondary routes. Cost can limit the motivation for this reuse as the rails themselves represent only 7% of track renewal costs. Historically, the rail life was also increased by transposing the rail: provided head wear was not too close to the limit the non-active gauge face was made active by turning through 180 degrees. Other than transport and welding emissions, this prevents emissions associated with the production of branch line rails.

Metal decomposition on tram rail

The cost of replacing grooved tram rail in the UK may be up to 3000 pounds per metre; digging up old and laying new embedded tram rail often necessitates the closure of roads, causing significant disruption to traffic. An alternative strategy to replacing a rail is to extend its life. A submerged arc welding process is used to restore the rail profile by depositing steel onto the worn rail surface. Careful temperature control during processing and alloy choice produces a high integrity weld. Additionally, the deposited steel has a higher carbon content than the original rail, which results in a more wear resistant surface.

Case Study 7: Carbon-fibre aircraft body

Interview with Technical Fellow, Boeing conducted by Muiris C. Moynihan

Boeing's new 787 Dreamliner aircraft makes significant advances in the use of composite materials in the aircraft's main structure. Around 50% of the aircraft by weight is made of composite materials, and most significantly the fuselage is made from a single fabricated piece. This change in design was motivated by the desire for weight savings and resulting fuel savings and has the co-benefit of increasing the life of the aircraft. The Dreamliner is anticipated to have a service life of 30–35 years, compared to 20–25 years for a conventional metal aircraft. This extension is principally due to the elimination of large numbers of connections and fasteners through the use of composite materials. Fasteners such as rivets act as stress concentrators and can also be sources of corrosion, which limit the fatigue life of an aircraft. The use of composites allows much more complicated sections to be made as single pieces, thereby eliminating large numbers of fasteners and extending the life of the aircraft. The smaller number of fasteners also simplifies maintenance checks, where each fastener must be checked for signs of corrosion or cracking.

Case Study 8: Restoring supermarket equipment

Interview with Development Manager, Tesco conducted by Daniel R. Cooper

Tesco has been operating a reuse program for 18 months and has realised great benefits from the process: capital savings on investment in new kit and product lifetime extension. Currently as store closures and refreshes are identified, those stores are surveyed and kit is removed for refurbishment to be placed back into new and existing stores. Items currently removed from stores include mechanical handling equipment, checkouts, and refrigeration units. The greatest challenge to the success of this process has been the perception of kit as 'second hand' and Tesco Design Standards that change frequently to keep the stores and brand contemporary.

Case study 9: Office block refurbishment

Interview with Associate, Expedition Engineering conducted by Muiris C. Moynihan

55 Baker St. is a concrete-framed office block originally built in the 1950s. By the early 21st Century it had become outmoded — its long, narrow corridors and enclosed offices were no longer suitable to the needs of the modern workplace. Because

of its city-centre location, a conventional demolition and rebuild project would have taken too long to meet developers' profitability targets, so instead an ambitious major refurbishment programme was undertaken. This involved stripping the building back to its structure; filling in the many stair and lift voids dispersed throughout the floorplates to create open-plan office and replacing them with a new centralised circulation and stability system; expanding the floorplate into spaces between the 'wings' of the existing building and creating atria; and removing columns at certain locations to improve flows around the final building. The servicing was entirely redone; the low floor-ceiling height of the existing building was overcome by a chilled beam system which gave maximum headroom over most of the floorplate. In all 70% of the original building structure was reused, saving 3,500 t of rebar.

Unusually, the design team had access to extracts of the original design calculations and drawings for the existing building, which greatly aided understanding and justification of what was there and why, hence only limited testing and investigation was required. Even so, some unexpected challenges arose on site which were difficult, but none that could not be surmounted by careful thinking and intelligent detailing.

More generally, two commonly cited ways of increasing a building's adaptability are having longer spans and increasing imposed loading. This enables a wider range of activities to be accommodated and could prevent demolition in future. However, the extra resources used to deliver the longer spans and higher loads should be balanced against the likely benefit from them. It is thought preferable to consider where capacity is most probable to be useful, for example putting extra load allowances towards the back of building where storage is likely, or adding capacity along edges (e.g. atria) where extension is possible. This has been successfully done on high-rise office blocks in London and additions subsequently made, once the user request them, with reduced costs and time.

Case Study 10: Steel mill upgrade

Interview the Jonathon Aylen, senior academic at Manchester Business School conducted by Alexandra C.H. Skelton

80% of the product range rolled on the modern wide strip steel mill has been developed in the last 20 years meaning that 60 year old mills have had to adapt to deliver this more diverse product range (Aylen, 2013). Higher strength steels have put pressure on the power, torque and load limits of mill stands and have been accompanied by demanding quality standards, rising energy costs and the need to

increase output in order to reap economies of scale in competitive markets. Aylen (2013), in a study of 7 strip mills built using Marshall Aid following the second world war, identifies four means by which mills have been upgraded or “stretched” in response to these pressures:

1. improved intensity of hardware use through experience and better maintenance e.g. through better scheduling and condition monitoring;
2. system wide effects of improvements in material feedstock and downstream processing e.g. accepting higher piece weight inputs that allow the production of longer, heavier coil;
3. modular improvements to existing plant e.g. rotating quick roll change rigs that reduce downtime and control systems that predict strip quality and accurately determine the number of passes required;
4. physical reconstruction of existing plant e.g. a switch from a semi-continuous or continuous layout to a $\frac{3}{4}$ continuous layout that allows greater utilization of the finishing train.

As a result of this activity, the average ratio of current installed capacity relative to initial design capacity is found to be 1.8 i.e. the capacity of these mills has close to doubled over their lifetimes to-date. The single outlier is the Linz mill in Austria which has a stretch capacity ratio of 8.3 achieved through over 30 significant performance enhancing modifications and by accepting heavier piece weight inputs. Productivity is not compromised in these upgraded mills relative to newly designed mills; in fact there is some evidence that established mills have an advantage. Aylen (2013) briefly discusses the possibility that mill stretch has been facilitated by initial over-design, e.g. the mill in Linz had a low initial rolling capacity but was contained in an excessively large building allowing the rolling line to increase within the building by just under 40%. In their paper on plate mill upgrade Bhooplapur et al. (2008) point to a second reason why mill upgrade has been possible. Microalloying is the favoured process for making modern high strength plate grades and in this process the greater strength of the steel is exhibited only in the late stages of rolling and cooling, limiting pressure on the mill stand and so allowing high strength steels to be rolled on mill stands that were built before these grades were envisaged.

Case study 11: Restorable washing machine

Interview with Director, ISE appliances conducted by Daniel R. Cooper

The main washing machine sub-assemblies are the housing, drum unit, motors and transmission, and pipes and pumps. Although customer misuse may damage the housing, this is rare. The pipes and pumps will clog over time, but with reasonable maintenance these components should not limit the life of the machine. The drum unit is typically made from plastic (though some more expensive machines are made from stainless steel). The steel bearings in the drum are typically contained within a sealed plastic housing. When these bearings wear and fail, the sealed drum unit must be replaced, as there is no access to the bearings. Drum replacement is expensive, therefore bearing failure typically leads to the washing machine being replaced. The motors used in washing machines are typically carbon brush motors (90% of domestic washing machines use carbon brush motors) contained within a sealed unit. The carbon brushes wear out and the sealed unit again means motor replacement is necessary to prolong the life of the machine. The expense of replacing the motor means that wearing of the carbon brushes often leads to whole machine replacement.

For a washing machine to be inherently long life and easy to repair the design must mitigate the two predominant failures discussed above — wear of the bearings in the drum and the carbon brushes in the motor. More durable bearings would provide inherent long life and an “old-fashioned” split-ring drum design would allow them to be easily replaced when they do fail. As for the motors, these should not be put in a sealed unit, allowing replacement of the carbon brushes if they wear out. Alternatively, more expensive, and longer lasting, induction motors could be used.

Case study 12: Disassembly and component reuse of oil rigs

Interview with Project Director for North West Hutton, Able UK conducted by Alexandra C.H. Skelton

North West Hutton is an oil rig that was built by Amoco in 1981 in order to exploit reserves in the northern most section of the British North Sea. BP inherited the installation through the takeover of Amoco in 1998 and North West Hutton was subsequently owned by a joint venture — 26% BP Amoco, with the remainder held by Shell and others. Despite being designed to process up to 130,000 barrels of oil

per day, the reservoir underperformed. Production peaked in 1987 before hitting the characteristic production cliff with production decreasing by 42% 1988–1989 and successive step decreases (with the odd minor production push) until, in 2003, production ceased altogether (BP, 2005). In 2007 the rig was decommissioned and dismantled by Able UK. Able UK sought to reuse as much of the rig as possible in order to maximise residual value. The accommodation block was refurbished and is now used as the Able UK offices. At the time of interview buyers were being sought for the heli-pad, and sections of the jacket (the legs) of the structure were cut into sections and sold on to be re-rolled into plate that could be reused.