Correspondence

Dr John Bellamy (F) has written in about the paper by Eugene J. OBrien & Colin C. Caprani on 'Headway modelling for traffic load assessment of short to medium span bridges' (The Structural Engineer, 16 August 2005).

As a former bridge specialist I read this paper with interest; I found the authors' basic concept of using statistical techniques to determine more realistic values for traffic loads on bridges to be valuable, particularly if it resulted in avoiding re-building or strengthening. Unfortunately there is a major flaw in the structural idealisation of the bridge used in the paper which I think invalidates their conclusions. For a huge number of vehicles and positions the authors have calculated three load effects and from these determined the appropriate traffic load for assessment/ design. The load effects are:

- Bending moment at the mid-span of a simply supported bridge
- Left support shear in a simply supported bridge
- Bending moment at central support of a two span continuous bridge

These appear to be the total effects on the bridge idealising it as a simple one-span or two-span beam. Modern bridge decks of the spans and widths considered by the authors function as pseudo plates with low transverse stiffness. A common form of bridge deck consists of closely spaced prestressed concrete beams linked by a thin *in situ* concrete slab; analysis of such a deck (see Bares and Massonnet '*Analysis of Beam Grids and Orthotropic Plates*' Crosby Lockwood 1968) reveals that the loading from a vehicle is carried pre-dominantly by the beams under the vehicle with the beams under the adjacent traffic lane receiving little or none of the weight of the vehicle. Other methods of constructing bridge decks such as voided concrete slabs and reinforced concrete slabs on steel beams work similarly. Thus two lanes of traffic are only marginally, if that, more severe than a single lane. The authors appear to have added together the effects from two traffic lanes and from this have concluded that the worst scenarios are with vehicles on adjacent traffic lanes (fig 9). I believe that if they had used a more realistic model for the bridge deck they would have concluded that the worst scenarios were for a single traffic lane.

Authors' reply:

Firstly, we would like to thank Dr Bellamy for taking an interest in our work.

In this work we did indeed add the effects of traffic in each lane, neglecting the transverse flexibility of the deck. This is a common approach in comparative studies (Cooper 1997) due to the considerable variability in transverse distribution factors between bridges - clearly voided slab decks for example have considerably more load sharing capacity than beam-and-slab decks (OBrien and Keogh 1999). The purpose of this study is to identify the importance of accurately representing the headways between vehicles. This greatly influences whether or not two following (same lane) trucks contribute to a maximum load effect. For bridges with low transverse stiffness, same-lane trucks will feature more prominently and accurate headway modelling will be found to be even more important than for the full load sharing case considered.

References

• Cooper, D. I.: 'Development of short span bridge-specific assessment live loading', *Safety of Bridges*, Ed. P. Das, Thomas Telford, London, 1997

• OBrien, E. J. and Keogh, D. L.: *Bridge Deck Analysis*, E & F-N Spon, 1999.

Graham Hutton, CEng, MIStructE, MaPS, Technical Director, Midas Homes has written in regarding Mehdi Khabbazan's paper on 'Progressive Collapse' (The Structural Engineer, 21 June 2005).

I have read with interest the paper presented by Medhi M. Khabbazan in *The Structural Engineer* regarding progressive collapse. It is interesting and informative and his statement that the detailed design for progressive collapse is generally an unfamiliar concept to structural engineers is particularly insightful. In my experience as a client (working for a residential developer) this is certainly the case and the revisions to Building Regulations Part A 2004 have meant that designers are having to cope with the new disproportionate collapse requirement where previously they did not have to do so. In my experience, the most difficult issue to address is that of Class 2A buildings where it is necessary to provide 'effective horizontal tying or effective anchorage of suspended floors to walls'. Mr Khabbazan's paper correctly identifies that it is difficult to achieve this in masonry buildings and suggests that a simple method to achieve this is by providing a thin layer of concrete topping, reinforced with a layer of steel mesh reinforcement. Unfortunately in most low rise residential buildings there are opposing aims which make this very difficult to achieve.

Firstly, all of the Robust Standard Details for acoustic insulation under Part E of the Building Regulations do not consider the inclusion of a structural topping to the pre-cast concrete plank floors and the risk to a developer of pre-completion sound testing is often high. The NHBC technical newsletter Issue 31 ran an article outlining the changes to approved documents A, C and P and stated that the requirement to provide effective horizontal ties or effective anchorage for floors to walls can be achieved by 'slabs bearing onto walls or by use of straps'! When specifically pushed they refer to the strapping arrangements shown in the appendices of BS 5628. I would be very pleased to hear members views on this as I am aware of developments that have achieved Building Regulations approval merely by using mild steel galvanised straps. Clearly there is a commercial benefit in adopting this approach but it is clearly (in my view) outside the spirit of the requirements of Part A. I cannot see how an effective anchorage or horizontal tie can be achieved by steel straps plugged and screwed to a masonry wall. In the event of a collapse the single block to which the straps are screwed would surely pull out of the wall. The details in BS 5628 are (in my view) intended to show good practice in providing lateral restraint to masonry walls under normal service loads (i.e. wind) not to provide restraint from collapse loads.

Members thoughts and discussions on this are greatly welcomed as whilst I would like to avoid using a structural topping in masonry flats, my conscience can't allow me to simply strap planks to a wall.

Authors' reply:

Firstly I am very thankful for Mr Graham Hutton's comments on my paper.

Secondly in Mr Hutton's letter there are two main queries which should be answered, i.e. available approaches to deal with the problem of progressive collapse in low rise residential masonry structures and their effectiveness in the event of an accidental abnormal loading.

The technique suggested in the paper, i.e. reinforced concrete topping is the most robust flooring system for a multi-storey structure but not the only available solution. Other techniques with less robustness such as precast concrete plank with reinforcement bars buried in grout filled joints between the planks and timber board and joists adequately tied are also common practice. If instead of effective horizontal tying, the option of effective anchorage of suspended floors to walls is adopted then the designer needs to ensure that the tie forces are safely transferred to walls. The transfer of these forces depending upon the practicality and workability issues can be through the physical means such as fasteners, angle cleats, straps, or through the shear strength or friction between precast planks and masonry wall. In any case the tie forces calculated per BS 5628 should be transmitted to supporting wall ensuring that there is enough resistance and capacity in the wall to withstand any shear or tensile forces generated by this action.

For a two-storey masonry apartment block with a distance of 3.5m between cross walls and precast concrete planks spanning between cross walls, according to BS 5628 the tie force per metre width of the precast planks, assuming a characteristic dead and imposed load of $6.5KN/m^2$, is: $(20 + 2 \times 4)(6.5)(3.5)/(7.5)(5)=$ 17KN/m and considering the available masonry fasteners this depending on the strength of the wall units may require between four and eight M12 resin anchor bolts/m width of the floor.

Certainly large enough to engage more that one block/brick each fastener usually will be fixed to one unit, i.e. block/brick).

The second part of Mr Hutton's query, i.e. effectiveness of these measures in avoiding progressive collapse goes back to the fact that we cannot realistically design the buildings to eliminate the risk of progressive collapse altogether. The design for progressive collapse is only to reduce the risk of collapse and in some buildings depending on their occupancy level and importance the risk can be higher than others. A building Classed as A2 according to the Approved Document Part A3 is only a medium Class as far as progressive collapse is concerned (please refer to the risk factor definition given in the paper) and as such only requires a limited measures against risk of progressive collapse. Many robust designed masonry buildings need only little additions to make them more resistant to abnormal accidental loadings.

Correspondence

Consulting structural engineer Andrew Smith (M) and David Yeomans, engineer, historian of carpentry and secretary of ISCARSAH have written in regarding Shanks and Walker paper 'Experimental performance of tenon connections in green oak' (The Structural Engineer 83/17 p 40–45):

Shanks and Walkers' article on mortice and tenon connections has shed useful light on their behaviour when loaded in withdrawal. However we feel it would be a disservice to engineers if it led them to think that either that is how they were intended to work or that such action should be included in the design of new frames. Starting from their experimental failure loads of around 6–9kN, determining the characteristic load and applying a factor of safety reduces the BS 5268 allowable load to around 2kN. We agree with their assessment that Annex G of BS 5268 also gives an allowable load of just over 2kN. The simple fact is that a mortice and tenon joint loaded in withdrawal is not of much use: being considerably weaker than both the traditional joints that were intended to carry tension and modern screwed, dowelled or bolted lap connections.

Traditional timber frames contain very few members in tension and indeed are framed to minimise their number. Mortice and tenon connections such as Shanks and Walker have investigated were never intentionally loaded in withdrawal, and where a modern sense of structure suggests that this might occur, such as in the windward knee brace of a cross frame, it was always balanced by another member that could resist the same forces in compression, under which such braces are both stiffer and considerably stronger. The pegged mortice and tenon connection was only ever intended to transmit compression parallel to and shear transverse to the tenon – the peg was only there to hold things together – and for these actions we already have reasonably reliable ways of calculating their capacity. Where a frame has eventually loaded a mortice and tenon joint in withdrawal, this is invariably because its deterioration has forced it to act in unintended ways.

There are members whose action does require them to resist tension – king posts, queen posts and most tie beams to do so permanently, scissor braces and cruck wall spurs occasionally – but of these only the last ever relied on mortice and tenon connections loaded in withdrawal, and then only to carry wind load, at which they frequently fail. Until the 17th and 18th centuries when iron straps and bolts were introduced, carpenters traditionally used the lap dovetail and the wedged half dovetail to carry tension forces. The former were used in cruck spurs, scissor braces and between tie beams and wall plates: wedged half dovetails were used for aisle ties in early structures and at the feet of king and queen posts for later trussed roofs.

Wedged half dovetail joints were also often used in the UK to frame a beam into a post within its height, so that the vertical component of the load on the beam assisted the dovetail action to resist withdrawal. In their fig 13, Shanks and Walker suggest that such an intermediate beam might be framed into a post using a mortice and tenon connection and again we would not wish engineers to think that this is a sensible way to resist withdrawal. Beams framed into a post within its height are more common in French and German than in English framing, though there the tusk or through tenon secured by wedges is common. In France tie beams are often cogged over the wall plates in preference to lap dovetails. In contrast to this regional variation of practice in tension joints, mortice and tenon joints are always used in compression.

The great value of Shanks and Walker's work is that it is the first proper research into the behaviour of any traditional structural joint in timber, but we urge them to now turn their attention to those joints that were intended to resist withdrawal, which do give trouble and which cost a great deal to repair. We would suggest that the lap dovetail should be next on their list: we have tried to find ways of calculating their strength but some of their action is simply not calculable with our present knowledge – for instance drying shrinkage causes the dovetail to become looser at its broad end, concentrating withdrawal load onto the throat of the joint. It would be a great shame if this pioneering research stopped here as there are joints that it would be really helpful to know more about.

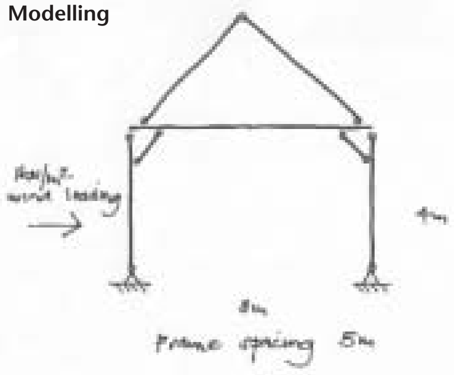
Dr David Brohn (F), has written the following, also prompted by Shanks and Walker's paper:

The paper by Shanks and Walker sets out the protocol for the testing of the strength in shear of oak dowels in typical mortice and tenon joints. The research is thorough and detailed and the results presented in a comprehensible way. Good science.

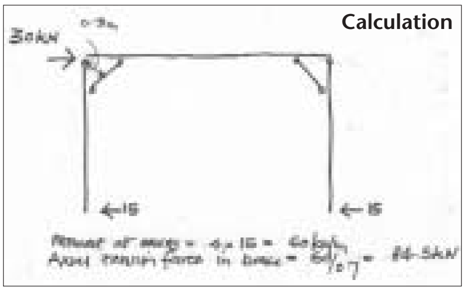
And it is all based on the entirely false premise that timber dowels or pegs are used in the jointing of timber frames to provide tensile strength of the joint. The pegs have a quite different contribution to make that is related to the technology of construction, but it is not to the strength.

It is true that it looks as if the wind braces are in tension because with a modern eye the **modelling** would have both wind braces pinned to the column and tie. The dimensions are of a small barn, 8m width, 4m to eaves and spaced at 5m. The windward brace would be in tension and the leeward in compression.

Modelling



Calculation



Making this as simple as possible we will assume a serviceability loading of $1kN/m^2$ wind loading which will produce an eaves point load of 30kN. This assumes a horizontal loading on the roof.

Calculation

The back of the envelope **calculation**, assuming that the brace connection is 1m from the eaves, shows that if the load is shared between the braces, the axial tension in the brace is 85kN!

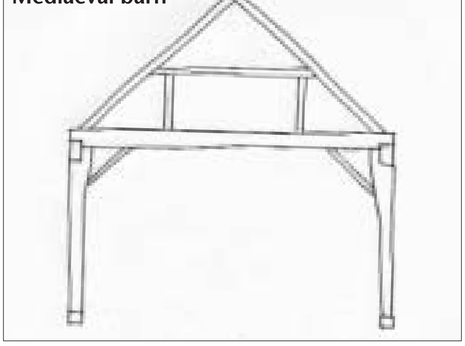
The test results show the performance of the pegs in shear to be similar to the classical elastic-plastic stress strain curve and if we take a collapse load of 8kN we would assume a safe load of say 4kN per peg in double shear. How do Shanks and Walker explain how these structures have survived for hundreds of years when the capacity of the connections is $^{1}/_{20}$th of that required?

In the early 1980s I was approached by a charity to advise them on the construction of a timber barn at a home for disabled people in Bristol. They had money from the Youth Opportunities Training Programme for the employment of the young people, but no money for equipment or components. In that airy way that academics have I recommended that they took on where the mediaeval carpenters left off in the 16lh century and build a **mediaeval barn**. No need for metal components.

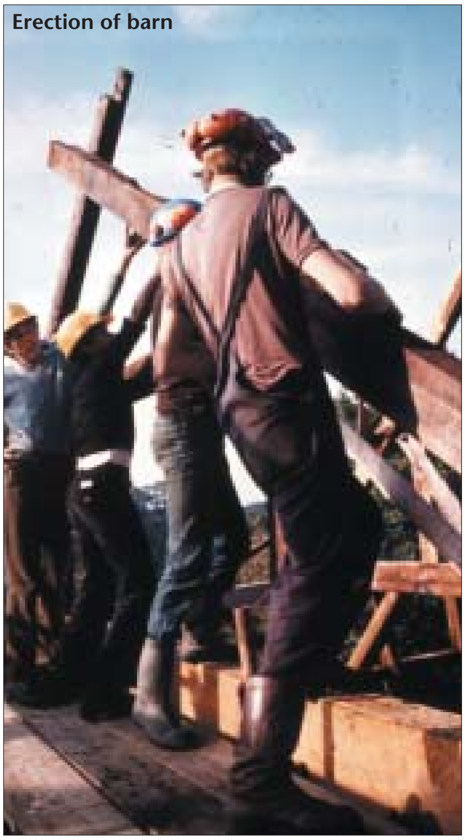
Two weeks later they called to say that they had cut down all the trees (this was the period of the Dutch Elm disease and there was plenty available) and what should they do next? I then realised that I knew nothing about mediaeval carpentry, but I had better find out quickly. Shanks and Walker found the same source, the remarkable series of books by Hewett. He was a carpenter by training and became the keeper of ancient fabrics for Essex Council. He had a wealth of experience derived from the dismantling of timber framed structures and their re-erection at open-air museums.

So I chose what looked like the most suitable arrangement for the frame and interpreting the Hewett drawings, the best joints for the barn.

Mediaeval barn




Cutting the joints


Erection of barn

The young and totally inexperienced trainees quickly took to the task and became skilled at **cutting the joints**, and with the team of three adults who supervised the project on site, the **erection** of the barn.

There are no written records of the construction from the mediaeval period and so we had to do a lot of interpreting. A cursory check on the sections showed them to be highly understressed and the key to the strength is what is left after the joints have been cut.

The pegs are used in two ways: firstly as Shanks and Walker recognise, they 'draw' the joint together, but as I discovered they hold the members together during construction too.

But back to the wind brace, the only joint that appears to be in tension. When the wind blows, the brace in tension allows sufficient displacement to take place in the connection due to shrinkage, so as to prevent a tensile force developing. At the same time this transfers the load into the brace in compression and the dowels are not loaded in shear.

In the 600 or so years of the flowering of English mediaeval carpentry from the first millennia, the joints are so arranged to prevent them being in tension. It is true that in the 16th century, the designers completely lost the plot and pegs were used in shear in the knee brace connection between the brace and the principal rafter and failed.

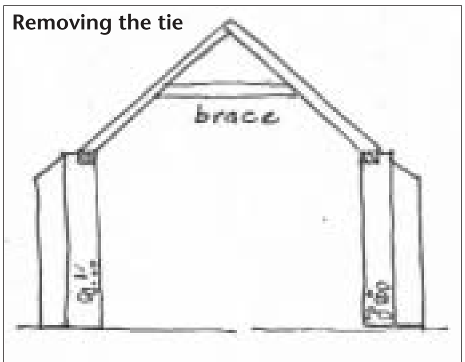
The client wanted to give a more vaulted feeling to the roof space in a church, for example and asked the builder to **remove the tie**. The first generation of builders realised that the tie force would have to be replaced and built substantial masonry buttresses in the supporting walls, co-incident with the trusses.

The next generation failed to understand the need for the buttresses, and the frames started to spread. This placed the brace halfway up the frame (usually in compression to reduce the deflection of the principal rafter) in tension. The **pegged joints** then failed.

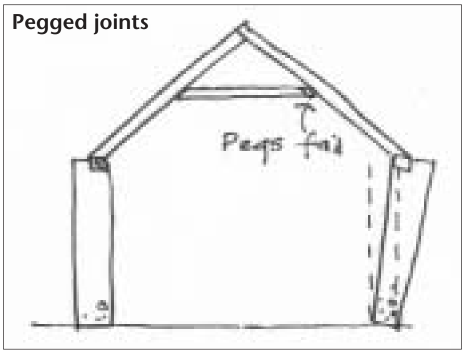
The most common attempt to strengthen the frames was to add **knee braces** with long mortice and tenon joints with a number of pegs.

If the walls were substantial this might work, but in many cases the pegs failed in shear and the knee braces separated from the rafter. All around the country you will find this type of construction with 18th century **iron straps** holding the knee brace to the rafters.

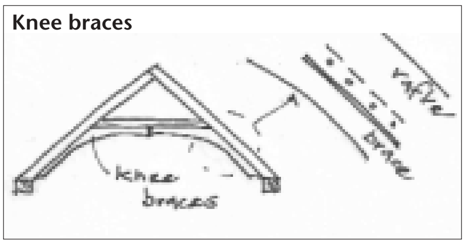
That is an understanding of the real behaviour of the structure and the technology of the construction.



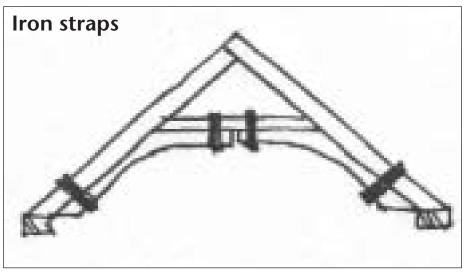
Removing the tie



Pegged joints



Knee braces


Iron straps

It is a familiar saying that 'Structural engineering is an art and not a science'. I suggest that this paper is an example of the separation of the science of structural engineering from the reality of the construction and not a good idea. There is quite enough with the misunderstandings around the use of computer software.

Authors' reply

The comments from Andrew Smith, David Yeomans and David Brohn's views are greatly appreciated. Research can often become detached and abstracted without input from unbiased professionals. However, we feel that clarifying a few issues in response will alleviate any confusion.

The research is predominantly aimed at understanding connection behaviour for the design and construction of contemporary green oak frames, where no precedence exists. Engineering design of historic frames, and specifically connections, can only be satisfactorily carried out if the condition of the pegs and surrounding timber is known. This will only be possible if all pegs have been replaced, if the frame has been dismantled for repair or if new members have been added to an existing frame.

As our research demonstrates, rotational stiffness of pegged mortice and tenon connections can be directly related to the pull-out, or 'tensile withdrawal', performance of the connections. Single 19mm diameter pegged connections have low bending resistance, however, in many situations large peg groups are used to provide significant bending resistance, such as peg groups in arched-braced frames.

A series of tests, conducted as part of this research, on full-size two-dimensional cross-frames with different brace arrangements have shown that the frames always fail by pull-out of a connection either through direct tenon withdrawal or through relative member rotation. Moreover, previous tests by Brungraber (1985), Bulliet *et al* (1999) and Erikson and Schmidt (2001) have also shown that knee-braced frames will fail by connection pull-out. Whether or not pegged mortice and tenon connections were intended to resist pull-out forces, it is clear that this is the critical mode for braced frame failure.

Knee-braced frames tested with and without tensile knee-braces have shown that the presence of the tensile knee-brace significantly increases the initial stiffness of the frame and influences the frame strength. As such, tension braces should not be discounted in the design of contemporary timber frames.

Arched-braced frames, such as Cumhill Barn roof in Pilton, resist vertical loads by pull-out resistance of pegged connections in the brace-collar and brace-principal rafter connections. Tests conducted at the University of Bath on five full-size replicas of arched-braced frames from Cumhill Barn roof have shown that frames failed by pull-out of the pegged connections by relative rotation of the principal rafter-brace or collar-brace.

Results and discussion of frame tests conducted as part of this research project will be published shortly. Furthermore, research is being continued into different joint types and frame configurations at the University of Bath.