Verulam

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Topics of importance openly discussed

In defence of brainteasers as teaching aids

Emeritus Professor Jim Croll has joined the debate about the value of the 'And finally...' teasers in *The Structural Engineer*.

It is difficult to understand why Bill Harvey (Verulam, January 2018) should be quite so 'incandescent' at recent, very laudable efforts in the 'And finally...' pages aimed at encouraging improved understanding of structural behaviour. His focus on the solutions for the simple rigid jointed portal frame, covered in the October 2017 issue, seems particularly baffling.

For the past few decades, I have been using similar examples in an attempt to improve students' understanding of structural behaviour. This is as an antidote to our increased reliance in university courses on methods that form the basis of most structural analysis software, and the use of such software to perform analyses in practice. Encouraging students to draw and relate bending moment and deformation diagrams one to the other allows the mutual importance and interrelationships between the principles of force equilibrium and deformation compatibility to be thoroughly understood.

So, for the 'And finally...' problem of October 2017, reproduced here as Figure 1a, the deformation would most certainly take the form shown in Fig. 1a, as was given by the suggested solution B. This deformation line would relate directly to the bending moment diagram of Fig. 1b, so that moments are drawn on the convex face of the deformation line and, of course, are zero where the curvature changes sign at the points of contra-flexure.

Hogging moments at each end of the beam, resulting from the constraint provided by the columns to the end rotations of the beam, have the effect of lifting the free body bending moment to ensure the moments are zero at the points of contra-flexure. All of which helps the student to understand the relationships between force equilibrium and deformation compatibility. As long as the student is made aware that the deformations are drawn to highly exaggerated scales simply to make them visible, then surely no one can take issue with this?

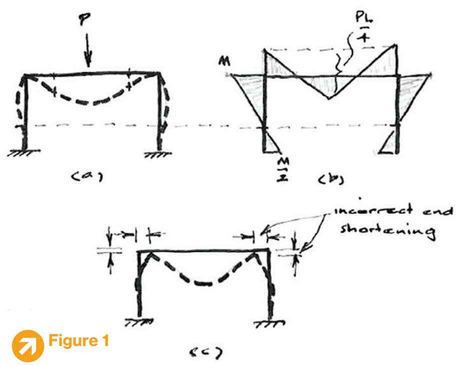


Figure 1

It is an understandable and common error that, when drawing deformation lines to these exaggerated scales, an almost unconscious allowance is made for the fact that were the deformations to be actually that large, then to preserve the original length of the beam and columns, the joints should be drawn, pulled inward and downwards, as shown in, say, Fig. 1c. So, the alternative solutions A and D are examples of this very common form of error.

But, of course, in designing our real structures, we should be limiting the level of deformation to around 1/250 of the span. Drawn at real scale, such deformations would be less than the thickness of the pencil lines and consequently invisible to the naked eye. So, at real scale, there would be no tendency to show the joints as shifting to compensate any perceived change in length.

Properly used, such examples can provide a powerful vehicle for the understanding of structural behaviour, as suggested by Martin Ashmead, and most certainly not the 'educational disaster' claimed by Bill Harvey.

I am currently putting the finishing touches to a book, based upon a course provided at University College London over the past 30 years, that makes extensive use of similar forms of qualitative analysis. This book, and the courses upon which it is based, go on to demonstrate how these physically based approximate analyses in the context of the static (lower bound) theorem of plasticity and ultimate state design provide powerful bases for the design of structures, reducing reliance on computer software and, by encouraging better structural understanding, aiding conceptual design processes. At the very least, they provide a method for interrogating the veracity of output from commercial software.

Solving the cranked beam problem

With his views firmly expressed in his previous contribution, Professor Croll has also commented on the trickier cranked beam problem discussed in the April issue.

Could I also proffer some comments on the resurrected cranked beam problem originally set by Martin Ashmead back in May 1982? Reading Bill Harvey's description of the system behaviour (Verulam, April 2018), reproduced as Figure 2a, I am afraid leaves me little wiser and clearly also left the editor rather perplexed.

Here is an example where the understanding being encouraged in the 'And finally...' pages can be usefully deployed. Replacing the vertical load (taken as $\sqrt{2}\,P$ so that the axial and normal components are approximately P) with its components normal and parallel to the axis of the cranked beam, it is possible to treat this as a pure membrane solution whereby the axial load component P is directly transmitted by column behaviour to be equilibrated by an equal and opposite reactive force P at the pinned lower support A, plus bending due to the normal component P at the top of the cranked section. The bending contribution would have

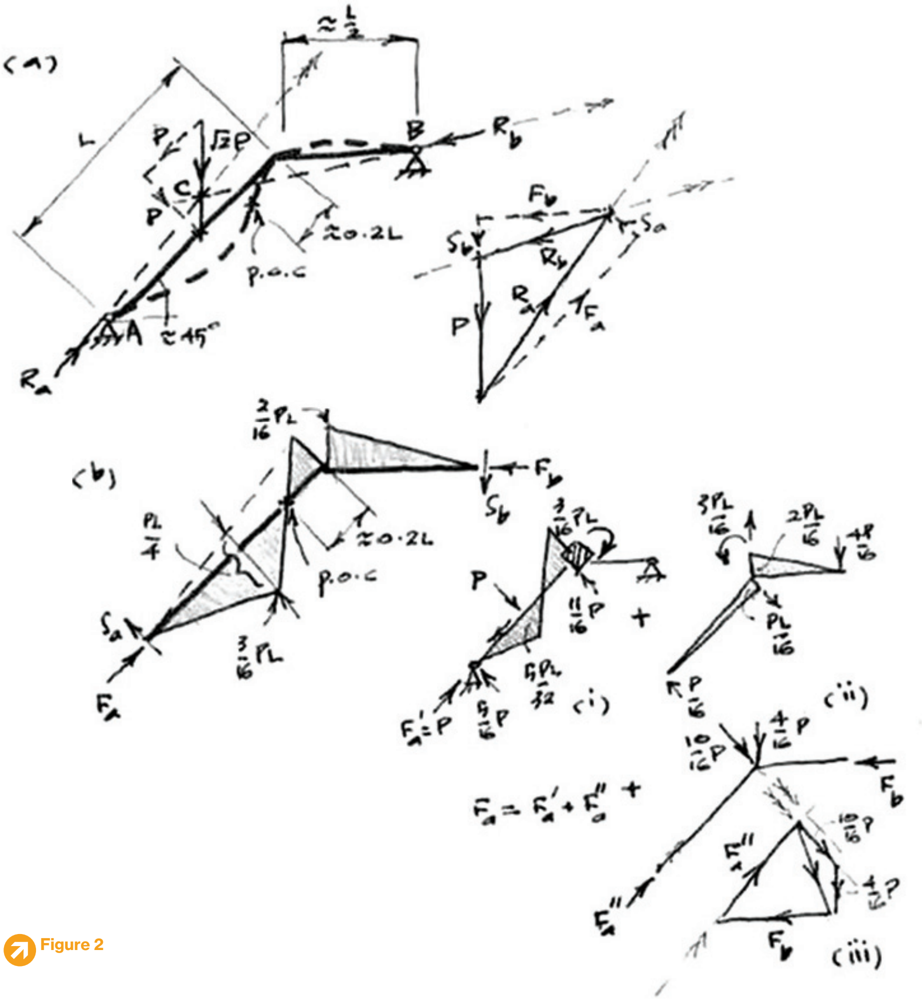


Figure 2

a deformation line as shown in Fig. 2a and an associated moment distribution as Fig. 2b.

Were it required for the behaviour to be converted into an approximate quantitative solution, perhaps suitable for the initial design of the system, it could be observed that the line of action of the reaction R_b must pass through the location on the inclined beam where the point of contra-flexure (poc) occurs. This in turn defines the point C where the reaction R_b intersects the line of action of the applied $\sqrt{2}P$ load. For this three-force system, equilibrium demands the three forces be concurrent, thereby uniquely defining the line of action of the reaction R_a, and the magnitudes of the reactive forces determined from the triangle of forces shown inset within Fig. 2a. Even with the qualitative nature of the sketch shown, it is possible to estimate the force magnitudes. Recognising these lines of thrust provides another way of interpreting the moments shown in Fig. 2b, since the moments simply represent the product of the end reaction times their normal distance from the line of thrust to the point on the frame axis.

How such qualitative methods can, with practice, be converted into quantitative behaviour is explained for this problem in the insets to Fig. 2b. Taking the joint between the inclined and horizontal beam to be conceptually rigid allows the moments to be given exact quantitative values for the propped cantilever, all as shown in inset (i). The out-of-balance clockwise joint moment, $3PL/16$, can then be eliminated by applying an anticlockwise moment to the original frame of the same magnitude, resulting in an anticlockwise rotation of the joint. Taking the horizontal beam to have roughly half the length of the inclined beam, this correction moment will then be distributed in the ratio 2:1 between the horizontal and inclined beams, as shown in (ii), resulting in the quantitative moment distribution of Fig. 2b, which is sufficiently accurate for it to form the basis of at least an initial design.

To complete the quantitative analysis for this frame, it is also necessary to apply the corrective joint forces at the knee, shown in sketch (iii). These are equal and opposite to the reactions developed at the knee as part of the idealisations illustrated in the insets (i) and (ii) of Fig. 2b. This then allows full specification of the axial forces in the members. Those with as much grey hair as myself will recognise the processes briefly described above as effectively the steps involved with the moment distribution method.

Professor Croll has taken this a step further than required in the original puzzle, which only demanded a prediction of the shape of the bending moment diagram. As ever, predictions of magnitudes by approximate methods reveal insights into performance. Professor Croll's methodology is at least partly based on an appreciation of system stiffness, which is the point Bill Harvey wanted to bring out.

Clarifying the cranked beam problem

Reader Melvin Hurst sends us a correction.

I am sure that I'm not the first to point out that, in your Verulam pages of April 2018, when you discuss Martin Ashmead's brainteaser of May 1982, there was a slip of the pen (or should that be slip of the keyboard?) on your part. At the beginning of the second paragraph you state that the poser is statically determinate if the supports at A and B are infinitely stiff in the horizontal direction.

However, the structure is only determinate if, as Bill Harvey notes, there is either a horizontal roller at A or a vertical roller at B. As it is, there are four unknown reactions, although the two horizontal ones must be equal and opposite, leaving only two statical equations to be formed from which the remaining three unknowns can be determined. This is, of course, the definition of an *in*determinate structure.

There is also a phrase missing from the last sentence of Bill Harvey's third paragraph (highlighted in italics): it should read '... and the force at A must be W vertically along with *a horizontal force* H, *with the resultant* inclined to meet force W where...'.

My view on the debate is that both equilibrium and stiffness must always be considered, although, for a determinate structure, equilibrium alone will suffice.

Melvin is, of course, correct. The teaser was indeterminate and his addition to Bill's contribution adds clarity. The essential message remains that actually the solution to the teaser is linked to the assumptions on support condition, which will lie between infinitely stiff and unrestrained, and the 'answer' is only as accurate as those assumptions. As Bill and Melvin state, stiffness must always be considered.

Water trapped in hollowcore floors

It's always nice to find an answer to a reader's queries. Here, based on his experience as Chief Engineer and Quality Manager of three hollowcore floor manufacturers, Cliff Billington replies to Sean Lightowler about water trapped in flooring units (June 2018).

The water in question is a result of rain. During prolonged exposure in the manufacturer's yard and on site, rain will penetrate the relatively porous top surface of the floor unit. However, the bottom zone, being under prestress, has no cracks and is pretty much impervious, and thus the water will accumulate in the hollow cores.

For plain, open-ended units, this is not a problem, as the water will simply run away. However, when used on a steel frame, the units often have a reduced-depth end to fit under the top flange of a beam. To compensate for the loss of cross-section, and to stabilise the remaining concrete, a concrete bung is formed (Figure 3). This bung obviously prevents water from escaping, and it is normal for the manufacturer to form drainage points to deal with this. These are formed by 'drilling' from above in line with the hollow cores, using a blunt drill bit, often a piece of rebar, so as not to damage the smooth casting bed.

The drilling does not go right through the bottom flange, and it is necessary to open the drain by piercing the remaining thin skin from above with a pointed piece of rebar. It is not recommended to drill at mid-span as Sean suggests. If a floor unit has a camber of say 30mm, then drilling at mid-span and releasing water will still leave 30mm of water at the ends, which can still cause staining. It is, in any case, not best practice to drill upwards with electrical tools into cascading water.

The hairline cracks are not caused by the water, although in extreme cases it is possible for a core completely full of water to freeze and split a unit in two. It is more likely that the cracks are already present and are simply made more obvious by water percolating through them and leaving dark streaks.

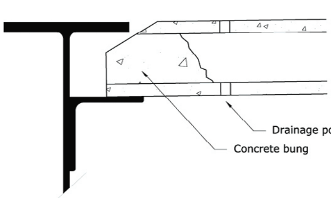


Figure 3

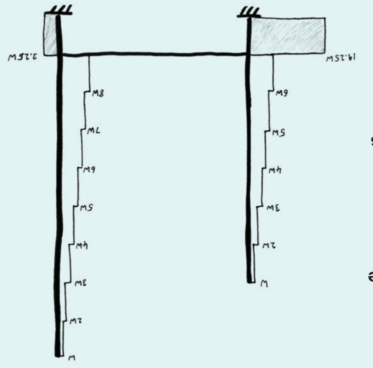
Well, there you have it.

AND FINALLY...

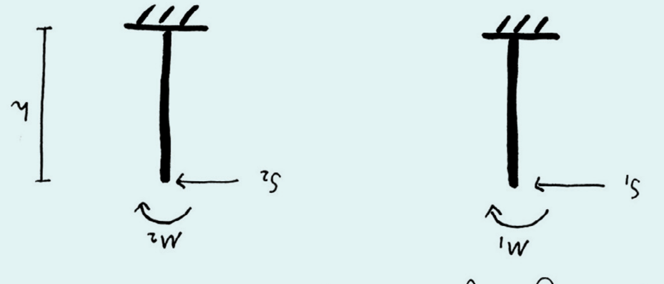
Answer to July's question

At first glance, it might be expected that the taller block would have the larger base shear, or that the shear would be shared equally between the two at ground level. In reality, the base shear in the shorter block is much higher. Note that it also has a larger base moment, so the foundation design could be more onerous for the shorter block.

The key is to remember that, due to the rigid podium, the deflection of each core must match at first-floor level. This deflection is made up of two components: the deflection due to the shear force and the deflection due to the moment. As the moment at first floor is higher in the taller tower, the shear force must be lower to compensate.



Consider a free body diagram cut just below the podium



Moments : $M_1 = (1+2+3+4+5+6)Wh = 21Wh$

$M_2 = (1+2+3+4+5+6+7+8)Wh = 36Wh$

Shear : $S_1 = 7W + P$

$S_2 = 9W - P$

where P is the force transferred by the podium at first floor

Compatibility : Rigid podium means first floor deflection in each core is equal

Deflection due to an end moment $= \dfrac{Mh^2}{2EI}$

Deflection due to a point load $= \dfrac{Sh^3}{3EI}$

$$\frac{21Wh^3}{2EI} + \frac{(7W+P)h^3}{3EI} = \frac{36Wh^3}{2EI} + \frac{(9W-P)h^3}{3EI} \quad (\times 6)$$

$$63W + 2(7W+P) = 108W + 2(9W-P)$$

$$63W + 14W + 2P = 108W + 18W - 2P$$

$$4P = 49W$$

$$P = 12.25W$$

$$S_1 = 7W + 12.25W = 19.25W$$

$$S_2 = 9W - 12.25W = -3.25W$$