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Topics of importance openly discussed

Clarifying responsibilities

David Irving writes to commend the latest Business Practice Note on the provision of information to steelwork contractors/fabricators.

A very useful note by Matt Byatt to clarify structural engineers' duties regarding steelwork (BPN No. 17, August 2018). The fact that it is necessary is telling in itself, borne out by the amount of explanation and cross-references to different documents and the various things the structural engineer should do. Too much scope for things to be omitted and go wrong, which is not really acceptable when safety is at stake.

I would advocate clearer, unambiguous design, procurement and checking responsibilities on the structural engineer to close the potential gaps. A UK-wide SER-type [Structural Engineers Registration scheme] process would address this important issue (and not just for steelwork).

The BPN refers to the National Structural Steelwork Specification (NSSS), which does define responsibilities and allocations, but too often these are ignored – as numerous contractual disputes testify. Engineers must not only produce safe designs, they must produce designs that are buildable within commercial constraints. Learning about these constraints is just as important as learning structural theory.

"I WOULD ADVOCATE CLEARER, UNAMBIGUOUS DESIGN, PROCUREMENT AND CHECKING RESPONSIBILITIES"

Eurocodes and safety margins

Alasdair Beal returns to the endlessly interesting topic of safety margins defined by codes.

Alastair Hughes (Viewpoint, June 2018) is undoubtedly 'sailing in the right direction' in his proposals for simplifying the Eurocode load combination rules. I would only quibble on three points.

1) A factor of 1.4 for all dead and live loads means yet more whittling away of the safety margin in normal structures: according to EC0, its safety factors have been set on the basis of comparison to long experience, yet the BS 449 safety factor was 1.7, in BS 5950 it was 1.45–1.55, in the current EC3 it is 1.4–1.45 and Alastair's proposal would reduce this to 1.4. I am uneasy about this 'tiptoeing towards the edge of a cliff': his proposed safety margin is an average of 20% lower than BS 5950 and 43% lower than BS 449. With high-yield steel readily available and serviceability increasingly governing design, there is no point in taking risks with further cuts in safety margins against failure. I suggest that 1.5 would be a more sensible 'standard' load factor.

2) Adopting a single load factor for all loads would get rid of the present silliness where water pressure (a 'variable action') requires a higher safety factor than earth pressure (a 'permanent action'). However, there would still be overdesign for wind load and the EC0 load combination rule would require a signpost subjected only to wind load to be designed to a higher safety factor than a building full of people. I know that, in theory, the design wind load has a 63% chance of occurrence in 50 years, but in practice this is only a three-second gust and its chances of hitting a structure at its most vulnerable angle are probably considerably less. The consequences if a building full of people collapses are far more serious than if a signpost blows over in a storm.

3) Alastair fails to get to grips with the other serious problem in the current EC0 load combination rules: apart from making calculations far too complicated, there is no definition anywhere of what loads may be considered as separate types of 'action' when dividing them into 'primary' and 'secondary' variable actions. In an *ICE Proceedings* paper in May 2010, I presented an example of a beam which supported areas of floor and roof subjected to four different types of loading and asked whether the 'correct' overall load factor for Eurocode design was 1.5 or 1.22. Nobody responded. When I repeated the challenge a few years later in Verulam, the replies were interesting: some respondents thought it was up to the individual engineer's judgement and the rest were evenly divided between those who thought the correct answer was 1.5 and those who thought it was 1.22. None could quote a clause in the Eurocodes which states how the EC0 load combination rule should be applied. If structural engineers cannot agree whether the correct safety factor for a simple beam designed to the Eurocodes is 1.5 or 1.22, this is a serious problem.

One solution would be to provide a table which listed which loads could be considered as separate 'variable actions' for the purposes of EC0 load combination rules. However, a more rational solution would be to go back to the origins of EC0 cl. 6.10, which is based on proposals by Canadian engineer Carl Turkstra in the late 1960s for an approximate method for achieving constant probability of failure if all loads and load factors are probabilistically defined.

However, in practice, as acknowledged by EC0, today's loads and load factors are based on experience, not probability theory. Also, the objective of 'constant probability of failure' makes no sense from an engineering point of view: consequences of failure are much greater for some structures (and some parts of a structure) than others. In these circumstances, applying Turkstra's equation to design calculations is rather like playing 'one song to the tune of another'.

Rather than trying to devise amendments to rescue it, it would be better to abandon it and adopt a more rational and practical approach such as either the table of load factors for different load combinations in traditional limit state codes like in BS 5950 ... or will someone be really daring and propose the system of simple permissible stresses and 'allowable over-stresses' which works so well in BS 449? What's so wrong with the idea of making calculations simple?

It's possible to produce many arguments for and against the way the codes are written and it's possible to cite all sorts of anomalies (including for BS 449). Perhaps we can all agree that there are different interpretations? What we might also all be able to agree on is that the codes are there to deal with uncertainty and to provide a standardised way of design that requires the application of engineering sense. Moreover, to quote Dame Judith Hackitt again, what we don't want is 'a race to the bottom' where rules are 'interpreted' for commercial advantage.

Robert Wodehouse compliments Alastair Hughes on his attempts at making sense of the Eurocodes.

Alastair Hughes is making a good attempt at breathing some sense into the Eurocodes.

Most structures in the UK are six stories or below and do not warrant complicated and involved design codes. Therefore, regarding risk, I would recommend members read the CROSS newsletter, excellently produced by Alastair Soane, and his editorial in the May 2018 issue of *The Structural Engineer*. Figure 1 of Alastair's note sets out the relationship between the safety concerns relating to Construction (38%)/Demolition (1%) + In-Service Requirements (25%) and Design (36%). This shows the overriding risk considerations of the first three items outweigh the Design risk (64% > 36%).

The producers of the Eurocodes have had many years to evaluate the suitable parameters for loads/stresses, etc. such that risk levels can be contained and not exceeded. These were benchmarked to old elastic design codes to maintain similar overall factors of safety. Therefore, the benchmark was benchmarked!

Structures do not fail due to the minor inaccuracy of a partial factor being either 1.35 or 1.4. Structures fail due to gross design errors down to poor checking, incorrect or changed information (not verified), lack of supervision, or indeed gross misdemeanours where, for example, someone adds on an extra storey or additional load without checking the structure.

In this respect, common sense indicates that steel sections or precast concrete sections could have reduced dead load factors, whereas *in situ* concrete partial dead load factors should be considered on merit relating to construction procedure and geometry of the structure.

Obviously, new materials will result in different dead-to-live load ratios and structural sections/geometry, requiring new methods of analysis. In this respect, one has to differentiate between analysis on one hand and risk levels/partial factors, as affected by installation/repair and maintenance, on the other hand. This is where adequate and relevant experience is vital.

The question of codes is one topic guaranteed to raise comment. So, in this case, Verulam will stay silent!

Precast concrete floors

Denis Camilleri and Albert Cauchi write in from Malta with some thoughts on precast concrete flooring following Nick Gorst's articles in the April, May and June issues.

We refer to the recommendations for cuts in planks to be made close to a support and for top cuts to be undertaken alternatively so as to keep clear of cores adjacent to the edge. But what happens when a hollowcore plank contains only four cores? Are the cuts then only made on two adjacent internal cores? When some of the cores are not infilled, does this affect the composite action because the T- or L-section of the composite slab is then partially hollow?

We would like to share Malta's experience in the use of these prestressed hollow planks. Here, such planks are generally utilised as transfer slabs, supporting about four floors of overlying cellular masonry residential construction. Slab spans generally vary from 3.5m up to 8m with end supports on masonry walls 230mm thick.

In these circumstances, it is considered that there is rigid support onto the masonry walling, with bearings varying from a minimum of 75mm for planks less than 350mm thick, up to at least 100mm for thicker planks. These planks are normally supported on continuous concrete padstones. The importance of a designed bearing width is stressed, since the remaining infilled reinforced dimension on the support can then form an integral part of the tied horizontal diaphragm action. This can then be incorporated with vertical ties to achieve the desired structural robustness. Noting the edge infill of these planks bears onto a reinforced concrete padstone which becomes L-shaped, the whole system acts as an encircling concrete tie. Proper detailing of the concrete stitching works to eliminate progressive collapse failures, as per EN 1991-1-7 Annex A.

For this transfer type of slab construction, shear loading is critical. And when high design shear values are required, core infilling provides high shear resistance.

When these hollowcore planks are supported on flexible supports, such as concrete or steel beams, the condition as noted in Figure 5b of Mr Gorst's Part 1 occurs[1]. Here, we believe[2] a reduction in the shear resistance in the region of 40–75% occurs. Any cross-sections with large voids and thin webs are particularly susceptible to strong reduction in shear capacity. Factors enhancing the shear resistance include adding reinforced concrete topping onto the floor and longer filling of the slab end voids. Further, the deflection of supporting beams is to be limited to within span: deflection ratios of 1/800 to 1/1000, or higher.

Could these recommendations justify the working of a flexible support so as now to be considered similar to that of a rigid support?

REFERENCES

► 1) Gorst N. (2018) 'Design of precast concrete floors in steel-framed buildings. Part 1: Slab design', *The Structural Engineer*, 96 (4), pp. 24–28

► 2) Pajari M. and Koukkari H. (1998) 'Shear resistance of PHC slabs supported on beams. I: Tests', *J. Struct. Eng.*, 124 (9), pp. 1050–1061

Fire spread in tower blocks

Melvin Hurst adds his thoughts on the Grenfell Tower tragedy.

I was particularly interested to read Allan Mann's timely article on fire engineering (Special Issue, January 2018). Not only has the Grenfell Tower tragedy concentrated every engineer's mind on the problems of fire, but I have had direct experience of two construction fires while working in the Middle East. Both were probably caused by lax welding safety procedures, leading to formwork being set alight. I was appalled at the damage to hardened concrete resulting from the intense fires. Some concrete was repairable, while other parts had to be demolished and rebuilt.

However, the most pertinent section in Allan's paper, in the light of the Grenfell fire, was that on compartmentation. In the immediate aftermath of the fire, and in the opening stages of the Inquiry currently under way, much has been said about the role of the flammable cladding in contributing to the disaster. But, in an equally illuminating article on how fire spreads, by Dave Parker in *New Civil Engineer*[1], it was spelt out very clearly that the fire which engulfed all four sides of Grenfell Tower could not have spread solely through the cladding. Although he maintains that vertical spreading of fire is inevitable, even if the cladding is completely incombustible, horizontal spread of fire internally or externally is virtually unknown in residential buildings in developed countries (open-plan offices behave very differently). Fire spreads naturally from one compartment (in this case a flat) to those above, although in this case, fanned by the wind, it also spread along one face of the building above the fourth floor.

In order for the fire to have spread to all four faces of Grenfell Tower, there must have been progression inside the building, jumping across corridors by means of inadequate fire doors or poorly sealed services openings. Thus, while attention will rightly be focused on the flammability of external cladding, smoke evacuation, means of escape, fire safety certification and emergency response procedures, equal attention must also be paid to ensuring that such internal fire spread cannot happen – this is what contributed significantly to the truly horrific death toll in this fire.

REFERENCES

► **1) Parker D. (2017) 'Grenfell: Burning questions unanswered', *New Civil Engineer* [Online] Available at: www.newcivilengineer.com/grenfell-burning-questions-unanswered/10022498.article (Accessed: August 2018)**

The Inquiry remains ongoing and hopefully a very full appraisal of the causes and mechanism of fire spread will be forthcoming. What has already emerged is what emerges from every tragedy: there is generally no single cause, no single error to explain what happened, but rather a mix and that mix always includes human error.

And finally... (July)

Our readers continue to enjoy the 'And finally...' brainteaser series, as Nikos Zarkadoulas writes. But Nikos adds his own thoughts about the shear teaser published in July and comes up with a different answer – linked to assumptions.

With no intent to undermine the rationale described in the interesting answer given, I would merely like to add my point of view. The first thing I was expecting to see in the question was the assertion of the rigidity of the horizontal beam-podium, which was indeed introduced in the description as being 'a rigid podium at first-floor level', which joined together both cores, which in turn were of the same cross-section.

However, no explicit reference was made to the nature of the podium end conditions, nor were any pins introduced into the scheme. Those omissions do not contradict the assumption that the podium beam itself could be infinitely rigid, both axially and in bending $(A, I = \infty)$, so if that podium beam were end fixed, it would restrain the two, side, core end joint rotations completely. More often than not, this is a realistic modelling of how thick slabs do interconnect shear walls/cores during horizontal loading distribution, at least in earthquake-prone areas.

Taking these assumptions into consideration, the podium beam does not deform; hence, the end sway displacements are both equal and, since the doubly curved columns ground to first floor are of the same stiffness, what the podium essentially does is to 'collect' all horizontal loads from both cores and distribute them equally.

Based on this end fixity, the shear force is then $V_b = (7w + 9w)/2 = 8w$ for each core in the ground to first-floor level (i.e. it differs from the answer given), with the textual caveat of '... joined together by a rigid podium' (one can make the tenable deduction that 'joined' here stands for both axially and in bending). In that case, what the podium enjoys is a constant tensile axial force $N_p = 1w$ along with a linear distribution of bending moments and a constant shear force.

And this is the beauty of indeterminate structures. The first paragraph of the answer states that: 'In reality, the base shear in the shorter block is much higher', but I beg to differ, since the answer is so dependent on the definition of what 'reality' is when it comes to the structural analysis of indeterminate structures. Force distribution is always predominantly governed by relative stiffness followed by equilibrium, and the relative stiffness cannot always be precisely defined in buildings, especially after crack formation and propagation following large horizontal loading events.

Well, Nikos has his point of view. For now, Verulam will refrain from comment, but no doubt other readers will have their own views, which are, as ever, welcome.

AND FINALLY...

Answer to September's question

C) 44mm

Working:

Deflection δ in a simply supported beam of length L with point load P at midspan, modulus of elasticity E and second moment of area I

$$\delta = \frac{PL^3}{48EI}$$

Take out a constant $Q = \dfrac{L^3}{48E}$

I_0 = the second moment of area of the original beam

The second moment of area of the additional piece is

$$\frac{I_0}{8}$$

The second moment of area of the final composite beam is

$$\frac{27I_0}{8}$$

The force needed to jack up the original beam is

$$\frac{(\delta+100)I_0}{Q}$$

The force needed to jack up the additional piece is

$$\frac{\delta I_0}{8Q}$$

So the total jack force is

$$\frac{(\delta+100)I_0}{Q} + \frac{\delta I_0}{8Q} = \frac{\left(\frac{9\delta}{8}+100\right)I_0}{Q}$$

Applied to the composite beam, this force produces a downwards deflection of δ

$$\frac{27\delta I_0}{8Q} = \frac{\left(\frac{9\delta}{8}+100\right)I_0}{Q}$$

$$\frac{27\delta}{8} = \left(\frac{9\delta}{8}+100\right)$$

$$\frac{18\delta}{8} = 100$$

$$\delta = 44\text{mm}$$