The Structural Engineering Technical Expert – What does he do?

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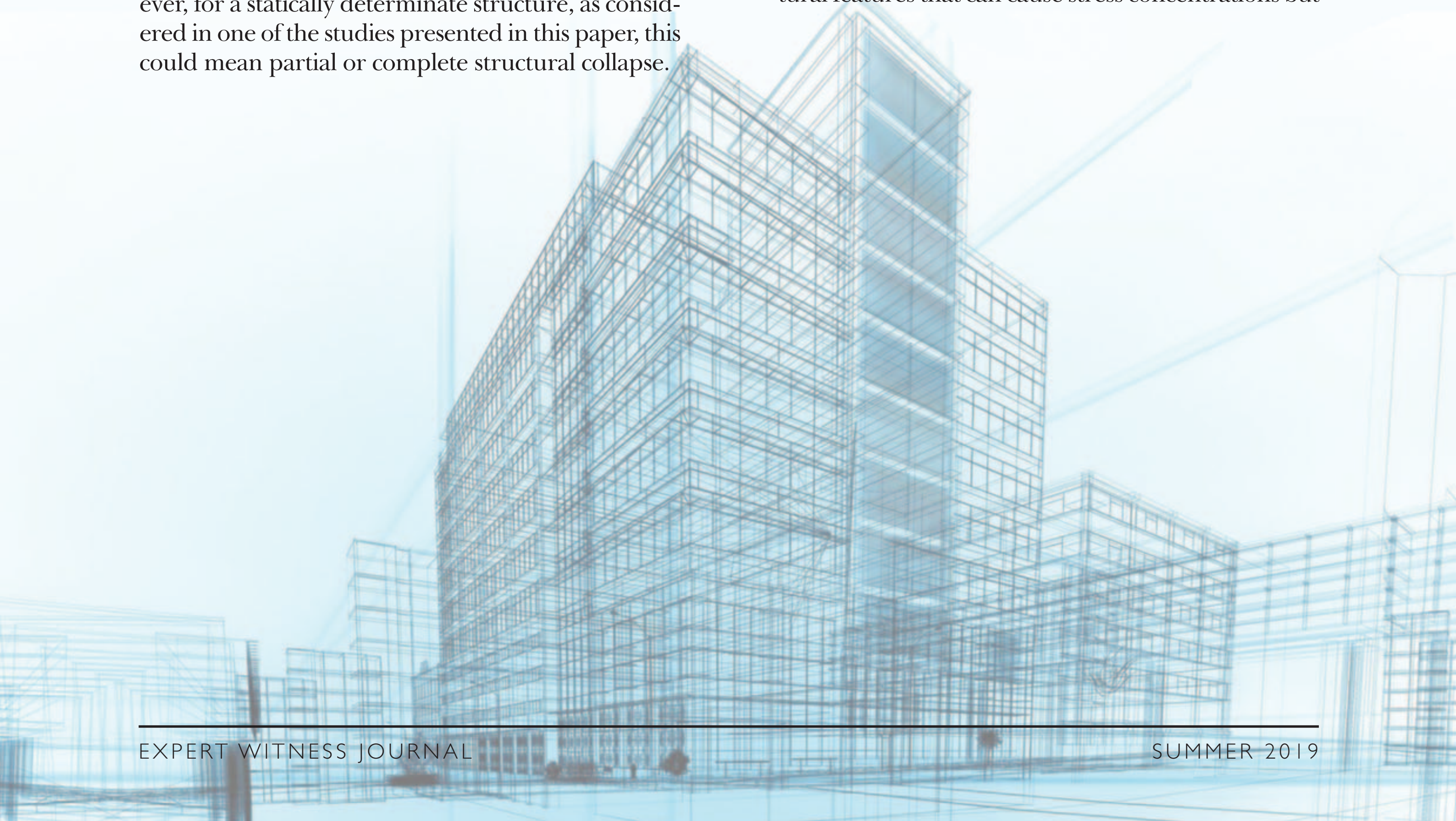
Introduction

To the uninitiated, engineering might well be a black art. How on earth do people design and build such structures as bridges, and how do they know that they will stand the test of time? This opinion might be reinforced by the regular failures that sadly occur with such structures. In reality, though, the bridge will have gone through a design process which will have used analyses to size the various structural members so that they have sufficient stiffness and strength to withstand any foreseen loads that the structure might see during its life. It should then have undergone a detailed design process whereby the various details in the design, e.g., bolted and welded joints, are investigated for such matters as fatigue resistance. Even seemingly static structures as bridges do see time-varying or transient forces due, for example, to traffic and wind, that induce oscillatory stresses about the mean value which have the potential, through fatigue, to initiate and grow a crack and ultimately fail. These, too, need to be considered at the analysis stage. In a statically indeterminate structure, one that possesses redundancy through multiple load paths, failure of one member may not necessarily mean structural collapse. However, for a statically determinate structure, as considered in one of the studies presented in this paper, this could mean partial or complete structural collapse.

Most people understand that stress is simply force divided by area and that for a safe design the stress needs to be limited to some value. However, unless the structural member or machine component is extremely simple, determining the stress is not a straightforward task. The engineer can begin the process of stress analysis by considering how the load applied to the member or component transfers through it to the supports. Beam theory, a staple undergraduate subject, is often useful here. However, whilst a structural member might well actually be a beam, it may well have details or features, e.g., holes, reinforcement etc., that require it to be considered as a three-dimensional continuum, at least local to the feature, in a manner similar to that required for a machine component.

Engineers like to simplify problems to those that have known theoretical solutions for the stresses. The plate with a central hole under a uniform tension field is a nice example. It shows that the stress is concentrated around the hole by a factor of about three, i.e., the peak stress is about three times the ambient tensile stress and that this figure is more or less independent of the size of the hole! There are many other structural features that can cause stress concentrations but



with no known theoretical solution and so if the engineer is to avoid the possibility of early fatigue failure at one of these points, he/she must be able to predict with reasonable certainty the stress concentration factor.

Whilst there is a plethora of published data on stress concentration factors likely to occur at particular design features, they usually apply for a given, and often idealised, set of loads and supports. The true nature of the stress concentration for the actual loading/support conditions can only be explored by detailed stress analysis using a numerical or computational technique such as the finite element (FE) method.

The FE method works by discretising the component into a mesh of (finite) elements. The elements are normally of simple shapes, e.g., triangular or quadrilateral for two-dimensional, planar problems defined by the position of the vertices or nodes. Within each element the stress is allowed to vary in a defined manner, e.g., a constant or linear variation, and usually it is the nodal values of the stress that define the, as yet undetermined, amplitude of the element stress field. The elements are then assembled using continuity conditions between adjacent elements. The complete numerical model then comprises a set of simultaneous equations which, once suitable supports and loads have been applied, can be solved for the unknown amplitudes of the element stress fields. The FE solution, whilst only an approximation, does have the property that it minimises the error between the FE solution and the unknown theoretically exact solution for the problem. Thus, in a nutshell, FE is an approximate but hopefully convergent method in that with mesh refinement the FE solution should get closer and closer to the theoretically exact solution even if this is unknown.

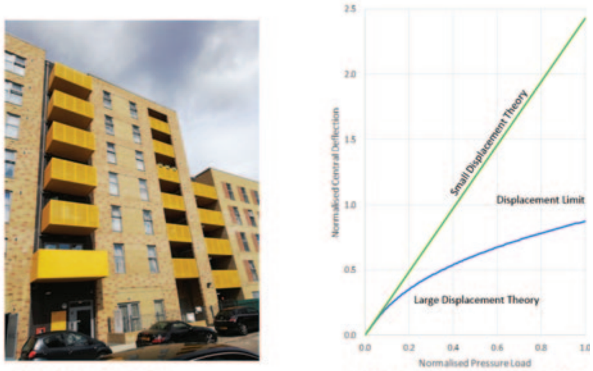
Whilst the above description of FE seems simple enough and although, nowadays, FE software is extremely easy to use, the whole subject is fraught with pitfalls for the unwary or uneducated user. I discussed in an earlier article for The Expert Witness Journal, [1], some of the issues faced by engineers when using numerical simulation techniques such as FE and outlined approaches for good practice. The engineer must always remember that Computer-Aided Catastrophes (CAC) can and do occur. The Sleipner incident is probably the most notorious example of CAC. This case is discussed further in [2] but, in essence, poor (unconverged) FE results were used to design the submerged concrete base of an oil platform resulting in an underprediction of the true stresses (by some 45%) such that the structure failed as it was being submerged into position on the sea bed. Whilst no injuries occurred, the cost of the incident was in the order of $700M. The only safe approach when using FE results is to adopt the Napoleonic Code of jurisprudence, i.e., Guilty until proven Innocent!

In this article I am going to present two short case studies of recent projects with which I have been involved. Both required FE analysis although for different reasons. Each project underwent significant verification procedures but these will not be presented here. What I would like to show with these case studies is, however, the logical processes which an engineer goes through in order to get to the essence of the problem at hand. In the first study I look at a seemingly straightforward problem of a plate supported around its perimeter and under a uniform load. My client's structural engineer analysed the plate using a traditional hand calculation and, finding the stresses to be too high, rejected the design and recommended that the plate thickness be doubled. Further consideration of the problem, and use of FE, however revealed additional reserves of stiffness and strength in the plate not considered by the structural engineer which meant that the client's original design could be shown to be perfectly adequate. This study comes from my core engineering consultancy business. The second study, which comes from a recent project where I acted as a Technical Expert, involves the collapse of a scissor lift caused by the failure of one of its structural members. The lift is actually a rather simple (statically determinate) structure and the stresses in the members can be established exactly using beam theory and hand calculations. However, the failure occurred in a portion of one of the members where reinforcement had been applied. The local analysis of this portion required a three-dimensional FE analysis to pick up the stress concentration caused by the step change in section properties at the curtailment of the reinforcement. The actual study involved providing an opinion on whether the collapse was a result of poor design or operational overload. The opinion I came to is not, for obvious reasons, discussed in this study but the process of arriving at the stresses on which this opinion was based is shown.

Case Study Number 1 – Perforated Aluminium Balustrade

This example comes from a recent project undertaken at the author's company, Ramsay Maunder Associates (RMA). The company involved in specifying, manufacturing and installing the balustrades had approached their usual structural engineer to ensure that their design met the appropriate codes of practice. The code of practice tells the engineer the loading that the balustrade should withstand and also the maximum acceptable displacement and stress. Balustrades similar to the one being considered here are shown in Figure 1(a).

The approach used by the structural engineer was to use standard tabulated data for the maximum bending moments in unperforated plates. To account for the perforations, the engineer then factored the bending moments up by the ratio of the appropriate cross-sectional area of the unperforated plate (thickness multiplied by plate width or length) divided by the cross-sectional area of the perforated plate. The factored moments were then compared with the yield moment for the 3mm thick aluminium plate to establish whether or not the plate had sufficient strength. It turned out, on this basis, that the plate was not strong enough and the engineer then calculated that the plate thickness would need to be doubled, i.e., from 3mm to 6mm, for it to have sufficient capacity.



(a) Balustrades in situ (b) Deflection as a function of load

Figure 1: Perforated aluminium balustrade

At this point in proceedings, RMA were approached to see if a more detailed analysis might be able to show sufficient strength for the 3mm plate thickness. Over the years, RMA have performed checks on tabulated engineering data and in some cases found such data to be incorrect – see, for example, reference 3. A good starting point for this project then was to check that the results used by the structural engineer were actually correct. A FE model of the unperforated plate was generated and confirmed the maximum bending moments used by the structural engineer.

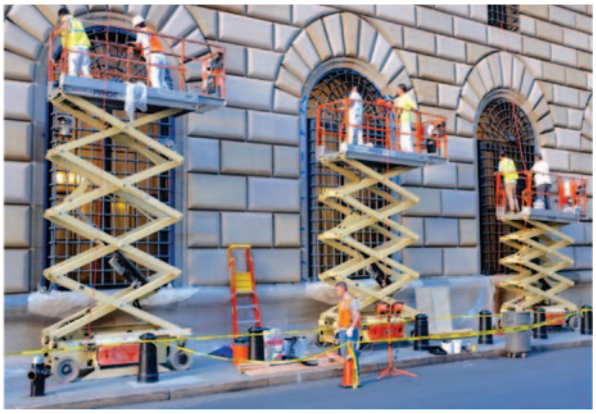
In looking at the FE results, the deflection was also noted and the maximum value, which occurs at the centre of the plate, was found to be about 2.5 times the maximum value prescribed in the code of practice! So, not only was the plate failing due to excessive stresses, it was also violating the code through excessive deflections.

Now, for an initially flat plate, transverse loads are transferred through the plate to the supports by bending actions. This is much like the way a beam transmits loads but for the plate there are bending moments in two mutually orthogonal directions. This is all well and good, but as the deflections increase, and, typically, once they become of the same order of magnitude as the plate thickness, membrane action begins to stiffen up the now deflected plate or shell. Membrane action involves forces parallel to the surface of the plate and can be particularly significant if the supports are such that the plate is not allowed to 'pull-in'. Indeed, it is the membrane action that makes an egg shell so strong – if you've not already done this, try squeezing an egg in your hand, you might be surprised just how much pressure it can take before it breaks. In order to account for membrane action, 'large', as opposed to 'small' displacement theory needs to be adopted. The problem becomes a non-linear one and is more or less intractable through hand calculation. The problem is however easily handled through FE analysis by simply switching on the large displacements feature in the solver.

The plots of displacement as a function of applied load for the plate are shown in Figure 1(b). With small displacement theory the deflection limit is reached at about 40% of the required load. The true behaviour of the plate, which is captured using large displacement theory, shows that the plate can take the full required load without reaching the deflection limit. Now, as the stress increases with increasing deflection, the stresses occurring in the plate are lower than predicted using small displacement theory and are such that the plate can be shown to be compliant with the code of practice for both deflections and stresses.

Case Study Number 2 – Reinforced Structural Member

This example comes from one of the author's recent engagements as a technical expert. The case involved a scissor lift that had collapsed early in the machine's life causing a fatality. The scissor lifts shown in Figure 2 are for illustrative purpose only and are not the same as the one considered in this case study. The author performed a structural analysis and assessment of the lift and was then able to provide an opinion whether the design was flawed or whether the operator had overloaded the lift. Whilst the failure is in the public domain, the project was executed under an NDA and so details of the lift have been changed for this article and the author's opinion on fault left open.



"High, higher, highest" by pburka is licensed under CC BY-SA 2.0

Figure 2: Images of scissor lifts in action

The scissor lift is an interesting structure in that without the hydraulic actuator it is a mechanism rather than a structure. Addition of the actuator turns it into a mechanism which can bear load due to self-weight and applied to the platform through occupants and cargo. It is what engineers term a statically determinate structure, which makes for rather simple analysis but this also means that it possesses no redundancy in case of the failure of a single member, i.e., the failure of any member will, depending on which member fails, lead to partial or complete collapse of the structure.

The basic structure can be analysed by hand calculations. The process proceeds by setting up the equations of equilibrium for each member, assembling these for the structure and is completed by solving these equations to obtain the forces applied to each member. It is a simple matter then to draw stress resultant diagrams, e.g., bending moment diagrams, which define the structural demand on each member. The design engineer would then select a member with a cross section with a capacity capable of meeting this demand. As this was an assessment rather than a design project, the author compared the demand with the capacity of the section as already designed.

The initial assessment showed that the square hollow sections (SHS) of the lift members had insufficient capacity to cope with the demand for the two members attached to the hydraulic actuator. The designer of the lift was obviously aware of this and had added reinforcement to the SHS in the regions of high bending moment. The reinforcement, however, was limited in its extent both around the cross section and along the length of the member, and led to a step change in section properties together with a fillet weld to join the reinforcement to the SHS. In a similar vein to Aristotle's view that 'nature abhors a vacuum', the engineer knows that 'structural members abhor step changes', and tend to respond by shooting the stresses up to infinity, at least theoretically!

The initial FE model used beam elements and the basic section properties of the SHS without any reinforcement. The member stresses in this model are high at the point of maximum moment and exceed the yield stress for the material at the point of interest. The point of interest is the member supporting the bottom joint of the actuator which, as a result of the offset of the actuator joint from the central axis of the adjoining member, leads to a step change in the bending moment. This can be seen in the stress contours plotted on the beam model of the relevant member in Figure 3; stresses greater than the yield stress are coloured in silver-grey.

In order to model the stresses in the region of interest when the member is reinforced, a solid continuum model was required. A local solid model was constructed and loaded with forces and moments derived from the beam model. Those keen to point out that reinforcement might lead to a different load path should remember that this structure is statically determinate, i.e., the load path is independent of the relative stiffness of the members.

The understanding of Figure 3 requires some explanation. Starting from the left we have the scissor lift modelled as using beam elements. Move to the right and we see the axial stresses in the member supporting the bottom joint of the actuator. We see a step change in the stresses corresponding to the step change in bending moment at the point where the actuator joins (with an offset) the member. The stresses in the corresponding solid model are shown offset to the right of this figure. The same contour range is adopted for both beam and solid models, and the correspondence of stress levels can be seen. In order to see the peak stresses, we must adopt a view normal to the upper surface of the member. This is shown in the figures to the right-hand side where both beam and solid model results are shown. It is seen here, in the solid model results, that the step change in section does indeed amplify the stress levels to such an extent that the region where the stress exceeds the yield stress for the material extends further than it would have done had the section not been reinforced.

The results presented above are under an overload condition for the lift and show unacceptably high stresses of the sort that might well lead to the rapid development of a fatigue crack. Reinforcement was applied to the region of high bending moment but it was not extended sufficiently beyond the region for the stresses to be brought down to a sensible level, i.e., well below yield. Under normal loading the

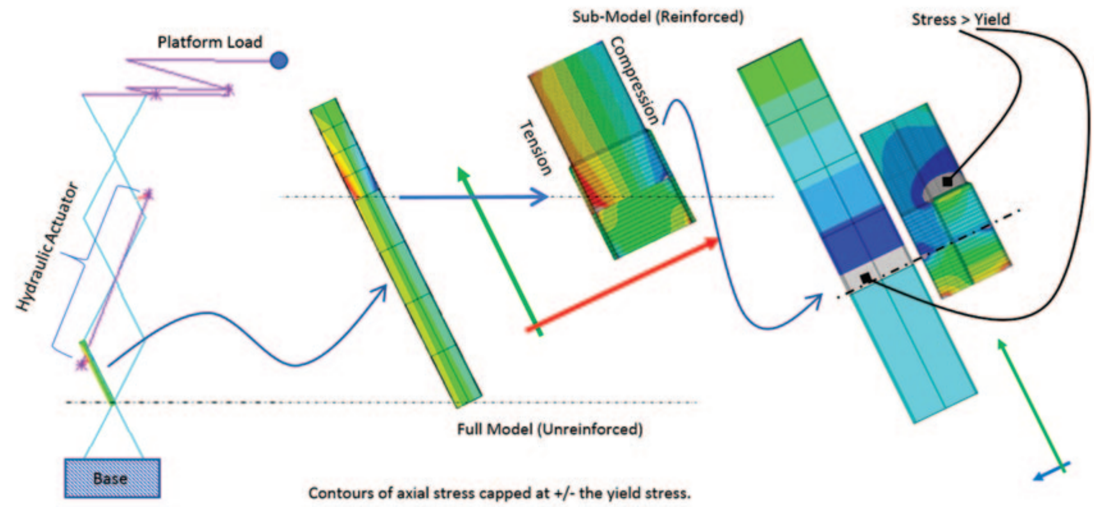


Figure 3: Stresses in a scissor lift member

stresses in this region were well below yield. It is common for mobile elevated working platforms such as this scissor lift to be overloaded and in speaking to a colleague who works in the auctioning of second-hand machinery, the author understands that hire companies generally sell their lifts off after only a year of operation precisely because they cannot guarantee that they have not been overloaded.

Discussion

Projects, such as the type described in this paper, provide challenges and are therefore extremely interesting and having projects coming from industry as well as through expert witness cases work well together with the two streams of business complementing each other.

The successful outcome of both projects required a developed understanding of structural mechanics together with a specialist understanding of how to model such problems using FE analysis so as to provide sound and robust results. Neither of these skill sets come as standard with a graduate engineer and, on the whole, require a mature and experienced engineer to be able to tackle such problems reliably.

The author has developed these skills and specialisms over a thirty plus year career working both as a mechanical engineer at the sharp end of design and analysis of turbomachinery and also as a structural engineer in the nuclear industry assessing structures for structural integrity under the simulated action of earthquakes. One of the key skills required in acting as a technical expert or as an expert witness is the ability to explain clearly and concisely what your opinion is and how it was obtained. This often requires distilling rather technical matters into a pithy and well-illustrated argument that is understandable by an intelligent layperson. This may, of course, need to be done orally at a hearing, but possibly more importantly the product of a technical expert is generally a report in a written legal format. In presenting the case studies in this short article, the author has attempted to use this distillation process and it is hoped, as the reader, you will understand and find a new realisation of what an engineering technical expert actually does.

In closing this article it should be noted that structural engineering and the analysis of stresses in both structural members and mechanical components is just one field of activity that typifies engineering practice.

References

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