CROSS

Confidential Reporting on Structural Safety

For an introduction to CROSS see www.structural-safety.org. Email: structures@structural-safety.org

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Reports In this Issue:

Reports sent to CROSS are de-identified, categorised, and sometimes edited for clarification, before being reviewed by the CROSS panel of experts. The panel makes comments that are intended to assist those who may be faced with similar issues. In the Newsletters the reports are shown in black text and the comments are shown below these in green italics. Reports and comments are also given on the web site data base.

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Introduction

Several reports in this Newsletter express concern about aspects of structural steelwork. Taken one after the other these may give the impression that there are many issues with steel design and fabrication, but this is not the general case because the vast majority of such structures are properly designed and constructed. CROSS reports are gathered together where there are similarities so that trends can be detected. Many of the problems highlight the importance of carefully selecting both consultants and steelwork contractors. Capabilities to look for in fabricators include the type of building that can be fabricated, whether or not firms have adequate quality management systems in place, their sustainability credentials and a guide to contract values they can undertake. Checks should also include their CE Marking capability and to reinforce this BCSA has introduced a new policy so that from 1st July 2014 only steelwork contractors with a certified CE marking system in place can be members. However there must also be greater awareness within the wider construction industry of the problems that can occur and steps taken to avoid them. As ever, any failure or collapse (putting aside liability issues) causes great disruption and distress and if there is a fatality the ramifications change the lives not just of the victim but of all involved.

When trends are identified then organisations that have influence are informed and can include groups, such as BCSA in this case, Institutions, Government Departments, and Local Authorities. If a change in recommendations or procedures is then made the circle is complete – from report to result.

More reports are needed all the time and the success of CROSS depends upon it. Individuals and firms are encouraged to participate by sending concerns in confidence to http://www.structural-safety.org/

395 Partial roof collapse at shopping centre

There was a recent partial roof collapse at a Shopping Centre which was, says the reporter, built in the early 1970s in a form which may have been used extensively throughout the UK during this period and possibly beyond. Luckily, due to the timing in the early hours of the morning, this roof collapse did not claim any loss of life, but has raised considerable concerns. The failure mechanism was ascertained from site visits but the underlying cause is not known, and the report was submitted in the hope that it will be useful in the understanding of the mode of failure and in the identification of similar connections which may occur in other buildings.

Collapsed roof

What should be reported?

- concerns which may require industry or regulatory action
- lessons learned which will help others
- near misses and near hits
- trends in failure

Benefits

- unique source of information
- better quality of design and construction
- possible reductions in deaths and injuries
- lower costs to the industry
- improved reliability

Supporters

- Association for Consultancy and Engineering
- Bridge Users Forum
- British Parking Association
- Communities and Local Government
- Construction Industry Council
- Department of the Environment
- DRD Roads Services in Northern Ireland
- Health & Safety Executive
- Highways Agency
- Institution of Civil Engineers
- Institution of Structural Engineers
- Local Authority Building Control
- Scottish Building Standards Agency
- Temporary Works Forum
- UK Bridges Board

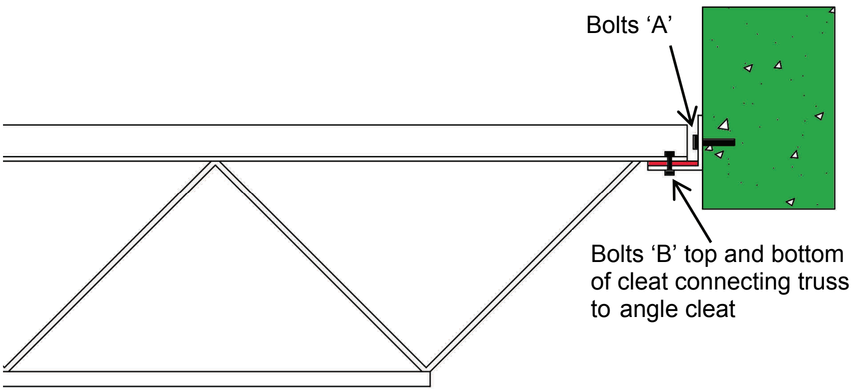
To find reports in the data base go to the ***Quick Search*** *box on any page of the* *Structural-Safety* *site and enter a subject e.g. "wall" and a list of summarised reports will follow. Searches can be refined using* ***Search data base*** *facility.*

Typical connection: cleat bolted to beam and top flange of truss rests on cleat

End fixing of truss to cleat

Failed bolt with part of shield still in place

Diagrammatic section



The building is of reinforced concrete construction, with columns supporting a grid of beams at roof level, which support the flat roof construction over shop units and a mall. The flat roof had a mineral felt weathering, on insulation board, on profiled metal decking. The metal decking spanned between steel lattice trusses, which themselves spanned between the reinforced concrete beams at roof level. The lattice girders were connected to the high level RC beams via a top flange bearing plate which was bolted to an angle cleat (bolts 'B') which was in turn bolted to the RC beams by bolts 'A'. The arrangement is shown in the photographs and the diagrammatic section. It was clear that some of the bolts used to secure the angle cleats to the RC beams (bolts 'A') had failed in tension. They appeared to have been pulled out of the concrete. A closer inspection of some bolts which were found in the debris showed that the bolts which had failed ("Drop-In Wedge Anchors" — manufacturer unknown) had done so within the shield of the anchor, within the embedded depth of the bolts. The threaded steel shields were later found to be no more than 1.6mm in thickness. The reporter did not see any evidence of corrosion, but was surprised at how thin the walls of the shells were. It appeared that it is most likely that the failure occurred as a chain reaction when a single bolt, or group of bolts at a connection failed (4 No bolts 'A' per connection). It was clear to see that the failure started in one corner, where the bolt shells had failed in tension. But further along, where the support cleats were still in position on the RC beam, the fixing bolts 'B' between the lattice girders and the cleats had failed in shear as the roof collapsed. The collapse followed a significant period of heavy rainfall. The fall of the roof towards rainwater outlets (which would dictate the "deep end" for any ponding water) should have accumulated more loading and possibly cause failure at that end if the outlets were blocked or flow restricted. However this was not the case and failure occurred at the "shallow end", the opposite end to the outlets. Remedial works to enhance the loading capacity of all of these connections were necessary.

Comments

From the limited information available it is difficult to identify the cause of this failure which apparently originated at one fixing point and resulted in a progressive collapse. What was the force that caused them to fail? The bolts pulling out may be a symptom rather than the cause and the following comments are not an explanation of what happened here but reminders of the issues that can affect a fixing such as this. The bolts attach the angle to the concrete so carry shear but also tension due to the eccentricity of the truss bearing on the angle. If the roof had ponding water and the beam was deflecting the point of bearing would move towards the edge of the angle increasing the tension significantly, failure could then have been by combined shear and tension. Normally it is assumed that bolts 'A' are in shear only but if shimmed out too much they move into shear + bending and that makes a big difference. Bolts 'A' could also be levered out to some degree depending on whether the cleat is 'stiff' enough. Given there is only one line of bolts at B, there will be moment due to the eccentricity of load between B and the concrete face. These are secondary forces that may need to be considered. Another possible cause could be corrosion of the 1.6mm steel shield although the reporter did not notice any evidence of this. Sequential temperature changes over the years or a sudden change in temperature might have had an effect. It would be helpful if anyone else has experienced similar failures and has an explanation as to why the fixings have lasted for 30 years without obvious problems.

329 Design deficiencies in long span steelwork portal

A firm was commissioned to design steel joints on a large, single-storey supermarket structure for a fabricator. Construction was underway and steelwork was due on site in a matter of weeks. Whilst carrying out a review of the design drawings the reporter's consulting firm identified two areas of serious concern and brought them to the notice of the frame designer. Additional steelwork was then added to the design to resolve the problems. The main problem was lack of restraint to a series of long span portal rafters. The frame is more than 50m wide twin span propped portal supported at the apex by a mixture of internal columns and ridge beams, hence each rafter spans over 25m. On one side of the structure there is a north-light roof, which comprises secondary rafters propped from the main rafters, this north-light continues the line of the opposite roof slope, projecting upwards from the ridge. The presence of the north-light resulted in the primary rafter having no purlins or any other restraining steelwork over a length of 12m. The section, a 533 x 210 UB, was clearly excessively slender for such a long unrestrained length and had not been designed for this situation. To make matters potentially worse, the frame could probably have been erected and supported its self-weight and the weight of the roofing, allowing the structure to be completed and potentially left in an unsafe condition. The application of, say, design snow load would have almost certainly led to collapse. A further, albeit less serious problem with lack of lateral restraint to columns was also highlighted by the reporter's firm and steelwork added as a result. The project had been checked and independently certified by a firm employing experienced engineers. This is the latest and most striking in a regular diet of defective design identified by the reporter's firm whilst carrying out connection design.

Comments

The main problem here is that the original designer did not consider lateral-torsional buckling of the very slender 12m long unrestrained 533 x 210 UB section and similarly did not correctly consider buckling of the columns. Both BS 5950 and Eurocode 3 give guidance on design for lateral-torsional buckling. Also the Steel Construction Institute has published a guide on 'Design of Single-Span Steel

Portal Frames to BS 5950-1:2000' which considers in-plane stability, rafter design and stability and column design and stability and contains a number of worked examples. BCSA, Tata Steel and SCI are currently preparing a similar publication for the design of portal frames to Eurocode 3. Member strength capacities are only justified if the assumptions used in calculation are clearly understood and verified in detailing. In steelwork, this applies to connection capacity and stiffness, and matters of restraint on beams and columns: that is both global restraint (to limit sway) and local restraint to prevent buckling. Portals have complex conditions where flanges are in compression: along the top edge of rafters and on the inside of their eaves haunch/column interface. Stability also needs to be considered during erection when necessarily permanent restraints such as purlins are absent. A number of failures have occurred through lack of restraint; FC Twente Stadium in Holland during erection (2011) and on Hartford Civic Centre Roof Failure (1978) (purlins not in the same plane as the compression boom and therefore ineffective). This example illustrates the importance of thorough design checks and the value of a 'review' undertaken by a more senior person who has the ability, and time, to stand back and ask the searching questions.

369 Substitution of undersized steel sections

Having read your recent warning on the failure of certified steel products (Newsletter No 29), despite having certification, a reporter feels it necessary to report a similar but more basic problem. Given the continuance of the recession he and colleagues are seeing an increased use of smaller than specified steel sections in domestic construction. For example a 203 x 203 x 60 UC might be specified on a drawing by an engineer and a 203 x 203 x 46 UC appears on inspection of the installation. Occasionally the correct size is even shown on the delivery ticket (Alert Anomalous documentation for proprietary products). On other occasions the builder has made a decision that the steel is over designed and reduces it without consultation. This is now occurring on applications in one city at the rate of once or twice a week. The reporter has checked with colleagues in other cities and a similar picture is emerging. This is not a new problem but it is on the increase. The reporter and his colleagues are giving builders a list of the dimensions of common section sizes and advising them to check their steel before installation. This has the effect of both helping and warning at the same time.

Comments

This is a serious matter particularly if the substitution of undersized sections is deliberate. The report implies the changes are made to save money for the builder rather than because the specified section is not immediately available. The supply of steel sections is generally well controlled from the steel manufacturer through the stockholder to the steelwork contactor and from there to site. A builder, or fabricator, who substitutes a smaller size, is likely to be in breach of contract (or worse) and may leave himself open to prosecution should anyone be harmed in the event of a collapse. Changing member sizes is potentially dangerous and extremely foolish. On one site a fatality during erection occurred because the wrong grade of bolt steel had been used. It is not possible to distinguish one grade of steel from another just by looking. It's not easy to tell one serial size of beam from another since flange and web thickness variations can be small. Clients including most domestic building owners should employ reputable builders and ideally steelwork contractors with certified quality management systems to ISO 9001 in place. The builder/steelwork contractor should also have a system in place to check the steel arriving onsite – e.g. checking the delivery note against the order, the original specification and the inspection document. This is part of CE Marking and is embedded in the factory production control system. The majority of steel sections usually arrive with the part number marked on the section (hard stamp or stenciled on) to aid identification. Designers should always be involved at the construction stage but on domestic projects this can be difficult. Small builders and fabricators must be made aware of the importance of using correct section sizes.

343 Inadequate steel beam splice

As part of a project a reporter provided the design for a steel beam. The client sourced this from a fabricator and it came with a mid-span splice. When the building control officer asked for a calculation substantiating the splice the reporter's client, the fabricator, apparently stated that the splice was a “full structural connection”. The reporter found that capacity of the splice was around 20 kNm which was only about a third of the design moment required. By this time the beam had been installed and partially loaded but fortunately no collapse had occurred. Remedial works were then carried out.

Comments

In many steel structures connections are the weak link. The words ‘full structural connection’ are meaningless and betray that whoever designed/detailed the splice did not understand what they were doing. A mid-span splice, which is unusual, was apparently introduced without the knowledge of the designer, and if this had been discovered prior to installation would doubtless have been questioned. Perhaps the conditions of engagement of the designer did not include site inspections. Connections are generally designed by the steelwork contractor and the majority of them have competent engineers capable of designing connection details to support the applied actions. Also BCSA and SCI have published a number of guides on the design of simple, moment and composite connections both to BS 5950 and more recently the Eurocodes. These design guides include procedures for designing beam-to-beam splices. Once again provided the client employs a reputable steelwork contractor this shouldn’t occur. In principle the architect (if there is one), as lead designer, should advice the client on the requirements for an engineer to be responsible for overall stability and on the engagement of competent contractors.

393 Steelwork connection design

Thank you very much for your latest Newsletter, says a reporter, who was particularly interested in an article on the design of steelwork connections. He has worked as project engineer on many projects and has also worked for steelwork fabricators designing connections for them on hundreds of projects. It is undoubtedly true, in his experience, that steelwork designs sent by project engineers to contractors are sometimes deficient. However on commercial projects the responsibilities of the parties are usually defined by NSSS 5 (National Structural Steelwork Specification – 5th edition) and the parties usually make some effort to work to this document. Of greater concern to the reporter is what happens on small domestic projects. Although the architect may refer to NSSS 5 in his specification, he frequently awards contracts to small builders whose steelwork fabricators do not have the expertise to design connections or prepare fabrication drawings. The architect may also not provide enough information on his drawings to enable the steelwork to be set out or the levels agreed (hence some of the connections cannot be designed and some of the fabrication drawings cannot be prepared). On such projects he frequently finds that:

1) although he asks to see fabrication drawings none are received and
2) he is not invited to site to check the as-built structure.

What often happens is that he is asked to design critical moment connections and the rest is "sorted out on site” between the architect and the builder without any reference to the structural engineer - obviously a highly unsatisfactory situation. This is presumably driven by the desire of the architect to save costs and the reporter’s firm only then gets involved if something has gone wrong. The reporter would be interested to know if other engineers are worried about such issues.

Comments

This illustrates very well the situation of a safety-critical industry being allowed to operate in a piece-meal manner. Indeed the architect is open to legal action should there be a significant problem involving harm to persons, for not identifying the hazard of inappropriate delegation, and for failing to engage a competent constructor. It is prudent for the structural engineer to ensure his appointment is fully qualified but even so he also has obligations to identify hazards and mitigate risks (e.g. by recommending suitable supervision) arising from his design, and pass on this information. As regards site visits a key attribute to ‘safety’ is confirming that what parties thought was being built was actually built. As said in the comments for report 369 the client must employ a steelwork contractor with the right skills to design and fabricate the steelwork. BCSA’s web site www.steelconstruction.org can assist parties in identifying a steelwork contractor with the correct range of technical and commercial skills required for the job. There is also a ‘find a steelwork contractor’ app for iPhones.

373 POLYETHYLENE PIPEWORK HANDRAILS

A reporter has seen handrails for edge protection being provided using Polyethylene Pipework. One instance was on a site where the handrail has been used on an access platform fixed to formwork. Other defects were also apparent on this platform, such as the lack of toe boards, excessive gap between rails etc. However it is the principle of providing the rails in PE that the reporter is particularly concerned with. He suspects that the driver for using the PE in this instance was to reduce the weight of the shutter when being lifted into position as well as the cost saving and possible difficulties with achieving the radius profile required. However, given the reduced robustness of the PE in comparison with say a standard scaffold tube and the consequences of a failure of such a system (on this example a fall of 6-7m) he is of the opinion that this design should be raised as CROSS report.

He has also recently been asked to produce a design for another contractor for this type of system, which he has declined on the basis that the design risk assessment would render the system unworkable. His firm highlighted issues such as thermal effects, UV stability of the handrail, accidental damage, restrictions on the type of work that could be carried out in the area being protected (no cutting etc.), deflection, plastic deformation, damage due to over tightening at clamp positions and other issues. This appears to be evidence of a more widespread adoption of this type of handrail system in the construction industry.

Comments

There are many types of plastic pipes and the properties of these pipes are not known. Here the concern is that the handrail construction does not meet either strength requirements or operational good practice. The application is certainly unusual and it is not known to the CROSS panel if such pipes comply with the usual standards for safety rails. Care should certainly be taken when using products for any situation beyond their normal design use and the risks stated by the reporter are very real. Handrail products may be advertised as having 'plastic rails' but these are plastic coated steel sections. Guidance for temporary and permanent edge protection is given in the following references but care is needed when specifying a product for a particular application such as acknowledging the difference between prevention of access to an area of danger rather than preventing a fall from height.

BS 6180 Protective Barriers In and About Buildings 1999
The Building Regulations Part K 2000
BS 6399 Part 1 Loading for Building
BS 6399 Part 2 Code of Practice for Wind Loading 1997
Health & Safety in Roofwork 2008
BS EN 13374 Temporary Edge Protection Systems

377 CERTIFICATION OF SEISMIC DESIGN SOFTWARE

A reporter is concerned about third party certification of seismic design software, and believes that users should carefully consider the assumptions that have been made in any such program. In the opinion of the reporter certification is no substitute for the proper checks required by National Standards, such as Eurocode 8.

Comments

Those involved in seismic design know that the frequency and stiffness of real buildings are difficult to model as they are affected by non-structural elements such as walls. Assumptions that are embedded in software must be identified such as the way in which lateral loads are distributed between the members, P-delta effects, and the consequences of non-linear behaviour. This is another generalised warning that software should only be used in the hands of those sufficiently competent to understand its presumptions and output. Seismic design and analysis is highly specialised and an art which need to be practiced with experience and caution. Results are affected *by the number of variables that have to be set during modeling, assumptions that have been made and the possibility of errors in the programming. In practice the resilience of a building to seismic loading depends not only on the accuracy of modeling but on the correctness of the detailing.*

391 Proprietary software for cantilevers

Whilst checking output of a cantilever beam from a proprietary software package a reporter noted that the section incorrectly passed the lateral torsional buckling check because it failed to check the absolute value of the hogging moment against the allowable buckling moment. Hence the buckling moment of say 10kNm was greater than the applied moment of -30kNm. He raised this with the product developer who is including an update in the next software edition but asks how this affects those not on a software support contract with them.

Comments

It would be assumed that any reputable software house will have a record of who has purchased software and would advise them of updates. Once again this raises the issue of over reliance on software without background knowledge or indeed basis knowledge. Cantilevers are always to be watched and manual checks made.

HOW TO REPORT

Please visit the web site
www.structural-safety.org
for more information.

When reading this Newsletter online **click here** to go straight to the reporting page.

Post reports to:
PO Box 174
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Comments either on the scheme, or non-confidential reports, can be sent to structures@structural-safety.org

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