The benefits and use of FE modelling in bridge assessment and design

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ABSTRACT: Structural analysis has progressed a long way from hand calculations and from when distribution factors first started to be used. From the first computer methods, grid and grillage analysis techniques evolved, leading through progressive enhancements to the advanced 3D graphical and analytical tools that we see and use today. Finite element (FE) modelling and analysis is being used more for bridge engineering because of the more economical and accurate assessments and designs its use produces. This paper illustrates the role that it can play in just some areas of bridge analysis, assessment and design with reference to bridge assessments and designs carried out by consultants on projects around the world.

1 BENEFITS AND USES OF FINITE ELEMENT MODELLING

For the evaluation and assessment of existing bridges, and also for new design, FE modelling allows for a more rigorous analysis approach to be adopted that can often lead to significantly more accurate and economical results being obtained over some codified methods. In the past, structural design codes such as those from the British Standards Institution allowed for a departure from a 'codified' approach. Others, such as the newly introduced Eurocodes have been more prescriptive often mentioning that FE analysis should be carried out.

When the structural components of a bridge do not comply with assessment code criteria, FE analysis can be employed to help to prove the integrity of the design.

When combined with bridge monitoring, the use of key measured structural data to effectively fine-tune and calibrate an FE model can lead to even greater accuracy in the results obtained for a subsequently loaded assessment model.

Assistance with problem diagnosis and the development of retrofit solutions are other ways in which FE analysis can assist greatly by allowing what-if scenarios to be modelled. By using FE modelling structural members can be optimized and innovative cost-saving designs obtained.

For particular applications, and using FE methods, automated model building can guarantee correctly-built models in accordance with design code criteria; the generation of critical vehicle loading arrangements and analysis of the effects of the loading on a structure can be rapidly achieved; and design checks such as those required for steel./composite bridge decks can be made easier, faster and more accurately than by manual methods.

Once designed, bridge erection engineering using FE analysis can assist in the preparation of erection manuals and provide contractors with accurate setting-out data and, for cable stayed structures, cable tensioning values and construction sequences can be proved. Note that demolition analysis may also require staged erection analysis to be carried out in order to accurately model historical repairs and derive any in-built stresses prior to disassembly.

2 CASE STUDIES

The following illustrative case studies provide an general overview into some of the many roles that FE modelling and analysis can play in bridge assessment and design.

2.1 *Comparison with design codes*

UK consultant, Atkins, used FE analysis to model a pair of steel beams during concrete placement, prior to the concrete slab providing lateral restraint to the beams. (Hendy 2008) For this situation the recently introduced Eurocodes give no formula to derive the critical bending moment. One span was loaded with wet concrete such that the lateral torsional buckling would govern the resistance of the beam group. From an eigenvalue buckling analysis the critical buckling moment was seen to be caused by the 20th mode, but at a load factor 50% greater than that predicted by BS 5400. A full nonlinear analysis using LUSAS carried out for the same paired beams with material behaviour based upon the Eurocode recommendations, and with initial imperfections based on the elastic critical buckling results, gave even better results, almost doubling the load factor predicted by BS 5400. (See table 1).

Figure 1. A pair of braced steel beams.

Table 1. Comparison of critical buckling load factor values for codified FE analyses of a pair of braced beams

Calculation method	Load factor (at failure)
BS 5400 Part 3	1.0
Elastic analysis and EN 1993-1-1	1.55
Nonlinear analysis using LUSAS	1.99

2.2 *When design codes can't be used*

When the diaphragms or geometry of a steel box bridge do not comply with assessment code criteria, FE analysis will allow a detailed analysis to be performed in order to help prove the integrity of the design.

Typical of many similar elevated and ageing structures built in the 1960s, the Midland Links Viaducts carry the M5 and M6 Motor-ways around Birmingham in the UK. A number of spans are supported on steel box girder crossheads (see Figure 2) and contain strengthening details, added in the period following publication of the Merrison Report, which were not easily assessed using codified methods. Maunsell (now AECOM) undertook detailed nonlinear FE analysis and proved the integrity of the diaphragms at the ultimate limit state [5]. Initial hand calculations to the methods in BS 5400 Part 3 indicated that panels within the support diaphragms of these box girders would yield below ultimate limit state loading. Additional analysis suggested that the intermittent welds between the diaphragm and the vertical stiffeners were also liable to yield. A linear elastic FE analysis confirmed this, and a detailed materially and geometrically nonlinear analysis was undertaken to prove the integrity of the diaphragms at the ultimate limit state.

Thick shell elements modeled the steel plates and elastic / perfectly plastic joint elements modeled diaphragm stiffener welds. Yield forces for the joints were set so that the resultant forces in the joints were limited to values corresponding to the weld yield stress predicted by assessment code BD21/97. Joint stiffnesses were chosen so that onset of yield in the joint elements corresponded to a resultant weld deformation of no more than 0.10mm, a value supported by research evidence.

The extent of yield within the structure was identified at each load increment and animations of the deformed shape and stress contours plotted at each load increment showed how the diaphragm redistributed load as it approached its limiting strength. Out of plane nodal displacement histories for nodes within the diaphragm were plotted against total load factor to confirm that buckling was not appreciable. The movement of the joint elements was output at each load increment and a spreadsheet was used to plot graphs of the displacement profile along the weld line each load increment. These graphs proved that the deformation of the welds would not exceed the limiting value of 1.0mm set as a safe limit, and showed AECOM that potentially difficult strengthening work was unnecessary.

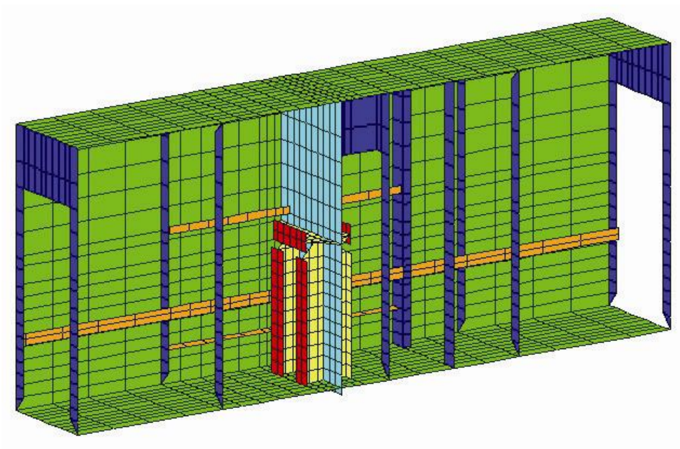

Figure 2. Crosshead modelling on the Midlands Links Viaducts

2.3 *Load rating of truss and through bridges*

The use of FE analysis for accurate load rating is a key benefit over codified approaches.

Chicago-based consultant, Benesch, was retained by Michigan Department of Transportation to perform load rating of two cantilevered steel deck truss bridges. One, the US-2 over the Cut River, was built in 1947. The other, the M-55 over Pine River, was built even earlier in 1934. For each, the main structure consists of 600 ft truss supporting a concrete deck with non-composite action via stringers and floor beams. Because of the bridge complexity and the existence of internal hinges, a finite element modelling technique was used to carry out a load rating for the two bridges to evaluate their structural capacity. Live load analyses were performed using HS-20 trucks in addition to Michigan 1- Unit, 2-Unit, 3-Unit legal vehicles, and MDOT Classes A, B, and C overload vehicles. Inventory Rating, Federal Operating Rating, Michigan Operating and Michigan Overload Class were computed for each bridge. For each bridge, two models were developed; one using LUSAS software, the second using alternative software to verify the results. In addition, hand calculations helped verify both sets of software results. For Cut River Bridge performing live load analyses using the FE method considerably reduced the live load forces on the truss members when compared to hand calculations and saved MDOT the unnecessary costs of retrofitting the bridge.

Figure 3. Cut River Bridge.

In the mid to late 1990s all bridges in the UK needed assessing for a 40 tonne weight limit in order to meet new proposed European legislation. As part of its bridge assessment program Railtrack plc (now known as Network Rail) required a 3 span skewed structure at Hackbridge, London to be assessed. (LUSAS 1997) The bridge consists of two brick arch approach spans and a central 12m span comprising steel edge girders. These girders support brick parapets and main steel troughing spanning between the masonry piers and onto their bottom flanges. An initial simple assessment identified a possible deficiency and a more rigorous analysis was commissioned proving its capacity. Mostly this type of work is done with a linear buckling analysis but occasionally nonlinear buckling analysis has proved necessary. Historically steel 'through' bridges like this have been widely used throughout the UK. Many UK consulting firms and regional authorities have used FE analysis to prove 40 tonne vehicle capacities on these types of structures that, according to the code, had insufficient lateral restraint and would have required a severe weight restriction or even closure.

Figure 4. Bridge at Hackbridge

2.4 *Calibrated models*

The use of key measured structural data to verify, calibrate and sometimes even fine-tune an FE model can lead to greater accuracy in the results obtained for a subsequently modified assessment model incorporating proposed changes.

In making a loading assessment, a bridge is typically first rated using conventional structural analysis methods using code specified load distribution factors. These methods generally require beams or slabs to be idealized using simple beam or grillage elements supported on idealized supports meaning that a variety of effects such as edge stiffening or partial composite action are not explicitly addressed. If load ratings from simplified methods do not meet desired requirements FE analysis can be used to obtain more accurate solutions. Then, if the results obtained from initial FE modelling are not thought to fully represent a structure's behaviour, field test data can also be used to calibrate and fine-tune the initial FE model. Whilst distribution factors are useful for a quick 'first assessment of a structure, and can also be used to verify more sophisticated modelling techniques, they are also over-conservative. In a paper presented at the Structures Congress in Texas (Catbas & Gokee, 2009) the use of AASHTO distribution factors was found to be typically 25-40% over-conservative in an analysis of 40 bridges that were studied for comparison with analysis using FE models. A detailed study on one bridge showed a bending moment of 1452 kip.ft (using AASHTO factors). This was brought down to 1003 kip.ft using modified factors, and further reduced to 553 kip.ft using calibrated FE models. Refined load rating analysis of a variety of in-service bridges in North Carolina (Das, 2010) using a combination of finite element analysis and field load testing saw controlling load rating factors for particular rating vehicles increase by up to 75% and in one case, for a bridge that had seen previous repairs made to its deteriorated condition, a decrease of up to 25% in its load rating factor.

On the West Gate Bridge Strengthening project both local and global FE models were developed to allow the existing capacity of the steel box girder section of the bridge to be assessed and provide an indication of the amount of strengthening that would be required to achieve a desired loading criteria. (Taylor 2009) A detailed shell element model was used for the majority of the steel bridge assessment work and the original construction sequence was also modelled. FE model-predicted modes and frequencies were compared with those obtained from measurements of key data from the existing structure, and very good correlation was obtained.

2.5 *Development of retrofit solutions*

Bridge assessments requiring retrofit solutions are another area where FE analysis can assist greatly with what-if scenarios. In the US, URS investigated a deck truss bridge of two simple spans of 218 feet where each span had a lower lateral bracing system that was found to exhibit excessive vibration under truck crossings. (Zhou & Biegalski 2008) Fatigue cracks were found at the gusset plate connecting the lower lateral bracing diagonal to the truss bottom chord. In-service monitoring was performed to measure the vibration properties of the truss and the lateral bracing members. The measurement results were correlated with the calculated natural frequencies of the structural system and the excitation frequency of crossing vehicles. From this it was determined that the vehicle passages between floor beams corresponded to observed vibration frequency of the lower lateral members. Based on the results of in-service monitoring and a range of FE analysis studies to ascertain the best structural solution, a suitable retrofit was developed to increase the natural frequency of the lower lateral system to avoid resonance with the excitation frequency.

Figure 5. Lower lateral bracing system on truss bridge

2.6 *Innovative new design*

Gateshead Millennium Bridge is a striking arched opening bridge designed by Giffords. (LUSAS 2000) Made of steel, and designed with the aid of FE analysis, the bridge stands 45m high and spans 105m across the River Tyne in Newcastle, UK. The 130m long deck is parabolic in elevation and of steel box section that tapers in plan towards the centre of the deck. It carries a pedestrian footway that varies from 3m to 5m in width as well as a 2.5m cantilevered cycleway. The main arch is also parabolic in shape and tapers both in plan and elevation. Whilst small river craft can sail beneath the bridge, for larger craft the cable-stayed double-arched structure pivots at the abutments through an angle of 40 degrees to give the 25m navigational clearance as specified by the client, Gateshead Borough Council. At the fully open position the suspension cables lay horizontal holding the pair of arches together. Huge 14 tonne castings, also designed using FE analysis, sit on either side and support bearings which withstand the outward and radial thrusts imposed. Using FE analysis was essential on this project to model the staged construction sequence, the lifting of the bridge into position, and the opening and closing sequence.

Figure 6. Gateshead Millenium Bridge, UK.

Far larger in scale, and requiring numerous complex detailed analyses, Modjeski & Masters Inc. designed a proposed New Mississippi River crossing that, had it been built, would have been the 5th longest cable stayed bridge in world. (LUSAS 2004) At 222 feet (68m) in width, the bridge would also have been the world’s widest cable-stayed structure. A staged construction analysis modelled an 800 day construction period and continued to 10,000 days to allow for creep and shrinkage over that period. Modjeski & Masters believes that this was the first proper 3D analysis of the staged construction of a cable stayed bridge, including creep and shrinkage, carried out in the USA.

Figure 7. The initial proposed new Mississippi River Bridge.

2.7 *Cost-Saving re-designs*

The Estero Parkway Flyover in Fort Myers, Florida, is an example of the role that FE analysis can play in saving money. Finley Engineering Inc. (Finley) redesigned the flyover for the contractor, ZEP Construction. (LUSAS 2007) In doing so, Finley's four-steel box girder design produced significant savings in construction costs over an initially proposed cast-in-place concrete box girder design. It was a good example of what can happen with value engineering when the owner, contractor and engineer come together to create a design that takes the contractor's strengths into account and that utilizes the best material and the most appropriate software for the challenges of the project. In this case the redesign from concrete to steel had an overall positive effect on the cost, schedule, and efficiency of the bridge, and the use of FE analysis helped Finley to meet its design deadline and to prove an alternative bridge design that will ultimately save the client nearly $3million in project costs.

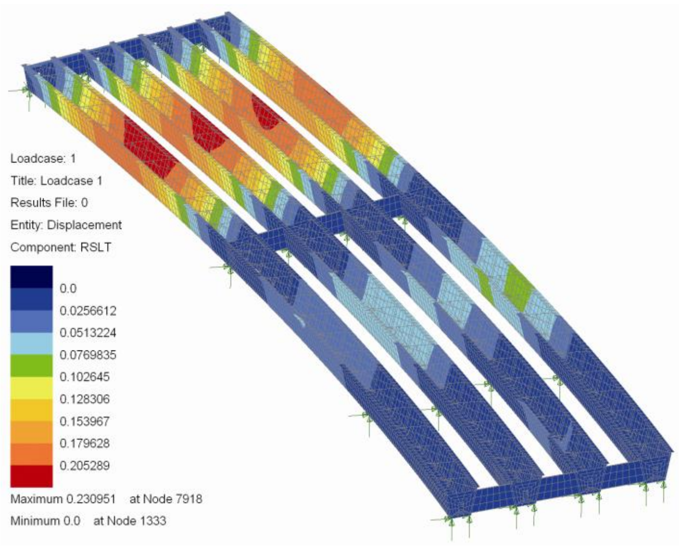

Figure 8. Estero Parkway Flyover: Resultant displacements from deck pour loading.

2.8 *Automated modelling and loading*

For high speed and light rail infrastructure projects the checking of design values against the International Union of Railways Code UIC 774-3 is often a requirement. The passage of one or more trains crossing a rail bridge causes forces and moments to occur in the rails that, in turn, induce displacements in the supporting bridge deck, bearings and piers. Application-specific finite element analysis software allows for accurate modeling of the interaction of the track with respect to any supporting bridge structures, and in particular, will help to ensure that any interaction between the track and the bridge as a result of temperature and train loading is within specified design limits. For this type of analysis, FE models are typically automatically built from data defined in Excel spreadsheets and results in summary, tabular or graphical formats can be obtained for all or selected parts of the track/bridge model for checking to specified design criteria. Automated model building guarantees correctly-built models compared to manual model creation and with the correct software the nonlinear material properties associated with the track/structure interface will be automatically updated according to the position of the passing train or trains. Eurocode EN 1991-2:2003 Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges encompasses significant elements of the UIC 774-3 modelling approach when evaluating the combined response of the structure and track to variable actions. For a UIC60 rail, the limiting design criteria are the same as those specified in the International Union of Railways Code UIC 774-3 meaning that Rail Track-Structure Interaction analysis software can also be directly employed to meet Eurocode requirements. Overall, use of this type of FE software provides a much faster assessment of thermal and / or train loading track interaction effects on multi-span structures than all other known methods.

AG	AH	AI	AJ	AK	AL	AM	AN	AO
Envelope - Temperature and Train (Max)								
	Maximum				Minimum			
	Value	Element	Node	Dist (m)	Value	Element	Node	Dist (m)
Disp X (m)	0.00943526		3074	606	0		1	0
Disp Y (m)	0.00039598		1286	314	-0.0005951		2204	464
Rot RZ (rad)	0.00021636		3182	624	-0.0001123		2740	551
Rel. Disp of Railbed over Decks (m)	0.00994447		2729	550	-0.0073356		3188	625
Rel. Disp of Railbed whole Track (m)	0.00994447		2729	550	-0.0073356		3188	625
Fx (N)	564894.989	1015	2028	435	-686322.12	1591	3182	624
Fz (N)	8434.29029	1362	2723	549	-5764.6313	1367	2729	550
My (Nm)	3204.85889	1594	3188	625	-560.17577	1359	2717	548
Rail Stress (MPa)	36.8276075	1015	2028	435	-44.743894	1591	3182	624

Figure 9. Typical Rail Track-Structure interaction results .

For road bridges, identifying critical vehicle loading patterns for a particular design code is one area where the use of FE analysis software can help enormously. Vehicle load optimization (as it is commonly called) usually involves the definition and interrogation of influence lines / surfaces to calculate a critical loading pattern for a particular desired effect such as maximum reaction, moment or shear at a certain point. Its use rapidly reduces the amount of time spent generating and loading models, produces worst-case traffic load effects more easily and much faster than by manual methods, and should lead to a far more efficient and economic design, assessment or load rating of a bridge. In the EN1991-2 Recommended Values example shown (which is for a maximum reaction at the first inner support), Group 5 loading is seen to dominate with the load pattern made up of LM3 with associated LM1 tandems and udl patches. The SV1800/200 loading is positioned adjacent to the influence definition, near the edge of the deck over spans 1 and 2. This is considered to be Lane Number 1 (EN1991-2:2003 Annex A clause A.3). Lanes 2 and 3 appear on the far side of the deck and are loaded only in the adverse area which happens to be in span 3. The 1m wide remaining area appears between lanes 1 and 3, illustrating yet another arrangement of lane rank and location, used to generate the most onerous traffic load pattern.

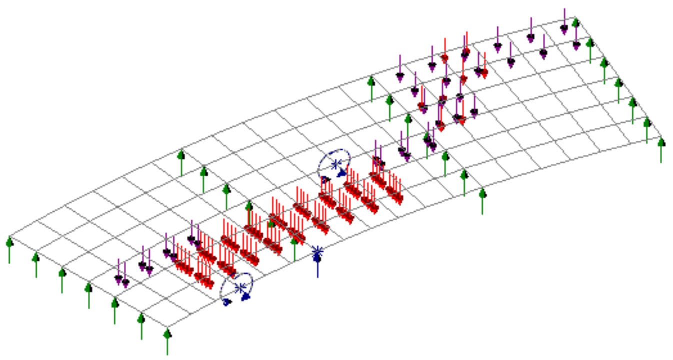

Figure 10. Example of an optimized vehicle loading pattern .

2.9 *Design checks*

For the design of composite steel and concrete structures, EN 1994 makes reference to many other parts of the Eurocode suite, meaning that carrying out design checks can be an extremely time-consuming and potentially error-prone process. As a result, the use of specialized checking software in conjunction with an FE analysis is pretty much mandatory if a design is to be optimized in a reasonable amount of time. To give an example, SPEA Ingegneria Europea spa engineering company, which as well as carrying out a comparative analysis between the former and the new Eurocode-based codes (Ferretti Torricelli, L. et al. 2010), also investigated and optimized the design of a 38m single span, steel-concrete composite integral bridge using geometry, material, moment and shear data generated from an FE analysis. Specialist design software Ponti EC4 - which carries out Eurocode design calculations for steel-concrete composite bridge decks - was then used to carry out design calculations for ULS bending, stress, shear and interaction; SLS stress, web breathing and cracking and fatigue checks for the main structural members and connectors. Optimized steel beam flange, web, and reinforcement bar sizes were obtained with only the reinforcing steel in end regions of the slab needing to be increased due to the hogging bending moments produced by the connection between deck and abutment. For each limit state that was checked graphs of utilization factor along a beam could also be produced. Overall, a saving of nearly 25% in structural steel weight was obtained with respect to preliminary design calculations.

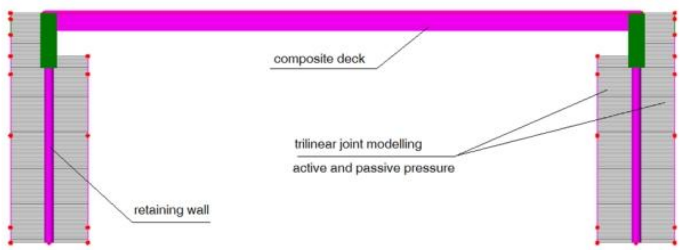

Figure 11. Model of integral bridge designed to EC4.

2.10 *Erection engineering*

Erection engineering, carried out by consultants on behalf of contractors is another aspect of FE use. On the HNTB-designed Bagley Street Pedestrian Bridge - a 417ft (127m) long, two-span asymmetric cable stayed structure (as shown in Figure 10) - the contract documentation placed design limitations on the 155ft (47m) tall pylon restricting the horizontal displacement at its top to two inches (50mm) and also restricted the maximum amount of bending moment about its weak axis at a specified level at its base to be 800 kip ft during its construction, unless any exceeded values could be proved safe (LUSAS 2010). By carrying out detailed finite element modelling of the bridge for all stages of construction specialist erection engineering consultant, Genesis Structures, was able to show that the effect on creep from the actual pylon deflection was acceptable, and that the overall moment capacity about the weak axis of the pylon was sufficient to resist the actual bending moments seen during construction. From the staged erection analysis carried out the proposed erection sequence was proved, cable tension time-histories for the temporary and permanent cables, and reaction time-histories for all abutments, bearings and falsework supports were obtained, and three-dimensional target coordinates and elevations were provided for the contractor's use for key points on the structure, including at the pylon stay housing, at temporary shoring, at box girder splices, along the box girder deck and at all stay cable connection points.

Figure 12. Bagley Street Pedestrian Bridge.

2.11 *Demolition engineering*

Demolition analysis may also require staged erection analysis to be carried out in order to accurately model and incorporate all renovations made to the structure during its lifetime. For the self-anchored Paseo Bridge in Kansas City, Missouri this included modelling an additional wearing surface and the replacement of the main stiffening girder bearings (LUSAS 2011). As a result of this modelling process the displacements and stresses seen during the bridge's construction could be appreciated and the forces in the main cables and hangers could be obtained for its final in-service condition. A survey of the existing structure confirmed the accuracy of the FE modelling. An additional model was developed to assess the effects caused by lowering of the main suspension cables and another model investigated detailed stresses and effects upon the pylon base and anchor bolt system. All analyses proved and reassured those involved that the intended demolition sequence could be undertaken safely.

Figure 14. Demolition of Paseo Bridge

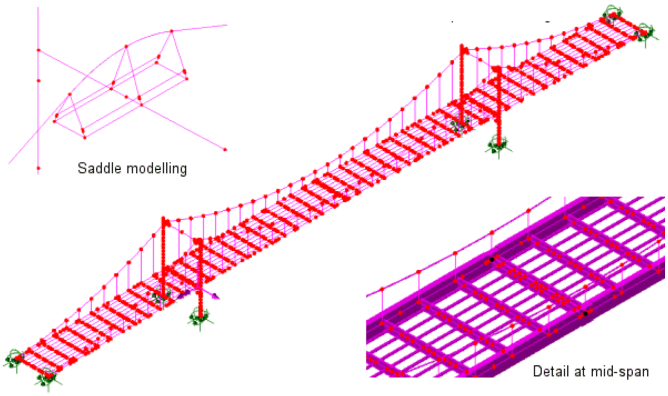

Figure 15. Demolition modelling of Paseo Bridge

2.12 *Conclusion*

Structural analysis has progressed a long way from when just hand calculations were used. Today's range of state-of-the-art finite element analysis software is being far more widely used to optimize new bridge designs as well as carry out more accurate load rating of existing structures. FE modelling has always required a fundamental understanding of structural behaviour and users will always need to know what limitations there are with any particular modelling technique. The use of FE analysis can give better, more economical, results when compared to 'codified' solutions. Properly targeted and supported FE analysis software tailored to the construction industry should always be used and allow for more advanced analyses such as nonlinear buckling or dynamic assessment to be carried out where necessary.

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