Verulam

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Topics of importance openly discussed

Modelling stability

There have been several responses to Bill Addis' James Sutherland History Lecture paper (April 2013). Here, Bill Harvey supplies the following additional note that may be of interest to our members.

Bill Addis (in his Sutherland Lecture) follows others, notably Heyman, in commenting that one can use physical scale models to test the stability of masonry (read no tension) compression structures including arches, vaults and domes.

This is true provided the structures don't get too big or the forces too eccentric. In real scales, the latter is the serious issue. If there is a differential stress across the section and at one edge the stress is sufficient to cause creep, the outcome will be progressive bending of the structure or rotation of the foundation, or possibly both.

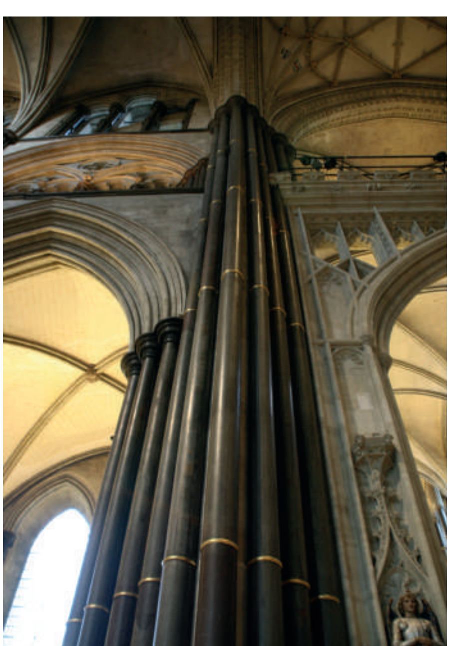

Figure 1

Stability in any large masonry structure is time dependent. Even if the structure is built with tight joints and so negligible creep, the foundations will creep if the stress gets a bit high and this will induce rotations that reduce (however slowly) stability. The bent columns in Salisbury Cathedral (Figure 1) show this action in progress.

The so-called 10 year rule may be good for 100 year structures but if movement manifests itself within those 100 years, 1000 year life becomes less certain. One cannot extrapolate over two orders of magnitude in time.

Gaudi may not have understood entirely, but by paring away his structure and following the force exactly, he could minimise the effects of creep.

I thank Bill for this information.

Precast concrete planks

In response to the publication of Technical Guidance Note 24 (Level 1) entitled 'Precast concrete planks' our regular correspondent from Malta, Denis Camilleri, has sent us the following cautionary note explaining how the form and condition of the support bearing can impact on the performance of this type of plank.

As outlined in the March 2013 issue, a good overview is given of this structural element as fitting in the overall building frame. These elements are noted as being supported on steel or concrete beams also known as non-rigid supports. For bearing on load bearing walling this is also classified as a rigid support. The minimum bearing width is given at 75mm for bearing on steel (Figure 3 in the guidance note) and concrete elements (Figure 4), whilst for bearing on masonry a minimum bearing of 100mm is given (Figure 5). However, no specification is given for the bedding mortar to accommodate the rotation at the support. Am I right in saying that when supported on a steel flange no bedding material is required?

Load tables giving safe allowable loads are produced for the various plank thicknesses and spans, for both the bare unit and a composite section with 75mm topping.

However, beside the safe load imposed on this floor element, the shear value of each plank is an important design consideration. This gains in importance when utilised in bridge decks or as transfer slabs in partitioned load bearing buildings. Figure 1 in the guidance notes shows the cross sectional shapes of panels available. The panel with circular cores normally has a lower shear capacity than the panel with elongated cores.

In a future technical note, additional guidance could possibly be provided on the following:

- Similar load tables, however this time outlining the shear values. prEN 1992-1 outlines the calculation for the shear capacity of a prestressed plank unit. Infilling of the holes towards the end of the plank achieves an increase in shear value for the plank. prEN 1168 outlines a calculation for the increase in shear value due to infilling of cores
- When precast prestressed hollow slabs are supported on non-rigid supports, tests have shown a reduction of shear resistance to these precast planks in the region of 40-77%. This has been noted due to the transverse deformation of the slab ends resulting from the deflection of the supporting beam. These beam deflections were noted to vary typically from L/1000 – L/300. This is reported in Pajari & Yang VTT research notes.

I thank Denis for this contribution. With regard to the general advice given in the March 2013 issue, to which Denis refers, it is also worth noting that care should be taken to ensure that the cores

do not become partially filled with water resulting in subsequent damage to decorated finishes.

Combination rules

Anthony Jones, writing from Manchester, has sent us the following explanation of the query concerning the combination rules given in BS EN1990 raised by Alasdair Beal in last month's issue.

I am writing in response to Alasdair Beal's query in Verulam May 2013. Whilst I agree that, for this rather unusual scenario, the code could be interpreted such that the four imposed loads could be considered as separate variable actions, I would hope that competent structural engineers would class the four loadings as one action in terms of load factors. Any other loading (e.g. snow drift, wind) would then make up additional variable actions generating additional combinations and the accompanying reduction factors.

As a colleague pointed out to me, the problem is more likely to occur when using analysis software, which is generally not written by engineers, and thus unable to make this kind of engineering judgment. Further reason to remember that design software can be a dangerous tool in the wrong hands and whose output requires detailed scrutiny.

Ray Badgery has also sent us a contribution on this subject (that has been edited to avoid repetition of Anthony's comments).

Based on the information contained within the Manual for the Design of Building Structures to Eurocode 1 and the basis of Structural Design published by the Institution (see clause 2.10.3: Combinations of actions):

You take the highest (leading) imposed (variable) load, in this case the 4kN/m^2, and use the 1.5 factor and apply the (0.7 x 1.5) factor to the other imposed loads.

I believe I have interpreted this correctly!

My thanks to both Anthony and Ray for their comments and interpretation of the rules.

And last but not least on this topic, Nick Eckford has sent us his views on the practical application of the combination rules.

Alasdair Beal is, perhaps, looking for answers where there are none. The code writers do not try to solve all the engineers' problems, especially the unlikely ones, but give us a set of rules and recommendations by which we can solve our problems.

As we know, the point here is that the ψ factor reflects the probability that loads do not peak simultaneously. We also know that the values we use for live loads (sorry, variable actions) are set such that virtually all situations of that load come within the value. We know that the reality of real life is that live loads do not reach that value. The only exception I have come across is someone wishing to install a waterbed in their bedroom; that was on the limit. So the reality is that the actual applied loads are somewhat lower than the values we take. Arguably the $\psi 0$ of 0.7 sets the value near to the statistical mean for the loads.

Looking at Alasdair's problem, it is clear that of the load values for the various areas are very unlikely to reach their design values simultaneously. In fact they are very unlikely to reach their design values at all. The code gives us a way of recognising this and permitting more economical design.

Haiti

Keith Lawrence has sent us the following contribution in response to Grenville Philips' Viewpoint article (March 2013) on earthquake resistant construction in Haiti.

Grenville Phillips' concern for the protection of cultural tradition in reconstruction following the earthquake devastation in Haiti is to be warmly applauded. It is a very important consideration alongside the fundamental objective of protecting the safety of occupants in the event of another terrible disaster such as that recently experienced.

Experience from elsewhere shows that perhaps the greatest cause of serious injury or death to the occupants of buildings comes from the collapse of concrete floors or roofs directly onto the people inside. So, given that the building, however designed, will be badly damaged by the earthquake, a primary objective must be that enough support to floors and roofs is retained to significantly reduce the risk of their collapse, and that these floors/roofs are constructed from lightweight materials, which will be less damaging should collapse occur.

"LIGHTWEIGHT ALTERNATIVES SHOULD BE IDENTIFIED. SIMPLE, LOCALLY AVAILABLE MATERIALS SHOULD BE PREFERRED"

In the case of confined masonry, the framework will remain largely intact after the masonry walls have collapsed and will continue to offer support to the floors/roofs. But the effectiveness of that support will also depend on the quality of jointing between the floor/roof and the framework. It is a matter of good design and detailing.

Reinforced masonry walls may offer a greater resistance to local collapse, but once they have failed there is presumably no residual support to the floors/roofs, unless some skeletal structure remains to fulfil that task. It seems that a skeletal structure, with enough strength and articulation to resist the shock loads generated by the earthquake, is an essential element of design. Detailing is critical, the most important characteristic being structural continuity at the joints, in all directions. Structural continuity between the beams supporting the floors/roof, and the floors/roofs themselves is also critical.

Perhaps the answer lies in the simultaneous use of both concepts i.e. the provision of a structural framework designed to remain intact after severe earthquake loading, together with reinforced walls, which by their nature, should be more resilient to earthquake generated shock loading.

The structural framework needs to be designed to cope with the induced loads of the ground shock wave. The frame doesn't have to be reinforced concrete. For low rise buildings a lightweight steel frame, embedded in the masonry and integrated with the roof and floor structures, can possess sufficient articulation to cope with the forces and the movements generated, so that it remains intact even after collapse of the masonry.

Walls should be constructed so that they are predisposed to collapse outwards, and not inwards onto the building occupants. This is a matter of detailing, and could be achieved by simply incorporating a light steel mesh on the inside face of the masonry, embedded in the finishes, and securely attached to the structural framework.

Interesting 'architectural' roofs don't have to be concrete. Alternative lightweight materials are available (GRP, thin gauge steel etc) which can achieve good visual results, without the inherent danger of collapsing concrete.

Concrete floors should be avoided for the same reasons. Lightweight alternatives should be identified. Simple, locally available materials should be preferred.

In conclusion, such structures respect the architectural heritage, offer much greater