Regularity and optimisation practice in steel structural frames in real design cases

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Abstract

Large amounts of energy and carbon are embodied in the frames of buildings, making efficient structural design a key aspect of reducing the carbon footprint of buildings. Similarly to a previous study which analysed real structures had observed that the unused mass of steel framed building could amount to nearly 46 % of the total mass due to over-specification of the sections, we find a value of 36 %. We observe that this value correlates with the design method, with software-aided design bringing significant improvements and with the design stage, where most of the optimisation seem to occur between the preliminary and tender stage.

We find that neither the regularity of the structure nor the cost, independent of the measure used, correlate with the mean utilisation ratio (UR). Conversely, we observe an apparent reluctance to design beams above a 0.8 capacity UR. This reluctance explains most of the unused mass in buildings. The rest of unused mass consists in cores, trimmers and ties (6 %), some of which bear loads not captured in this analysis but are otherwise necessary for stability reasons, and in edge secondary beams (3 %) which design is constrained, and should not necessarily be considered as 'unused' mass.

Keywords: Efficiency, Design, Steel frames, Optimisation, Design practice, Regularity

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1. Introduction

The efficiency of many technical systems in common use are reaching their theoretical efficiency limits. This is notably the case of buildings which can now be designed to be operationally carbon neutral as they operate (Cotterell and Dadeby 2012). However, the growing needs for construction has an impact through the carbon and energy embodied in the buildings, notably the frames. With the threat of global warming, new objectives (Rhodes 2016) have been established for developed and developing countries for carbon release. Further improvement of the operational performance aspects of new buildings cannot help significantly to reach the targets. There is therefore a pressing need to find new ways to reduce embodied carbon.

This is a particular concern as the embodied carbon in buildings can represent as much as 70 % of the whole life carbon of the building (Dimoudi and Tompa 2008, Nadoushani and Akbarnezhad 2015) for warehouses and sheds, and can still reach 30 % in office buildings. The strategies for the reduction of this embodied carbon are different depending on the material used for the frame: concrete, steel or timber. The choice of material for the building frame depends amongst other considerations on the function of the building and the economic constraints associated with its construction. Lowered carbon footprint of concrete-framed building requires finding new supplementary cementitious materials, as the current production of slag and fly ash is fully exploited, or of insufficient quality (Snellings 2016). In the case of steel-framed buildings, improvements in the energy and carbon efficiency of the steel production process are unlikely as they are already close to their limit (Cullen et al. 2012). In this work, we focus on the design of the structural frame of steel-framed buildings.

A different approach to lowering the carbon footprint of buildings is to improve the structural design. Strategies for efficient design of buildings depends on the choice of the structural system. This is a complicated decision which depends on the capabilities of the design firms, the norms and codes (including seismic), the time allotted, the budget and the preferences of the client. Therefore, although it is not feasible to assess the quality of a design in terms of the fundamental choices made, it is possible to measure how closely the specifics of the design match an ideal, figured by an exact adherence to the code. In this work, therefore, we do not assess the design itself. The codes themselves can affect the absolute efficiency of the design. Modern codes such as the Eurocode define limit states for elements instead of working stresses. This paradigm is much more efficient than the working stress design methods used previously, for example, the change in the Canadian code resulted in structures which were 15 % lighter (Kennedy 1984). The Eurocode, in its latest iteration, is one of the most advanced codes, introducing provisions for plastic design — which is uncommon — but also has small safety factors. Some of the provisions on plastic design were already found in the British Standard. With respect to the safety factors, the reliability of steel elements has been well established over a century of experience and improvements (Byfield 1996). Therefore, the ideal structure following the Eurocode is also quite close to a 'optimal' structure making maximum use of the materials whilst still being extremely safe. Although the design of efficient structural systems, notably using plastic provisions, is a complex topic — portal frame structures are usually very efficient structures — it is possible to study how *optimised* a structure is. For a given topology of beams and columns, with the loads specified, it is possible to establish the lightest elements required to build the structure according to the code. The choice of connexions, whether nominally pinned or moment bearing affects the overall efficiency of the design, *but has no bearing on how optimised it is*. Optimum design according to codes has been studied since computer modelling became possible (Saka 1990).

Despite structures built exactly to the code being safe, the engineers seem to frequently design well within the limits of the code. A previous study by Moynihan and Allwood (Moynihan and Allwood 2014) analysed 79 steel-framed buildings, and the utilisation ratios of all beams and columns were collected. They concluded that 46 %

of the steel mass in beams and columns is not load bearing. They have suggested a number of factors which can explain this: rationalisation, *i.e.* using the same section across the building frame, chosen to match the highest requirements; elements from older buildings designed with pen-and-paper are not optimised because this process would have been too time-consuming; UK universal beams and sections cannot satisfy requirements exactly — nonetheless, many fabricated elements were found to have relatively low utilisation ratios where section properties could be allocated to suit the structural performance. In general, this ground-breaking study both identified a great potential for savings and opened questions relating to the design process which led to this performance gap.

As the Moynihan and Allwood study was the first of its type, we have followed a similar methodology, but with a more detailed analysis of design approach. We collected detailed information on the roles of elements, as well as the limiting factor of the design of each beam, the floor type and the design methodology for each project. The objective was to identify the design practices and goals which explain the UR but with a more detailed analysis of design approach and the underlying causes of the observations.

2. Materials and methods

We have analysed the floor plates (excluding supporting columns) of 30 buildings, 27 'real' at various stages of the design process and 3 'model' buildings found in design handbooks (Table 1). The beams represent about two-thirds of the mass of a typical steel frame. These steel-framed buildings are office/commercial or educational buildings. For each floor design, the details every beam for which we were able to gather sufficient information for was recorded. Their type, length, mass, and connection types were noted. Fabrication details such as the presence of cells in the web or the application of a pre-camber were also noted. Each beam role is also noted as being either a primary, secondary or a core/trimmer/tie. Edge beams are marked as such.

The case studies cover both traditional pen-and-paper (labelled 'None') and computer-

Table 1: Overview of the case studies. Sectors are Commercial (C), Education (E), and Model (M). Floor systems are Trapezoidal (T), Pre-cast Decking (P) and Re-entrant decking (D). All case studies are from the UK.

#		Year	Stage	Storeys & Height		Model	System
1	C	2005	As Built	13	50.0	None	T
2	C	2009	Tender	17	66.0	None	R
3	C	2006	Construction	5	17.5	None	P
4	C	2013	Construction	3	12.0	None	R
5	C	2010	Construction	6	21.8	None	R
6	C	2008	Construction	3	11.0	None	R
7	C	2016	Preliminary	10	45.0	Unknown	T
8	C	2006	Construction	5	23.3	None	T
9	C	2001	Construction	3	11.4	None	T
10	E	2016	As Built	3	11.8	Full Frame	P
11	E	2017	Preliminary	2	8.0	Full Frame	P
12	E	2017	Tender	2	9.0	Full Frame	P
13	E	2012	Construction	3	11.6	Full Frame	T
14	E	2016	Construction	2	7.7	Full Frame	R
15	E	2006	Construction	3	9.3	None	P
16	E	2013	Construction	2	7.6	Full Frame	T
17	E	2005	Construction	3	11.2	None	R
18	E	2013	Tender	5	11.2	None	R
19	E	2016	Construction	2	6.3	Full Frame	T
20	E	2014	Construction	3	12.6	Full Frame	T
21	E	2013	Construction	3	11.6	Full Frame	T
22	E	2014	Construction	2	8.7	None	P
23	E	2016	Tender	3	11.4	Full Frame	T
24	C	2014	Construction	1	5.9	Unknown	T
25	C	2016	Tender	13	54.9	Unknown	R
26	E	2018	Tender	4	17.2	Full Frame	T
27	C	2016	Construction	2	5.7	None	P
28	M	—	—	8	26.8	Floor Plate	T
29	M	—	—	8	26.8	Floor Plate	T
30	M	—	—	8	26.8	Floor Plate	T

aided optimisation (marked ‘Full Frame’) design methods, and different slab forms of construction: pre-cast, and composite metal deck both trapezoidal and re-entrant.

2.1. *Evaluation of the UR in the case studies*

Each floor beam has been recalculated using the CSC Fastrak software(CSC ????) according to the known design loads of the structures. The original digital plans were used when available, otherwise, they were redrawn. The software gives the utilisation ratios according to the bending moment, the deflection, the natural frequency, and the shear forces. The dominating UR of the beam is the largest of these four, which is deemed limiting. Based on this information, it is possible to measure the approximate over-design of each beam and the corresponding mass. It is also possible to relate the dominating UR to geometric and functional information. The role of parameters such as type of decking, design method (computer modelling or pen-and-paper) can then be related to the overall design.

The plans for all the case studies were entered in the software manually. The beams were re-calculated according to the standard which was used at the time, either the British Standard BS-5950 or the Eurocode EC3. However, as most of the design is dominated by bending, deflection or natural frequency, the results presented here are independent of the standard chosen as the formulas used in the British standard and Eurocode for these criteria are identical.

To ensure consistency, the following starting assumptions and restrictions apply:

1. The modelling was restricted to a single floor plate of each building, as opposed to a full frame analysis. Modelling a full frame would require many more assumptions to be made involving wind loading and stability systems, and would take significantly longer. By analysing a single plate only the vertical loads need to be established, which can generally be easily extracted from the design information. Any members determined to be part of the lateral stability system (such as in braced bays) have been omitted from the data collection, as have any members that form part of a portal frame. This decision also enables us to directly compare efficiencies between buildings with different numbers of stories.

2. Whilst gravity loads for the general floor finishes (Super-Imposed Dead/SDL) and the imposed loads were generally easily available, loads for cladding were a lot more difficult to determine in some cases. Where specific loads have been given these have been applied, and for retained façade projects cladding loads have been ignored. In all other cases beams have either been omitted from the data collection, or beams were marked as edge beams.
3. Similarly, any beams that only take load from stairs and lifts have been omitted, or if included marked as core members.
4. Any 'unusual' beams were also omitted from the study. This included any curved members, angle sections or tapered beams. PFC sections were generally omitted if they formed trimmers only, but included where they formed load bearing beams. Hollow sections were only included if it was known that they weren't designed to resist torsion — generally torsion resisting beams were omitted.
5. Transfer beams with incoming point loads were omitted, unless coming from an existing model. This is due to the difficulty in accurately determining the loads imparted onto the beam.

Care was taken to account as much as possible for the constrains which come from the construction stage.

1. Overall frame stability — steel frames are often inherently unstable during construction, until all vertical and horizontal bracing and any diaphragm floors are in place. However this is standard in the UK across all (normal) jobs, and the practice is for the fabricator/erector to provide additional temporary bracing based on their construction sequence. This rarely affects final steel sizes and hence was not considered.
2. Composite beams — Composite beams are unable to achieve their full increased capacity until the concrete has adequately cured. Because of this, they need to be checked for a construction load case, where they are expected to take the weight of

the wet concrete plus a nominal construction live load under their 'plain' section condition. This is a feature built into the Tekla/Fastrak software, and therefore has been taken into account in the analysis.

3. Precast planks — The stability of the beams can be affected depending on the plank installation sequence. Where a beam supports two sets of planks, the centre of mass of each will be offset from the centroid — therefore if the planks are installed entirely along one side before the other you end up with a torsion in the beam that needs to be accounted for. It is impossible to know without having worked on these projects whether this was an issue. However any redesign of beams for the temporary condition would generally be the responsibility of the contractor, and would thus not appear in our analysis.

A key question to evaluate the design is the regularity of design: small buildings with simple shapes can have a very high mean utilisation ratio: in the data set the case study 1 has an mean UR close to 1 with almost no dispersion. It is however an outlier in a number of respects: it is both very small and very simple. Therefore, the optimal design for that building offers no scope for rationalisation trade-offs. In general, a measure of the regularity of each design should be related to its mean UR.

2.2. Regularity measure

A hypothesis for the underutilisation of the elements is that rationalisation induces a mismatch between the constraints and the range of available section profiles. Rationalisation is the use of a reduced set of profiles dimensioned to match the stricter design constraints rather than a more extensive set of profiles, tuned to the full range of constraints. Under this hypothesis, the more regular a building is, the lower the effect of rationalisation: the constraints being effectively similar, the same profile can be optimal for a larger number of beams. The converse, which is that a wide spread of constraints in the structure satisfied by a reduced set of sections results in low UR is obvious.

Therefore, to show that rationalisation could be occurring in the case studies, more complex buildings should be have lower UR, *independently* of the number of sections they use for their section size.

Regularity is a difficult thing to measure. To have a more robust analysis, we have used a number of measures for regularity. The first one (top five measure) was used in the original study by Moynihan and Allwood: the fraction of the total mass taken by the five most common elements. The second (Pseudo-Gini) is an extension of this idea, inspired by the Gini coefficient (Milanovic 1997). Third is the Shannon index which is a measure of diversity rather than regularity: a more diverse profile selection could indicate a less regular building. Finally, we have used a measure of Kolmogorov complexity (Kolmogorov 1968) on simplified descriptions of the design. All these measures describe the relative roles of frequent and rare sections in the structure. They are not direct regularity measures of structure geometry, but rather assume that the distribution of section types reflects the regularity of the design. The Kolmogorov measure comes closest to a real measure of the complexity of the assembly.

There may not be a completely satisfactory measure of the regularity of a design. Nonetheless, if none of the proposed measures correlates with the UR, we can conclude that in all likelihood, regularity and thus rationalisation is not a significant factor in the efficiency of a design.

2.2.1. Top five measure

This measure has the benefit of being simple: it is the fraction of the total mass of a given case study taken by the five most common sections. The disadvantage is that is favours considerably smaller structures built with fewer section types. n is the number of section types:

$$I_5 = \frac{\sum_{i=1}^{5} m_i}{\sum_{i}^{n} m_i} \quad (1)$$

This method also implies that the fabrication process is cheaper when the number of sections is reduced. In turn, this assumes that retooling is expensive. In reality, the operations of large fabricators are heavily automatised and the time needed to produce any section reflects more the complexity of the links and cells which may require human intervention. Retooling operations represent negligible amounts of time: The machines are multi-tool, and beams spend most of their time moving on the floor of the workshop going from post to post, and not being machined. Nonetheless, small savings are possible when purchasing stock steel in bulk, and smaller fabricators are less well equipped. This approach can be extended to be independent of the total number of sections used in a construction.

2.2.2. Pseudo-Gini

The following approach extends the top five measure by replacing the arbitrary cut-off of 5 with a measure of the distribution of mass. A real Gini index measure would use the covariance of the section mass with respect to its rank. The measure proposed here is an approximation of the Gini coefficient: they are both measures of the skewness of cumulative curves related to a linear model.

A perfectly irregular design would have its mass equally distributed among all the beam section it uses, whereas a regular design would have nearly all its mass in only a few sections. Therefore comparing the cumulative mass of the sections with the uniform solution is a measure of the regularity of the design. using m_i the total mass of section type i, and n the number of different sections. This index is computed as:

$$I_G = \sum_{i}^{n} \frac{i}{n} - \frac{m_i}{\sum_{i}^{n} m_i} \tag{2}$$

This measure is not as biased against heavier structures, but it gives a regularity of 0 rather than one for a structure built with a single element type, which is an unexpected behaviour. Indeed, a structure built with a single element type may be either perfectly regular or perfectly irregular depending on how its elements are assembled.

2.2.3. Shannon index

The Shannon index is an information-theoretic measure of diversity. It is used commonly to measure the richness of ecosystems, in this case, it measures the richness of the section selection. Small number of sections representing a large fraction of the total mass of steel may indicate a more rationalised construction. The total mass fraction for each section type i is $\frac{m_i}{\sum m_i}$. The Shannon index of a case study I_S is

$$I_S = -\sum_i^n \frac{m_i}{\sum_i^n m_i} \log(m_i) \tag{3}$$

This is a measure of diversity rather than regularity, and thus is in general larger when the number of section types grows. To use it as a regularity measure, we have renormalised the results:

$$I_S^{\text{renorm}} = 1 - \frac{I_S - \min I_S}{\max I_S - \min I_S} \tag{4}$$

2.2.4. Kolmogorov complexity

The Kolmogorov complexity was introduced as a measure of regularity. It is defined as the size of the smallest programme which can reproduce a dataset, typically encoded in binary. For example, a very simple dataset, containing only '0' repeated a given number of time can be produced by a very small programme, while a very complex dataset requires a much larger programme to generate.

The Kolmogorov complexity was measured by compressing text files containing:

- the case study number
- the type of floor
- their mass
- the steel grade
- the type of section (if UK universal beam or column), else `n/a`

```
29 → Steel Braced → 60.9 → S355 → 457 x 191 x 82 → n/a → 7.5 → Edge
Primary → Pin/Pin → No → No
```

Figure 1: typical line of the file describing the sections. → mark tabulations separating the columns.

- the type of section (if fabricated), else `n/a`
- their length
- their function
- their boundary conditions
- whether they are pre-cambered
- whether they have cells in their web

Taken together, these form a 'bill of materials' which describes the case studies. The files were UTF-8 encoded, and the programme `bzip2` version 1.0.6 was used for the compression. An example line of such a file is given as figure 1. The compressed file sizes were reported in 8 bit bytes using the command '`ls -l`'. This value was used as the Kolmogorov complexity I_K.

The encoding is not perfect, and a binary representation may have been preferable. However, the initial size of the file is small, and a binary encoding may not have left significant possibilities for further compression, reducing the sensitivity of the approach. To use is a a regularity measure, we have renormalised the results:

$$I_K^{\text{renorm}} = 1 - \frac{I_K - \min I_K}{\max I_K - \min I_K} \tag{5}$$

3. Results

3.1. Utilisation ratio overview

A representation of all beams analysed per project and per role is seen on Figure 2. This figure shows the large spread both between and within projects. The distribution of primary and secondary beams depends on the specific layout of each floor. Groups of points extending horizontally usually indicate a single section type used repeatedly in the same configuration.

The secondary beams make up the largest fraction of beams. Assuming a typical rectilinear floor plate this makes sense, as they will be the beam most often used to span over the typical bay width. These beams will often also be the ones designed first by the engineer, as not only do they make up the greatest number of beams by % but they often dictate the typical structural depth of floors. It is reasonable to say that more care will be taken in the design of these beams, and this is reflected by the correlation in the graph.

Conversely, the core/trimmer/tie beams (in grey) will often be the ones least thought about. They are often required to 'fill in the gaps' within the structure, used to tie columns together and frame out slab edges and lift cores, *etc.* It is generally the case that a typical size might be taken for these beams, often a 203 Universal Beam section, as this represents the lowest section size preferred by fabricators. The chart also indicates a correlation between increasing ratios of these member types and a reduced average utilisation ratio. Primary beams generally appear to have little impact on the average utilisation ratio. The lower percentage of these is likely to be a factor, however as they are often the deepest beams within a floor plate it is likely that more detail will have been put into their design.

Edge secondary beams appear to follow a similar pattern to the core/trimmer/tie beams. This is likely down to the fact that the Engineer will utilise identical sections to the general internal secondary beams, which will render these members inefficient due to the reduced loading. It must be noted that often the analysis of these members may not include an accurate assessment of the cladding loading, so the data is slightly less reliable.

The large variation observed is unsurprising as every project is unique, but also highlights the challenges in distinguishing any particular design trend. The 3 model buildings are very different and have been excluded in the following analyses (see for

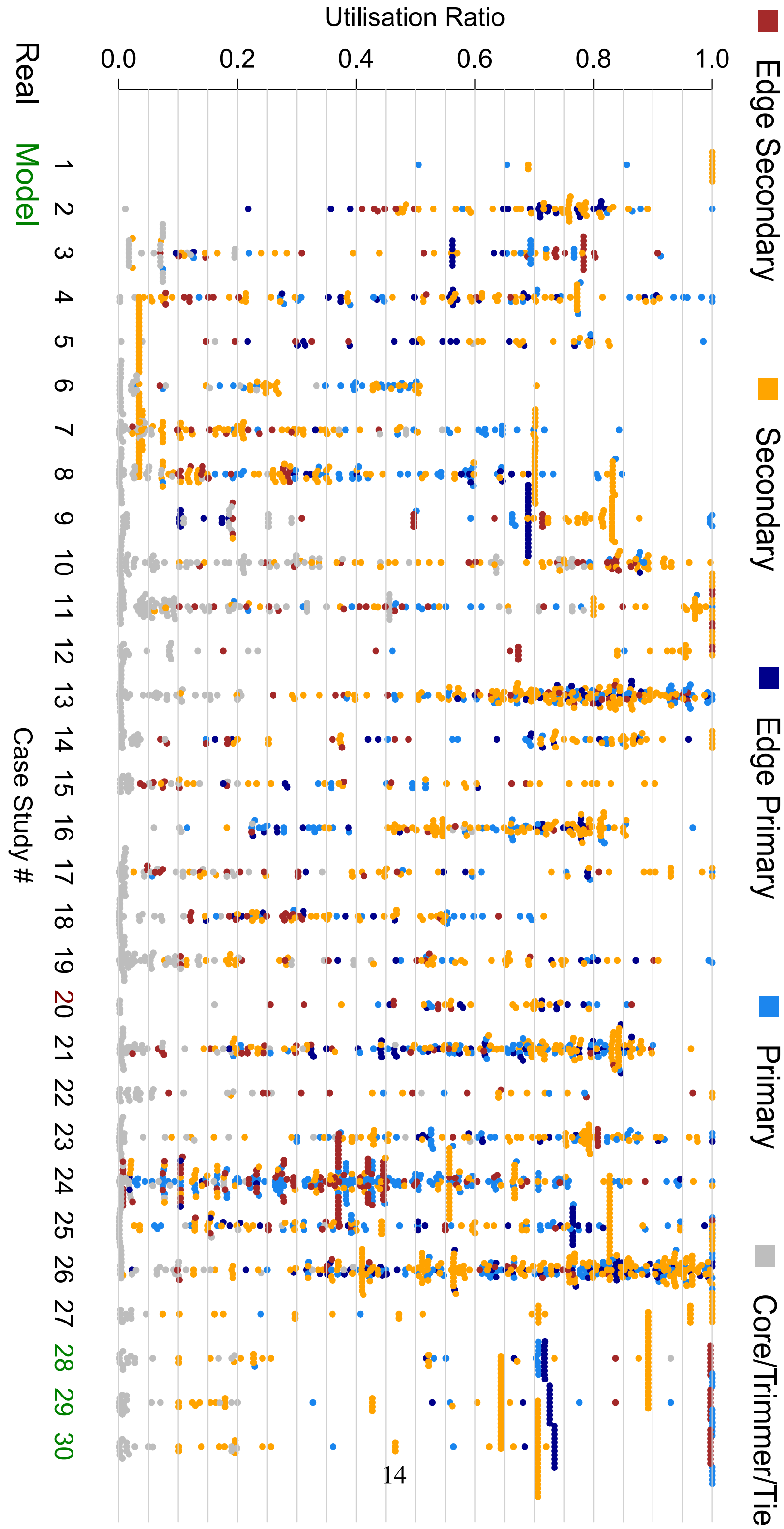

Figure 2: Overview of the projects analysed in the study. Every dot is a beam, and the colours reflect their roles in the designs. This plot illustrates the considerable differences between designs.

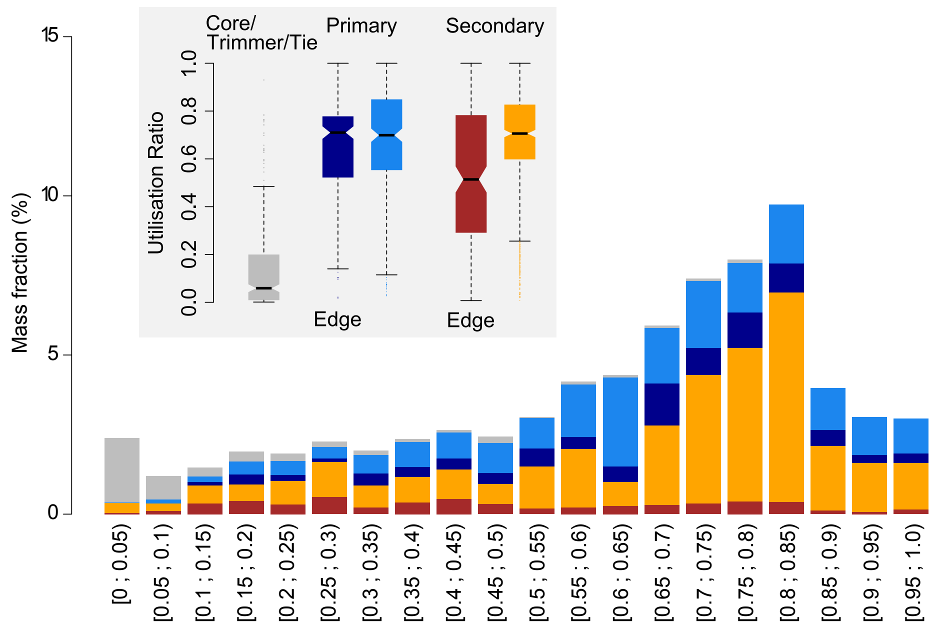

Figure 3: Utilisation ratio as a function of beam type in the studied projects (excluding models). The colours mark the beam types, with the overall distribution of UR as a function of beam type represented as box-plots in the insert. the notch on the box-plots indicates the standard error of the medians. Non-overlapping notches indicate statistically significantly different medians.

example Figure 9: the distribution of UR and beam types is clearly different from real structures).

3.2. Overall design

The overall dataset exhibits a striking distribution of the UR: a peak at very low UR corresponding to the core, trimmers and ties, a main peak at 0.8 with a long tail towards lower UR and a sharp drop-off beyond that point (Figure 3). This profile holds for both primary and secondary beams. However, the peak for primary beams is less sharp. Edge secondary beams have significantly lower utilisation ratio. This is likely because their sizes, notably their depths are prescribed by the links to the façade and therefore they cannot be optimised. Further, as cladding details were not consistently known no allowance for their loads has been applied in this analysis, artificially lowering the UR.

Figure 4 shows the amount of steel m_{unused} which is underused in the structure. This

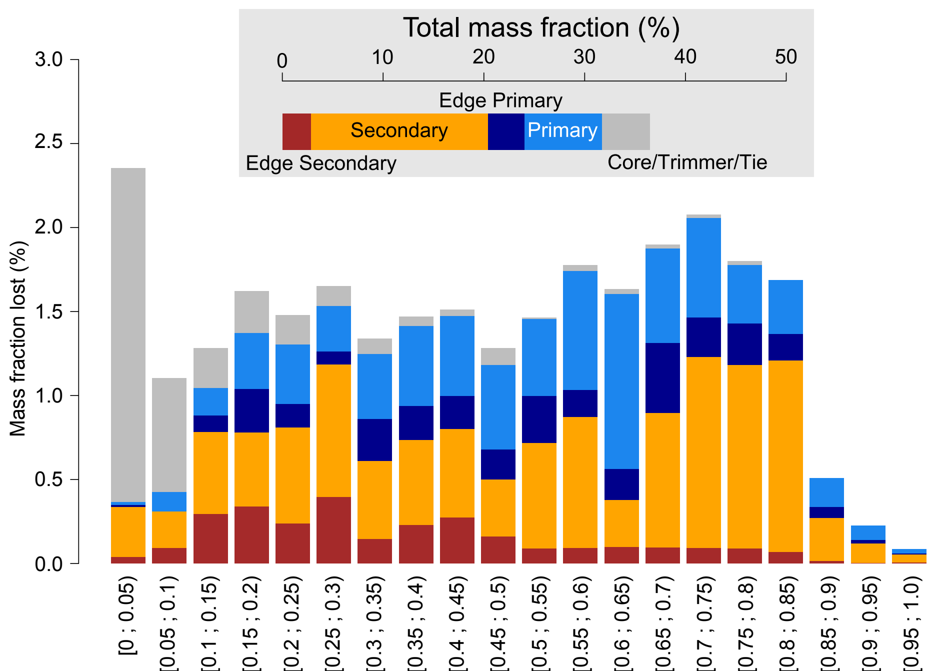

Figure 4: Under-utilisation of the steel mass in the elements analysed in this study. This figure describes how the unused mass is distributed as a function of the role and UR of the elements.

value is obtained for each element of mass m_{element} as:

$$m_{\text{unused}} = m_{\text{element}}(1 - \textsc{ur}) \tag{6}$$

The graph indicates that the steel mass, aside from the cores/trimmer/ties is underutilised fairly uniformly: there are no obvious patterns in underutilisation. Importantly, the large drop after 0.8 is not due to beams in the 0.8-1.0 being very very light, or all very close to UR = 1, but is rather due to the fact that very few beams have UR > 0.8.

3.3. Reproducibility of the results

The key observation is the characteristic distribution of UR across projects.

To verify that this observation was statistically significant, a convergence analysis was performed. All permutations — or when this number was too large, at least 20000 permutations — of all subset sizes of case studies have been analysed for their average UR distribution. This is reported on Figure 5 with, for reference, the theoretical convergence for samples with random UR distribution. If the UR of case studies were randomly distributed, this study should have identified the real mean distribution of UR within 9.6 % using the usual expression for the standard error ϵ_{std}.

$$\epsilon_{\text{std}} = \frac{E}{\sqrt{n}} = \frac{50}{\sqrt{27}} \approx 9.6\% \tag{7}$$

With E the expected value for the difference between two uniformly distributed numbers between 0 and 100 % (50 %) and n the number of samples. In this case, the calculated values are all under the theoretical curve, which indicates that the distribution of UR in all case studies is related to the average UR distribution we report. If this were not the case, we would expect the calculated points to lie on or close to the theoretical line.

The average difference between the weight of any 0.1-wide UR bin in any case study and the average from all case studies is 2.0 % in relative terms. By comparison, the theoretical value would be $\frac{1}{\sqrt{27}} = 9.6$ % if the UR of beams were uniformly distributed. This indicates that UR distributions from case studies are always more similar to the

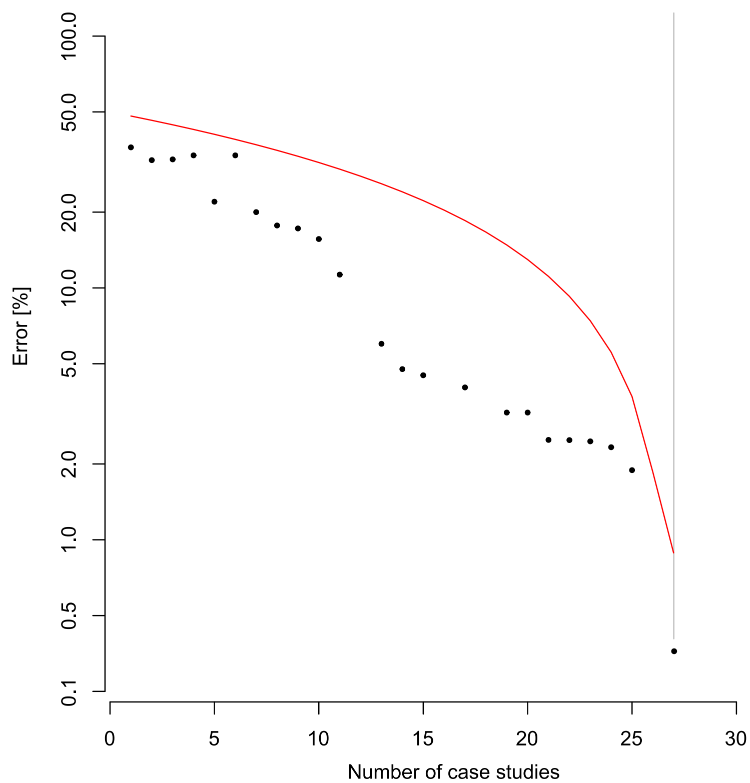

Figure 5: Average norm of the difference between the average UR distribution of a subset of n case studies and the UR distribution of all studies. The red line is the theoretical convergence for random distributions of UR.

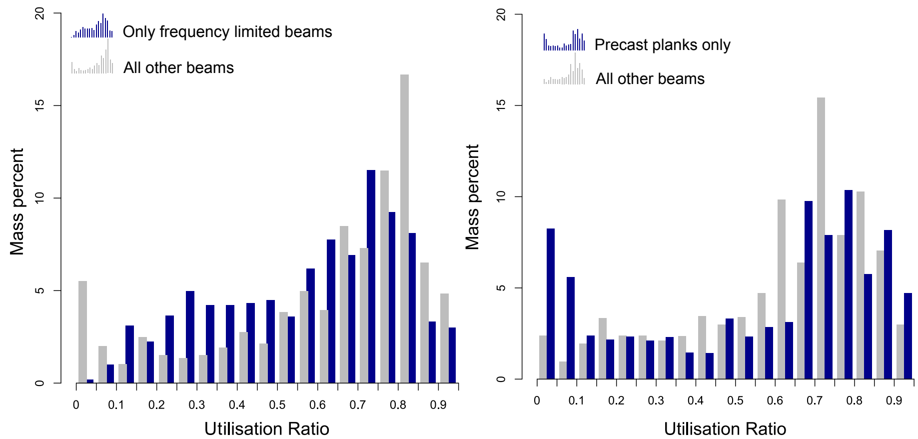

Figure 6: Distribution of the UR where the beams where the limiting factor is natural frequency have been separated (left) and where the precast planks have been separated (right). No significant difference in the distribution of UR is found.

average UR distribution than to a random one.

From this analysis, we can conclude that the global UR distribution we observed is likely representative of the real UR distribution of all steel-framed buildings of similar size and age in the UK. Further, we conclude that the UR distribution in buildings is significantly related to the UR distribution of all buildings[1].

3.4. Limitations of the analysis

The analysis had to make assumptions, as not all the design parameters were known in all cases. This in particular could affect the reliability of the analysis of composite floor plates. A complete natural frequency analysis needs to take into account the connections to the columns, which was not possible in this work. To verify that the results were independent of the floor type – precast or composite – and that the possible errors in frequency analysis do not affect the overall distribution of UR, we present the distributions where beams possibly affected by these issues have been removed (Figure 6).

[1]This result is not trivial: completely uncorrelated random distributions will still converge to an average.

No significant difference in the distribution of UR is found when those possibly confounding factors, floor technology and possibly erroneous calculation of the natural frequency, are controlled for. The higher noise of the distribution is explained by the smaller sample sizes.

3.5. Role of design methods

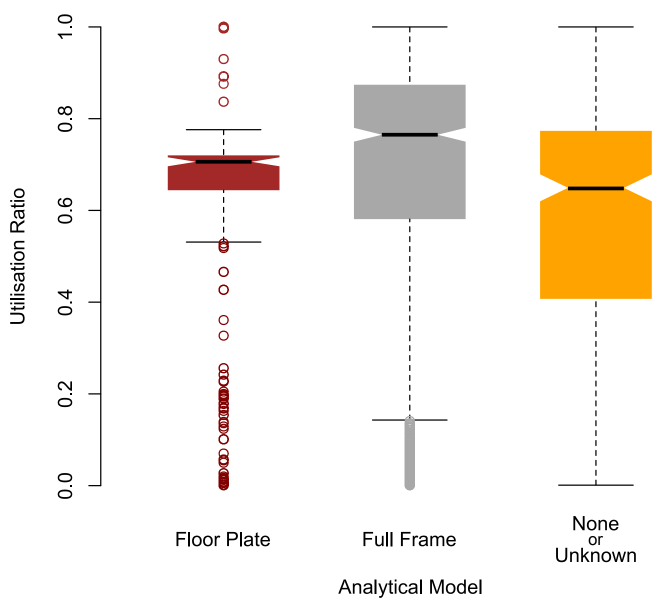

Figure 7: Box-plot of the utilisation rations of the beams analysed for this paper as a function of the choice of model. The 'Floor Plate' only cover model structures. The beams with 'Unknown' analytical model are likely to have model 'None' but this could not be ascertained. The notches mark the standard error of the median: non-overlapping notches indicate statistically significantly different medians.

The analytical models used to choose the beams in each of the case studies were (Figure 7):

Floor plate models were only used for the model buildings. They treat all the beams in the modelled floor as a single unit.

Full frame models take into account the behaviour of the complete frame of the building. They can be used to select the optimal beams.

None is the label describing the beams calculated using pen-and paper models. Without an automated calculation method, it is more labour intensive to choose the optimal beam amongst the choice of UK universal sections.

Unknown describes the beams where we could not be certain which analytical method was used. However, due to the age of the designs, it is very likely that they should be counted in the 'None' category. We found that the average utilisation ratios was the same in the none and unknown cases.

The beam designed without analytical models ('None' and 'Unknown') have a mean UR of 0.64 *versus* 0.76 for the cases studied designed using full frame computer models.

3.6. No relationship between cost and UR

Structures can be very differently priced, and this could be expected to have an impact on the UR, as rationalisation of the section sizes would seem a more attractive proposition when the budget is tight. However, we found no correlation between the price per square metre in the sample and the median UR of the beams. This suggests that the budget does not affect the overall optimisation of the structures (Fig. 8).

The cost of the buildings has not been corrected for their age as we also could find almost no correlation between age and price: the projects are too different and too geographically spread.

3.7. Optimisation process during the design

The case studies cover structures at different stages of the design process. These are 'Preliminary', which are quite rough beam layouts, 'Tender' which are optimised designs produced to gain contracts and 'Construction' which reflects the utilisation ratios of projects sent for fabrication and erection (Figure 9).

The density plots from Figure 9 reflect the distribution of the mass of steel in the floors as a function of their utilisation ratios. These density plots have been generated using the R software using identical smoothing kernels. The model structures have been

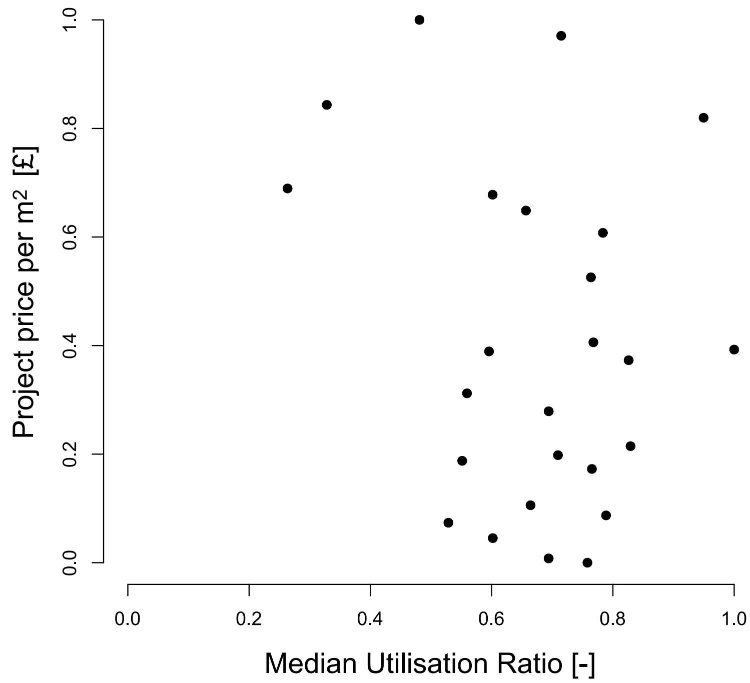

Figure 8: No correlation is found between the price per square metres (normalised between 0 and 1) of the analysed buildings and the UR.

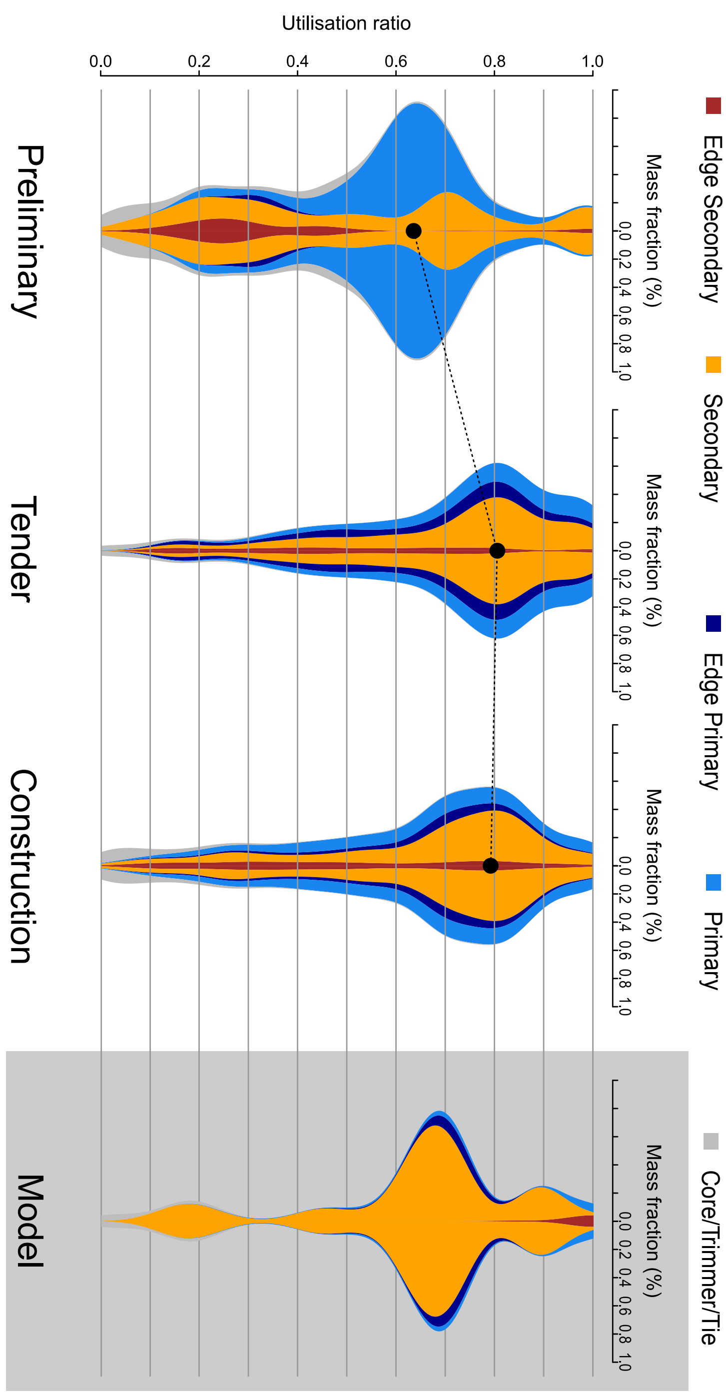

Figure 9: Density plot of the utilisation ratios of the analysed beams as a function of the project stage. The black dots indicate the mode UR.

excluded as they do not reflect real design practice; interestingly, they recall preliminary designs. Cores, trimmers and ties are found at the tail at the low end of the UR distribution, and a peak at UR = 0.8 is observed. In preliminary designs, a large number of low UR load-bearing elements are present, and the mode UR is only 0.64.

3.8. Regularity and efficiency

A key hypothesis put forward to explain the unused mass of frames in the previous study was that designers 'rationalise' their designs, optimising the section which bears the largest loads and using it everywhere else. If this were the case, we should observe that more regular designs where the effect of rationalisation is small to be more efficient.

Such an effect is not visible for any of the regularity measures used (Figure 10). Rather, it would seem that the efficiency of the design (measured by the mean utilisation ratio) is independent of the shape and mass of the building frame.

4. Discussion

The distribution of frame mass according to its utilisation ratio follows a characteristic pattern first observed in (Moynihan and Allwood 2014), with a similar mean UR of 65 % *versus* 55 % in the earlier study. We observe the same pattern in this study, indicating that the selection of case studies is consistent with the previous findings, and that the pattern is a fundamental characteristic of the current design practice (Figure 3). The pattern is independent of whether the beam elements have UK universal beam or column sections, or are fabricated. Therefore, the under-utilisation of the steel cannot be attributed to the usage of less-than-perfect universal sections. Further, the large range of available sections allows UR to be as high as 0.95 in many cases.

Although the the change of a single beam could change the overall behaviour of the structure, this is not a factor which was considered in this analysis: The most important effects concern stability, which would depend on the columns – which were not analysed – and vibration – which was computed on a beam-per-beam basis as discussed above. In

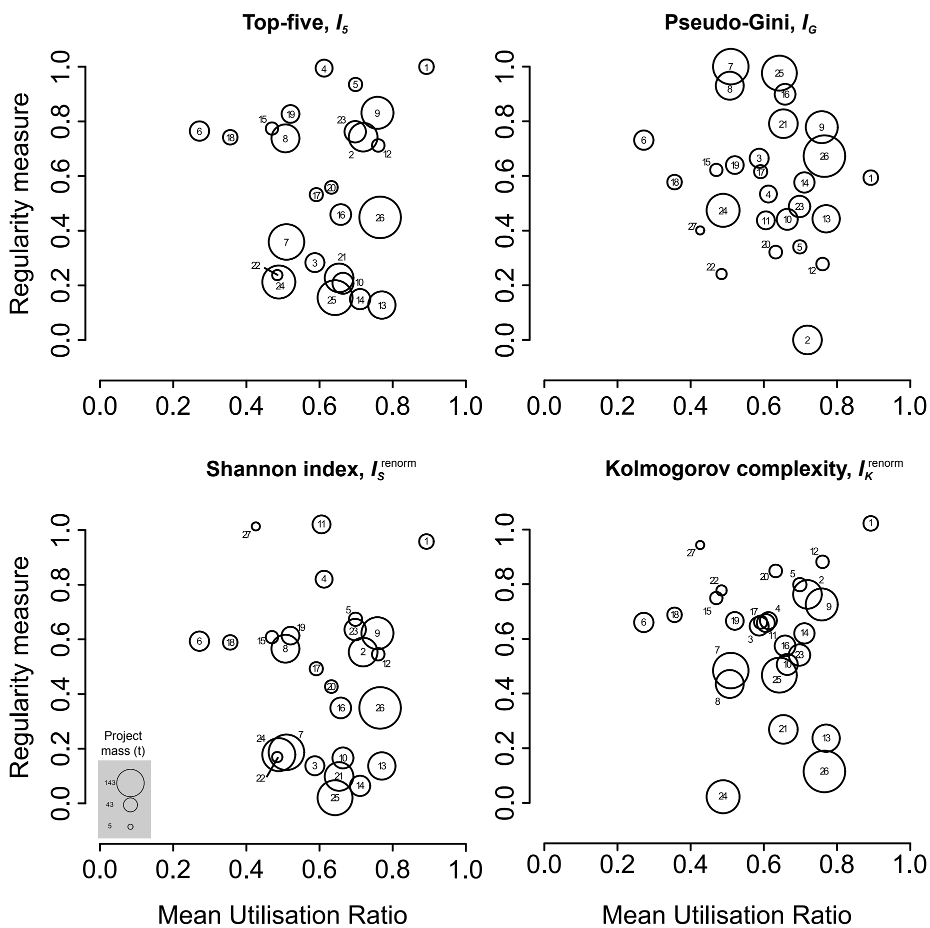

Figure 10: Overview of the regularity of all real projects analysed in the study. Projects are labelled according to their numbers. The area of the circles is proportional to the mass of steel in each case study. The model projects have been excluded. A large commonality is observed between the measures, but none correlate with the utilisation ratio.

practice, real designs are never optimised to the level that changing a single beam could significantly affect the spread of the load on the structure as this would be unsafe.

Not all beams offer the same opportunity for optimisation. Core/trimmer/ties beams have a very low utilisation ratio as they are either not load-bearing elements, but are required for the stability of the structure, or in the case of cores, bear loads not captured in this analysis. They therefore do not represent lost mass in this analysis. Primary beams are less aggressively optimised in general. Primary beams tend to be less optimised because their dimensions can be dictated by the ceiling heights and they sometimes need to accommodate cells to allow for the passage of services. Secondary beams represent the largest potential for improving the optimisation of designs (Figure 4). Whereas, any change in primary beams later in the design process can trigger many further changes, however it is not clear why *secondary* beams could not be more optimised.

Although finding efficient algorithms for optimising the structure itself, *i.e.* the topology of the beams, is still an open question (Kaveh et al. 2012), the optimal choice of beams for a given structure is a solved problem. Indeed, we find that structures designed with modern computer tools have significantly better mean UR (Figure 7) than those designed traditionally. Nonetheless, these remain well below 1. In particular, the 'Full Frame' and 'Floor Plate' models shows that the computer-aided choice of section effectively improves the median UR of the beams to 0.76 from 0.64. As the design time needed to change the beam selection in a computer model of the frame is very small, the UR is also likely a reflection of the goals of the designer and of the optimisation process.

The optimisation process seems to occur predominantly between the preliminary and tender stage (Figure 9) of design. The result of this process is reducing the number of load bearing low UR elements, and in general refining the selection of beams. After the tender stage, most of the design work consists of integrating the services and detailing. It is possible that the utilisation ratio reached at the tender stage are too conservative as the beams do not see their UR further rise as the project goes from tender to construction.

Once a project reaches the 'construction' stage, it can still undergo further changes, but these are not the direct responsibility of structural engineers. Fabricators will design the connections, and in certain cases optimise the design further, selecting different sections than the ones specified by the structural engineers. None of the projects studied has a sufficient scale for this to have been an economically viable option. Therefore, the designs of the projects analysed in this paper were finalised with the sections as designed by the structural engineers.

The regularity analysis did not show any correlation between the complexity of the building, its mass, its cost, the floor technology, and the utilisation ratio of its elements (Figure 10). This indicates that the design strategy leading to the observed utilisation ratio does not depend significantly on the specific building, and must reflect general industry design practices. The hypothesis underlying the notion that rationalisation occurs is that bulk discounts can be had if fewer section types are used. Interviews with fabricators indicated that the bulk discount for using similar sections is small, as operations are highly automatised and fabricators have in general little difficulty to cope with complex orders (private communication).

Collectively, these observations indicate that the underutilisation of steel in the frame does not come from difficulties in the design or rationalisation, but rather reflect defensive design practices by engineers. The strong incentive to design safe buildings is compounded by the need to design defensively to guard against changes in requirements during the design process.

5. Conclusion

Following the study by Moynihan and Allwood, we could confirm the principal finding that about 35-45 % of the steel by mass of the load-bearing frame is not required in terms of structural efficiency. However, only part of this is over-design, as the cores, trimmers, and ties representing 6 % of the total mass are necessary for the stability of structures and are mandated by the codes, and a further 3 % of the mass is underused in secondary edge beams whose design is frequently constrained by the available space. Nonetheless, these beams are still oversized in many cases: in general, the smallest available section should be used. The original study had suggested that rationalisation, was a likely culprit for the overdesign. We could show that this was likely not the case.

The remainder of the underutilisation can be explained by the design practice of the engineers. To guard against changes during the project, the engineers seem very reluctant to design beams with UR beyond 0.8. In effect, this results in *at least* 20 % of the mass of steel frames which is not necessary for the purpose of safety or service. Small changes in the design target could create important material savings at no cost. For this to be practical, one should assess how often the defensive design practice prevented re-designs.

We could establish that computer-aided design improves significantly the UR of structures. pushing the mean value from 0.7 to 0.8, a 15 % improvement. General use of automated design tools in the industry will yield substantial savings in embodied carbon and energy. We also found that secondary beams could in general be more optimised than they are currently.

There is probably an opportunity, before sending the plans to the fabricator, to perform a round of optimisation. If the model structure is already coded in a computer aided design tool, this operation should not be onerous. Nonetheless, there may be little incentive to do this after the tender depending on the form of the tender. Thus, design and build contracts may offer more scope for optimising designs.

Importantly, this study shows that further improvement in the design of steel frames should come from more elaborate strategies, in particular taking into account the design of connections when choosing the sections or designing composite deckings. Such a strategy would allow the selection of thinner sections without otherwise changing the design practice.

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