Verulam

Queries, comments, correspondence, and curiosities

A happy New Year to you all! The Christmas holiday has proved inspirational to a number of readers...

Deflection of portals

Mr D. H. Camilleri has written from Malta and is concerned about the vertical deflection of portal frames. He writes:

I have often wondered regarding the validity of the ridge deflection d_{RE} quoted in section 11.5 of *Manual for the design of steelwork building structures* given as:

$d_{RE} = d_E X \cot \theta$, where d_E is the horizontal deflection.

This corresponds with previous publication BCSA No 29 – 1966, *Plastic design of portal frames in steel to BS 968.*

The results obtained in various portal frames analyses still leave me uneasy. Below is a list from various designs.

Span (m)	d_E (mm)	d_{RE} (mm)	span/d_{RE}
10.00	8.13	77.35mm	129
18.00	17.85	111.5mm	134
18.50	13.40	125.0mm	148
30.00	24.00	288.0mm	104

These span/deflection ratios appear not to conform to serviceability requirements. What should these be for roof slopes varying from 5° upwards, with profiled metal cladding installed?

Have members had any problems either satisfying themselves that deflections are within reasonable limits or in persuading the regulatory authorities? Are reliable in-field measurements establishing the impact of cladding on actual deflections? The theoretical spread at the knee of a portal can appear daunting when based on simple calculations, and this has occasionally caused concern regarding walls tied to such portals.

Engineering salaries

An engineer who gives his name, but prefers to remain anonymous, has reacted to the Engineering Council's Survey of Engineers (Verulam, 4 November 1997):

I am one of the silent majority of the Institution. However, following your article entitled 'Engineering Council – Survey of Engineers' I felt that I had to express my dismay over this matter.

I am a Chartered Structural Engineer and I know that I speak for a very large percentage of my colleagues. When average salaries are supposed to be in the order of £40 131 p.a., I would like to be Mr Average!!! This order of salary would translate to an earnings *per capita* of over £100 000 (including overheads). It is interesting to note that none of the consultants in the *NCE* listing came anywhere near this figure. I have actually considered exchanging my corporate status to become an Incorporated Engineer as, at an average rate of £29 918 p.a., I would still be getting quite a substantial salary increase. I cannot believe that such a survey can be published in all seriousness. I consider comments, such as those made by Mike Heath, as nothing more than the words of a politician who does not appear to be in contact with the troops on the ground. Mr Heath would not appear to have read Verulam over the past few years.

If my remarks prove incorrect, would someone please include an application form?

Are other members too stunned by the survey to write, or do you believe its figures?

BS 8002 *Earth-retaining structures*

The battle of the giants – fought in mud! – continues with unabated fury. Mr F. M. Rymill from Ruislip writes:

In his reply to the views expressed by various correspondents Mr Akroyd shows again that he does not understand the difference between (a) assuming that a worst credible load will somehow increase by, say, 1.4, which would indeed be incredible, and (b) providing a structure which is 1.4 times stronger than the worst credible action. My example of the tank of water was to make precisely this distinction. No, Mr Akroyd, I do not expect 10m of water under any circumstances to exert the pressure of 14m of water. But I do believe it to be sensible for the tank to have an ultimate strength of about 1.4 times the applied maximum action. And, thankfully, so do BS 8007 and BS 8110.

The foreword to BS 8110: *Part 1:* 1985 explains with admirable clarity the basic approach to designing structures using limit state methods. The assumptions regarding the ULS are basically as (b) in my opening paragraph, not (a) as Mr Akroyd assumes. Regrettably, clause 3.2.7 of BS 8002 is far from clear. Presumably, 'accordingly, the application of partial load factors.... is not normally required' means that the load factor for the ULS should normally = 1.0. This is both unsound and impractical. A RC wall with an ULS design based on a load factor of 1.0 would result in quite untenable stresses at the serviceability limit state. The designer would then have to increase the section, recalculate the stresses, by trial and error until an acceptable solution was found – a waste of time which could be avoided by using a load factor of 1.4; a point made very clearly in BS 8110, clause 2.4.3.1.2. Again, Mr Akroyd does not appear to see that there is a direct relationship between the stress at the SLS and the load factor adopted for the ULS.

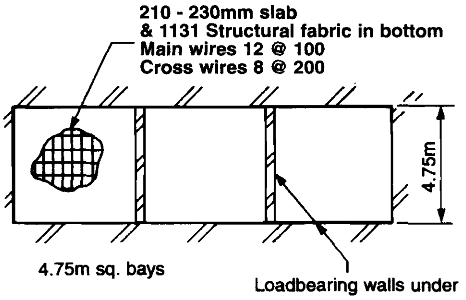
Lastly, the fact that the pressure on the wall will decrease should the wall collapse does not lead me to the conclusion that a load factor of 1.0 will produce a safe design. Surely, the aim is to provide a wall that does not collapse. To do this the wall must have an ultimate strength which exceeds the maximum credible force by a reasonable margin, generally taken to mean 1.4 or thereabouts. The wall would then be both safe and serviceable.

Surely, there can be no doubt that guidance notes are required.

Mr J. E. Hall, from Holland-on-Sea in Essex, has responded to Mr Rymill's previous contribution:

(1) I was interested to read the comments of my old friend and colleague Fred Rymill on the interface between BS 8002 and BS 8110. I am in general agreement with them. In the case of water pressure the horizontal thrust and load are accurately known, water is a permanent dead load, so the ultimate limit state factor of 1.4 should be applied when designing a RC structure to contain it

(2) Soils, however, do not behave like water; their horizontal thrusts vary depending on their angle of issued friction ϕ'. The now defunct DCES branch of PSA issued an advisory letter concerning the ULS factors given in Table 2.1 of BS 8110: *Part 1* for earth and water. I no longer have this letter, only a pencilled note which says water and equivalent fluid pressures, i.e. active soil pressures. In para. 2.4.3.1.2 there are further notes on soil pressure and then a reference to *Part 2* of BS 8110; Table 2.1 gives reduced partial factors to be applied to worst credible values (partial factors vary between 1.0 and 1.2) and also notes in paras 2.2.2.3 and 2.2.2.4.



Notes:
1. For lighter loadings, this could apparantly be extended to 5.5m sq bays!
2. There is no top reinforcement! This would be a complication explicitly ruled out by BS 8103.
3. Transverse reinforcement is trivial and would not satisfy BS 8110 (i.e. proper design) even as anti-crack, let alone 2-way bending.
4. Who will be sued concerning the likely cracking? Building Control? Engineers will not have been involved.
5. For approximately square bays, there would be a tendency for 1-way spanning depending on which way the mesh was thrown in. This could seriously affect the resulting loadings on support structures.

Fig 2. Example of a concrete slab 'design' to BS 8103: Part 4: *1995*

defects are potentially hazardous? Are 'cleverer' Codes unduly rigorous? Have we allowed excessive expectations to build up among clients? Totally crack-free concrete is rare, but we have got better at spotting the almost invisible crack, in stonework also? Perhaps ignorance avoided much needless repair. Any opinions?

Barrow Bridge Chimney

Pauline Poscoe, writing from Chorley, Lancashire, is examining the potential for restoring this chimney, which is allegedly of historic importance as an expression of Bolton's industrial past. She is looking for historic information, particularly where it might be relevant to its restoration, and ideas for reuse of the chimney as a feature in a Heritage Centre. The Institution's Editorial Department can provide an address.

Deflection of portal frames

Francis Beale has responded to comments by Mr Camilleri (Verulam, 20 January 1998) and writes from South Croydon:

Based on my experience in the 1970s of working for a national contractor on design-and-construct work, we always separated brick walls from portals because of the horizontal deflections. However, when surveying existing structures for extensions, etc., where work had been constructed by other contractors, this always caused us a considerable amount of angst. However, cladding on portals must be able to resist considerable loading in its plane because of live loading which must limit lateral movements, but be difficult to quantify.

The practice described, of keeping walls separate, is not uncommon, but trades one problem for another as walls still need to be stabilised. For tall walls acting as cantilevers this thus provides a different challenge.

More next month from a large post bag, much of it incredulous, on engineers' alleged salaries.

of the value of CPD and of making a declaration that they will participate in it.

One of the major complaints from members, which is a constant bellyache coming through loud and clear from the CPD returns, is the lack of status of professional engineers in the UK, with every Tom, Dick and Harriet who picks up a screwdriver being able to call themselves an engineer. But surely to achieve status we have to deserve it. If three-quarters of our membership are not prepared to make a declaration that they will keep themselves up to date with developments in their profession, the fault is clearly in our own stars that we are underlings.

I was Chairman of the review panel which brought in the present system. It was deliberately designed to reflect the fact that our members are professionals, and not naughty fourth-formers, who must give a detailed return stating which meetings they went to, dates, etc. Our return genuinely consults the members as well as providing our Institution with a minimum of statistical information. It probably takes between 15 and 30 min to complete.

Our system is not compulsory – yet! We wanted members to get used to making returns and to seeing the benefit of the exercise. Other professional institutions treat their members as children by introducing a compulsory system straight off.

In my opinion, compulsion will come in due course.

Why should we introduce compulsory CDP? Well, interestingly, another thing which our members were very keen on in their returns was that professional engineers should be licensed/registered. I entirely agree. That may be one of the ways of dealing with Tom, Dick and Harriet. But why should we license people of whose competence we have no knowledge. It might be argued that the last time we heard from most of them (other than to receive their subscriptions) was when they took their Part 3 – in some instances, 30 years ago. Doctors and lawyers, both of whose membership are registered by law, have to undertake regular compulsory CPD of a fairly rigorous nature. We engineers cannot have our cake and eat it.

Of course, the deadline for returning CPD forms this second year has already passed. Hopefully, a greater proportion of members will have returned the forms and will be joining those whose CPD commitment is now clearly indicated in the *Yearbook*.

With luck, the message is gradually getting through.

One last interesting point. I understand that a recent legal case concerned an engineer who was being accused of negligence. He was asked whether he would show the court his CPD diary. He could not produce it (presumably, because he had not kept it!). Then the expert appearing for him – another engineer – was asked for his CPD diary; he had not kept one either.

The case went badly against them, and lack of competence and knowledge of their subject demonstrated through their lack of commitment to CPD was a major factor in their mutual downfall.

And why should it not be so?

Members may recall Steve Evans' comments in the November issue of the journal and the reasons he gave why CPD, now demanded in several professions, was essential to the status of engineers. Without in any way wishing to denigrate the need for CPD, we should point out that there are special factors relevant to structural engineers which are perhaps less so to the members of some other professions. A general practitioner will continue to consider the medical requirements of his patients and thus needs to update his knowledge of a field with which he is familiar, even if perhaps out of date. A structural engineer brought up on analysis may well find, as he gets promoted, that analysis is only one of many wider issues which he did not previously learn about. CPD must therefore cover a much wider range of options. Compulsion firstly implies a check on the adequacy of the CPD undertaken. If registration comes with it, there is also danger that others – perhaps governmental agencies – will wish to have a hand in the registering process. This has not brought undivided joy in countries where it has happened. We need to take care so that we avoid that old game where sunshine on the other side of the street seems more attractive. Those countries that have compulsion frequently admire the UK because professions regulate themselves. Would we wish to take a risk on their systems which they may prefer to change?

Comments from members, and especially members working in countries where there is compulsory registration, would be very welcome.

Deflection of portals

Mr C. S. Westerbrook has written from Romsey in Hampshire:

With reference to the concerns raised by Mr D. H. Camilleri (20 January 1998) regarding the ridge and eaves deflections calculable from the formula given in *Manual for the design of steelwork building structures,* may I make the following observation?

No guidance is provided within the text of clause 11.5 regarding this serviceability check for the reducing effects of:

(a) haunching — applicable at the eaves and apex in most conditions;
(b) portal stanchion to base connection — in most situations the 'pinned base' condition provides some degree of fixity;
(c) cladding — even the lightweight systems providing some overall stiffness.

In addition to the above, it is common practice within the commercial sector of the steelwork industry to preset the rafters for theoretical dead load deflections in an effort to create the true geometrical shape prior to the addition of any superimposed loading.

When the preceding factors are considered, the 'actual' deflections encountered are greatly reduced and found in practice not to cause any concern.

Having spent over 35 years associated with portal frame construction, I believe that any deflection difficulties are usually caused by fabrication or erection errors or, in some cases, by the choice of an incorrect mode of construction for the client's brief. I trust that this is a correct perception in the use of portal frames.

Does this satisfy members regarding portal behaviour?

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BS 8002 — Earth-retaining structures

Mr B. N. Sharp, who is a member of various BS committees and working parties addressing maritime structures, has written from Ealing, London W5, also referring members to the PIANC Report by a Working Party on 'Recommendations for the construction of breakwaters with vertical and inclined concrete walls – Report of Subgroup C'. He continues:

It has not been explained that BS 8002 does not apply to maritime structures. This is of great concern, as water loads can greatly exceed that of submerged soil, and we therefore now have no BS guidance at all. The Subgroup C subreport, and the full sections covering this topic, and a summary section, elucidates guidance as requested by your contributors – and would at least merit public comment. It will eventually be summarised in a published reprint by PIANC, of which the main subject is, of course, wave and hydrodynamic loading and structural stability in relation to probability and wave loading.

BS 8002 is referred to in BS 6349 (Maritime Code) and I have not got very far in explaining to BSI that BS 8002 does not cover maritime structures (e.g. BS 634g) and is a source of controversy as regards the design of structural members. Dr Bolton actually says that BS 8002 can be used directly in a BS 449 (working stress) Code or a limit state Code – this cannot be correct, as numerous contributors at the meeting, and since, have pointed out.

Mr Sharp has enclosed the current PIANC draft which interested members may wish to obtain.

Deflection of portals

Following on previous letters to Verulam (21 April 1998), Mr J. S Marshall from Lichfield, Staffordshire, has written, expressing his concern at inadequate guidance regarding portal deflection. He says:

Why can't the IStructE, ICE, BSI, SCI, BSCSA, *et al*, get together and produce definitive guidelines?

Dotted amongst many and various publications from around the world are methods for assessing deflections, together with recommended limits to satisfy serviceability, but none of the above-mentioned bodies seems interested in producing a set of rules.

Qualitative expressions such as 'some degree of fixity' and 'some overall stiffness' abound when considering the effect of cladding, baseplates, and connections in general. The commercial sector seems to imply that accurate deflection assessments are impossible but happily constructs frames in predeflected shapes to compensate for these supposed unknowns.

If the SCI, BCSA and like bodies cannot produce definitive rules for frame deflection limits because of their reliance on the commercial sector for funds, then the learned bodies should step in. How can anyone justify designing a frame to the exacting requirements of BS 5950 only to say, at the end of it, the design model is wrong and the deflections should be ignored? Are they to be ignored when calculating the P-Δ effects which are reported to be a requirement of the forthcoming BS 5950 revisions?

Surely an all-embracing document stipulating maximum THEORETICAL deflections is long overdue. All designers could then work to the same rules which local authority checking engineers would be expected to enforce.

Michael Searle, writing from Oxford, gives a specific example where the use of accepted procedures appears to provide misleading answers:

Further to the concerns expressed by Mr D. H. Camilleri and Mr C. S. Westerbrook about deflection of steel portals, I have found that the *Manual's* formulae tend to be on the conservative side and that the calculation of eaves and ridge deflections is generally disregarded by designers whose experience has indicated this not to be critical. However, BCSA publication no.19: 1963 gives an apparently simple method of calculation of deflection anywhere on a frame using area moments.

For a portal under any load condition with fixed feet held in position and direction a simple hand-calculation provides vertical and horizontal deflections at, say, the two eaves positions.

The transverse deflection across eaves levels may also be easily obtained. This may then be checked as its value will be the relative value from the previously calculated eaves deflections, either their difference or sum.

For a portal under any load condition with pinned feet held in position but not in direction a modified approach is given (the theoretical slope of the post at the pinned connection increasing frame deflection).The transverse calculation appears to remain as for the fixed feet condition but with different moments to suit.

By way of checking the method I took a 10m span portal, 5m to eaves with a 2 in 5 rafter slope, constant I throughout of $4400cm^4$.The loading consisted of a single 20kN horizontal thrust applied at the left-hand eaves pushing the frame to the right.
For the fixed frame the transverse deflection was calculated as 3.78mm and the individual deflections at eaves level 22.59mm and 18.81mm, resulting in a relative movement of 3.78mm, providing a suitable check.

So far so good. However, when considering the pinned condition the transverse deflection was 5.01mm which seemed reasonable but at eaves the values were 120.55mm and 89.77mm, giving a resultant difference of 30.78mm.

Careful checking has failed to show a numerical error, so a fundamental one is suspected. Also, I should have expected the pinned deflections to be higher than the fixed condition but increased both by the same ratio, which they are not.

Has any other engineer had the same difficulty with the BCSA method or do I stand out alone? I should be delighted to be put right on this method which is basically so simple and could be used to consider the effect of partial fixity of the usual bolted down 'pinned' connection which, as suggested by Mr Westerbrook, is significant when considering deflections.

I also considered the effect of haunching which the area moment method can cater for and found that the small haunches used these days have only a small effect on deflections.

By way of a postscript, a colleague was visited by a rep. displaying his laptop PC and demonstrating a portal frame design. A frame size was produced and checked by the rep. who was asked to find the eaves deflection and at a press of a button 100mm appeared. My colleague, observing the likely damage to brickwork, was told – don't worry about that, that's not your concern, you're only designing the steelwork, let somebody else worry about the design of the brickwork!

Have other members identified reliable methods of assessing deflections, preferably backed by field tests, or is the suggested working party necessary to identify reliable methods and promote their application?