certainly the movement of the digger from which the video was taken, but there must be more to it than that. The broken face was dirty in a way that shows long-standing cracks through both the wall and the arch below. Much of that cracking runs down through the thickness of the wall and close to the edge of the arch. How could that develop?

Can I invite readers to use Google Maps to view Station Road in Brixton and consider how much more serious this incident could have been? Unless we understand the root cause, that danger is ever present. Pressure on the engineers to get the railway back open does not help in gathering facts and considering the implications. The assumption among clients that engineering is just a matter of turning handles is calculated to lead to disasters.

Competence is all too often a will-o'-the-wisp. The most important thing is to know when you are venturing into unmapped territory and tread with caution.

But do not ignore what the structure is telling you.

Readers should certainly look at the Nine Elms wall failure and they might see it as an example of the lack of structural safety referred to in an earlier letter. It's a failure example that would be an object lesson such as those Mr Webster referred to in his article in July.

As alluded to by Bill, it also highlights that: a) 'safety' is not an absolute value, the amount of safety we need is linked to 'consequences': the higher the consequences, the more 'safety' we need; b) 'safety' is not permanent, it can degrade with time; and c) we all need skills in understanding causes of degradation and skills in inspecting aging infrastructure.

CP114 and partial safety factors

MALCOLM MONCRIEFF

In response to Ishaq Ebrahim's letter in the July issue, I grew up with CP114. It was a simple code for simpler times. Very many safe and robust buildings have been built based upon designs using this code. Many of those buildings will still be performing well today. The pressure for longer spans and taller buildings and for lighter and more flexible structures led to problems arising from greater deflections, vibrations and temperature movements. The result was the introduction of CP110 in November 1972. This code introduced the change to partial safety factors.

Partial safety factors have drawbacks, but the notorious collapse of the Ferrybridge cooling towers seven years earlier made it difficult to argue against them at the time. In CP114, the whole safety factor was applied to permissible material stresses and none to design loads. It so happened that the deadweight of the Ferrybridge towers was just sufficient to prevent vertical tensile forces developing in the shells when the overturning action of the design wind force was applied. As a result, the towers contained only nominal vertical reinforcement. It could be shown that a small increase in the design wind speed would result in failure of this reinforcement.

Whether or not this was the actual cause of the collapse can still be debated, but attention had been drawn to a flaw in the underlying principles of CP114, albeit one that would occur rarely in practice. CP114 could have been revised to deal with this special case. However, the decision was taken to move to the more academically acceptable partial load factors and, for better or worse, the profession has had to deal with them ever since.

We have had letters on the subject of structural safety and its complexity. One attribute of safety is 'insensitivity' to assumptions. All of our criteria are approximations and the stability of any structure should not be adversely affected by small changes in any variable. Changes in wind loads are problematic since forces are proportional to v^2.

The Ferrybridge collapses provided another lesson in sensitivity. The shells had a single layer of rebar which gave tensile resistance but no wall bending resistance. Thus, the wall strength was sensitive to an assumption that no bending would arise, which in turn depended on the walls being cast to exact profiles, a quality that was not achieved in practice.

Brickwork loading

DENIS CAMILLERI

I would like to respond to Nick Kramer's letter in the June issue on the origin of certain brickwork stresses.

In a similar vein, this has also intrigued me for more than 40 years. I have also been through CP111, BS 5628 and now Eurocode 6.

It would be interesting to compare the stresses quoted by Nick Kramer, with Malta's design data for a semi-compact limestone masonry of crushing strength 20N/mm^2, applied with a low general-purpose mortar strength of 2N/mm^2.

The 0.43N/mm^2 stress quoted, probably elastic for brickwork, is to be compared with Malta's data on ultimate stresses of 5.12N/mm^2 (EC6 stress 2.33N/mm^2/BS 2.4N/mm^2, on dividing by the relative material safety factors) for a 230mm thick masonry unit being 265mm deep.

Having delved into the office files of the late '70s, early '80s, it is noted that the elastic masonry stress adopted as per CP111 stood at 1.77N/mm^2.

As a comparison with the ultimate codes, the elastic stress as loaded with an averaged load factor of safety of 1.4 and a material factor of safety of 2.2 produces an ultimate stress of:

1.77N/mm^2 × 1.4 × 2.2 = 5.45N/mm^2 (compared with the above 5.12N/mm^2).

Prior to the adoption of CP111 in Malta, the masonry wall strength had been based on longstanding empirical practice within the profession at an elastic strength of 10.5ton/ft^2 (1N/mm^2). This is noted as being below the value adopted by CP111 at 1.77N/mm^2. As noted above, locally there is an insignificant difference between the loadbearing capacity of loadbearing walls designed to EC6 or BS 5628.

There exist cellular residential masonry buildings in Valletta exceeding a life of 100 years, eight storeys high, which do not exhibit structural distress, while conforming to the stresses quoted above.

In the case of local bearing stresses, the above ultimate stresses are increased by 50%, as highlighted in both the BS and EC masonry codes.

Unsurprisingly, the laws of physics and of what is safe don't vary from country to country, so values utilised always provide for useful comparison, especially if they have stood the test of time.

Cost of PI insurance

MARK DUCKETT

We recently renewed our professional indemnity (PI) insurance and note that the costs have skyrocketed since last year, increasing by close to 250%, after a significant increase the previous year. Historically, we have been asked by a select few of our clients to provide PI cover to the sum of £10M or we would risk losing work with them in the future. We have therefore been obliged to increase our PI accordingly.

The cost to achieve this level of PI now is very close to being commercially uneconomical for us to continue in this vein. In effect, the cost amounts to taking on two new engineers for no return.

We are aware that others in all sectors of the construction industry are in the very same boat and understand that some are even considering how to carry on practising faced with the costs now involved.

I am aware that most warranty documents and appointment documents are worded in a fashion that one must maintain the PI cover for the period required provided that such insurance continues to be available on commercially reasonable terms.

My question is twofold:

1) Have others found themselves in this situation this year or previously, and how have they dealt with the issue?
2) When is it really considered that insurance is available but the terms are commercially unreasonable? Is there a figure in terms of percentage of turnover, for example, where you can safely argue that it is no longer viable?

I would be interested to hear of the experiences of others.

Mark raises a topic that affects all of us in practice and we are aware that much disquiet has been expressed elsewhere on PI cost increases. It's important we understand what is driving these increases and thereafter what we can do about them. Verulam is asking for readers' feedback and the topic will additionally be raised with our Business Practice Committee.