Queries, comments, correspondence, and curiosities . . .

Rubber isolators

Messrs Z Rigbi and J Atlasovitz have written to me from Haifa, Israel, to relate an exercise which provides a useful performance indicator on rubber isolators.

In January 1996, we reported[1] on the design and installation of a series of rubber isolators based on carbon black loaded chloroprene rubber (CR). Each isolator was so designed as to compress between 2 and 2.5mm under the load imposed by the specific column. Over the 19 years that these isolators were in place they behaved well and accepted additional shear strains which were not part of the original specifications.

The engineer for whom these isolators were built was concerned that they might have deteriorated over the years and wished to have a reasonable estimate of their ultimate life. They are actually installed in an ideal ambience – a tunnel free from sunlight or ozone sources and at a fairly constant temperature of approximately 16°C. To satisfy his request, part of the structure was raised by a series of hydraulic jacks and one of the isolators was removed for further study. A substitute isolator of the same design and specification was placed in position and the structure lowered down on it.

The isolator, which had been removed, was taken to a laboratory and the layers separated by means of a reciprocating metal saw, using an oil in water emulsion as a lubricant.

As results show, it was apparent that the CR compound had been slightly softened by the emulsion, presumably both as a result of the absorption of water

Due to the presence of zinc chloride, which is a by-product of the reaction of the zinc oxide curative with the chlorine split off from the rubber, and from the oil taken up by the poly(chloroprene) base itself. The results obtained are given in Table 1.

Although the rubber softened substantially, apparently as a result of the absorption of water and oil in the lubricating medium, its strength has surprisingly increased over the years.

Table 1: Changes in mech props

Mech Prop	Original	Removed from isolator
Hardness Shore A	72	55
Tensile Strength MPa	12.76	17.2
Elongation %	220	395

It can be expected to give many years of additional service. This conclusion is similar to that obtained for isolators in other applications.

1. *The Structural Engineer*, **74/2**, p 29-30, Jan. 1996.

This feedback information is, I suggest, useful to the construction industry. I would be pleased to collate through this column information on retrospective testing of materials

Crack diagnosis

Denis Camilleri, from whom Verulam has received letters in the past, writes from Malta with reference to Roger Johnson's technical note in the Journal of 15 October 2002 and seeks guidance:

An innovative crack classification has been suggested. The three classifications include aesthetic, serviceability and stability, with a decision matrix compiled, depending on whether the crack is static, cyclic or progressive.

However, no allowable crack width values have been inserted into the matrix, which would help towards reaching a decision. Referring to IStructE guide *Subsidence of low-rise buildings* and ICE guide *Has your house got cracks?*, the following guidance is given.

Crack widths below 1mm are definitely aesthetic, but could possibly reach 5mm without affecting serviceability. It appears that if a crack cannot be penetrated by a £1 coin (3mm thick), then it may safely be classified as aesthetic.

Crack widths between 5mm and 15mm would cause serviceability problems, but cracks above 15mm would cause stability problems.

Returning to the decision matrix, if an aesthetic crack becomes progressive, would it still be termed aesthetic? What would be the allowable crack width for an aesthetic progressive crack?

The length for monitoring the cracks was not specified. Should the minimum period relate to the taking of measurements in the summer season followed by the winter season or vice-versa? This should give an indication whether a cyclic, progressive or static movement is occurring, with a preliminary decision taken on whether movement is due to settlement or subsidence amongst other causes.

The table in BRE Digest 251 can be very useful in relating crack widths to degree of damage, but I expect that readers will have their own methods of making assessments of cracks in buildings and formulating the answers to the queries raised by Mr Camilleri.

Reduced pullout forces for highly stressed flanges

Martin Double of Ewell, Surrey corresponded directly with Henry Dalton regarding his letter in this column in the Issue of 1st April 2003 and has kindly forwarded a copy of his letter sent to Mr Dalton which I am pleased to include:

I was interested to read your contribution to Verulam, it's a pity that your whole contribution was not printed as some detail may be misunderstood as a consequence. I would be grateful if you could provide more detail, particularly as this is a hobby-horse of mine.

My own thoughts, on the matter discussed, and associated matters, are as follows.

Some aspects that may cause additional stress, or stress concentration, but are often ignored (or not understood!):

- As you rightly point out the stress due to flange bending will be increased due to direct stress in a member resulting from both axial load and bending.
- Shear stress in the member as a whole, due to global forces, and in the flange, due to local bolt forces, may also reduce the flange bending capacity.
- 'Weak Yield Lines', my own term to describe those yield lines that form long after initial yield has occurred. The question is should these yield lines be allowed to contribute to connection capacity when they may never occur before the structure as a whole is grossly distorted and near collapse? For example, the yield lines in a flange that run parallel to the web are generally the first to develop, whereas those perpendicular to the web may not develop until the initial yield lines have been strained 10-20 times or more beyond yield point.
- An added complication to the last point made, and may be a complication to your own studies, is that yield line patterns extend generally through at least 90° and sometimes to 360°. The 'strong & weak lines' will vary in relation to their angular position. The manner in which global stresses are added to local stresses will therefore vary. i.e. global stresses may be directly added to some local stresses while local stresses that are perpendicular to global stresses will not be added in the same way. Is this considered in your graphs?
- Many of the yield lines in popular use do not appear to be correct in that they are clearly not derived from the equations for 'minimum work done'.

Some aspects that may be beneficial and offset the above (or not) are:

- Local stresses have traditionally been allowed a higher permissible stress (capacity now!) But only if the local stress can redistribute quickly into a larger area of section that will not be stressed more than permissible. A point often forgotten.
- For single-storey buildings the axial loads are generally only about 10% of capacity or less, and bending generally governs member sizing. At the position of the tension bolts, in a moment connection to a single storey building, the column moment is close to zero. Such a connection will therefore not suffer from the effects of stress addition that you describe, but may be

refer to references at end of attached 'No ESCAPE from CDM 13*' for more details.

I would urge all engineers to take the CDM regulations to heart, they are there to protect people's lives and stop us as an industry killing on average two people a week. To this end I hope to see the Institution do more to educate its members regarding their responsibilities under the CDM regulations.

**The attachment contains an extract of the HSE regulations document which for copyright reasons we cannot publish. As reference is made to this text in the attachment it would not make sense to include the rest of the attachment text. We do however attach the references mentioned plus a web site, for those wishing to read further, below.*

- *Website: http://www.hse.gov.uk/pubns/cis41.pdf*
- *Construction (Design and Management) Regulations 19 9 4 SI 1994 No 3140 HMSO 1995 ISBN 0 11 043845 0*
- *Construction (Design and Management) (Amendment) Regulations 2000 SI 2000/2380 Stationery Office 2000 ISBN 0 11 099804 9*
- *Managing health and safety in construction: Construction (Design and Management) Regulations 1994: Approved Code of Practice and guidance HSG224 HSE Books 2001 ISBN 0 7176 2139 1.*

I think the answer to the malaise, if that is what it is, is that CDM is not perceived as engineering* per se *but more of a chore or inconvenience. Nevertheless, even if more paperwork is involved, adherence to the principles of CDM should lead to a healthier set of site safety statistics.

[Ed – Readers can, of course, obtain copies of the regulations mentioned from HSE.]

Front cover of the journal

Clive Shearer has written to me from Washington, USA to say:

In response to Simon Pole's objection to the Journal covers, I add the following thoughts.

Having advertising on the cover does not bother me, although the images can be rather mundane. The warehouse that graces the 17 June cover is a case in point. However, the reality is that:
1. Probably more than 80% of the buildings designed by structural engineers are rather prosaic, so this represents reality.
2. It provides income to a not-for-profit organisation. I do agree that the 'image' of the Institution is not necessarily enhanced by these photos, but then how many architects, contractors, non-structural engineers, and general members of the public see the Journal? Probably very few indeed. So, in short, it matters not. Mr Pole's assertion that only 1 in 4 members peruse the Journal makes me wonder about the source of his statistics. I read with a chuckle his further suggestion that 'too many people are put off from tearing open the shrink wrap by a rather poor first impression'. Mr Pole, please let us know how you get these fascinating tid-bits of information about the habits of the genus '*structuralis engineerus*'. I have no statistics or field observations to back up my belief, but I feel sure that members are able to see beyond a cover. Members who want to read the Journal will read it. Members who choose not to read it will not read it. The cover is incidental.

As a further suggestion to popularise the Journal, why not have a spot, say half a page each month, dedicated to members' photos? One per month. They could be sent in as a print or as a jpeg e-mail attachment. I am sure many members would be delighted to submit their own shots of their latest structure. The Editor might be the judge, and the prize simply the honour of having one's building published without going through the onerous article publication process. Perhaps the best submittal for the year could be voted upon by members with a prize of a tie or other token.

Mr Shearer's idea is a good one, but it is one that IStructE already promotes.

[Ed. Firms that are aware of the value of publicity already send us interesting images of their schemes as press releases which we endeavour to use in our news and p&s pages.]

Crack diagnosis

Roger Johnson of Bristol replies to Denis Camilleri whose letter was published in the Journal of 3 June 2003.

I would like to thank Denis Camilleri for his letter which appeared in the 3 June edition of *The Structural Engineer* regarding the technical note: 'The significance of cracks in low-rise buildings'.

He should appreciate that the technical note is a summary of the half-day course held at the Institution on the subject of 'Crack diagnosis in low-rise buildings'. This course has taken place at the Institution on four occasions and will be repeated in March 2004. It was not possible to include all the material covered in the course in the technical note. To do so would be the equivalent to writing a book!

In any event I will try to answer his questions:

There are number of documents with tables of crack widths relating to repair (not diagnosis). BRE 251 '*Assessment of damage in low-rise buildings*', IStructE '*Subsidence of low-rise buildings*' etc. as well as the ICE guide mentioned in his letter. These documents are mentioned during the course. Whether the crack is significant or not depends on the type of materials, the type of building etc. What is important is to obtain an understanding of how the building is behaving, which is undertaken during the initial inspection and data gathering period. I have personally come across many instances of professionals looking at a crack, measuring its width, referring to a table (BRE 251 for example) and then pronouncing whether the crack is significant or not without assessing whether the crack is affecting serviceability (letting in water, affecting insulating properties etc) or monitoring to check whether the crack width changes are cyclic or progressive.

With regard to the decision matrix, of course if at any point in time an aesthetic crack is found to be progressive, then it is reasonable to conclude that if left unchecked the crack may eventually cause serviceability damage. The important point is that the significance of an aesthetic crack found to be 'progressive' will be treated very differently to an aesthetic crack which is 'static' even though the crack width may be the same when first inspected and measured. A crack is aesthetic if it is not affecting the functioning or the serviceability of the building.

With regard to monitoring period, again, during the initial inspection, a hypothesis on possible causes can be developed and the monitoring will contribute to confirming whether the initial hypothesis is correct. It is not possible to be specific on an appropriate monitoring period for all cases. Each case has to be viewed on its merits. Many cracks are not caused by foundation subsidence or settlement. What is important is to start monitoring as soon as possible, to maximise the time available.

I do hope this has gone some way to answering your comments. If he is able to attend the course in March 2004, then I will be pleased to see him, although I will imagine that this may be difficult if he is living in Malta!

A useful summary from Mr Johnson for those involved in this aspect of the structural engineer's work.

Manual for the design of rc structures

Mr Wickramaratna contacts me from Sri Lanka in relation to a clause in the 2nd edition of the above publication and comments as follows:

I refer to *Manual for the design of reinforced concrete building structures – 2nd Edition*, July 2002.

Section 4.10.5.2 – Axially loaded reinforced pad footings
a) Item No.1 – refers to 'ratio of the overall depth "h" to the projection from the column face "a", given in Table 39' – but table 39 gives d/a, d being effective depth of base.
b) Again in Item No.1 – 'effective depth "d" should not in any case be less than 300mm – but the earlier version of the manual refers to "h" not less than 300mm.
c) The steel percentages given in table 39, is it related to 'd' or 'h'?

Please could you verify?

With only a copy of the 1st edition to hand, I would say that the reference to 'h' being not less than 300mm should read 'd' not less than 300mm in order to provide sufficient depth for the column bar anchorage length. It is usual to write depth ratios in terms of the effective depth but steel reinforcement percentages are usually written in terms of the overall depth. If I am wrong in this instance I shall pass on the correct version in a future issue.

Emails can be sent to Verulam via: reynolds@istructe.org.uk

Letters should be kept as short as possible, and preferably clearly typed. Illustrations cannot be redrawn: please ensure they are suitable for publication.