Launch of CROSS-US

We are pleased to publish this very first newsletter of Confidential Reporting of Structural Safety-US (CROSS-US). Established in the summer of 2019, CROSS-US is part of a growing international network of CROSS entities, all based on the original CROSS system started in the UK in 2005 under the leadership of Alastair Soane. CROSS-US is the fourth, following the UK, South Africa, and Australasia, in what is envisioned to be a large global network for sharing information on structural failures, near misses, and other safety concerns. The potential global impact of improving practice and reducing failures is enormous.

The motivations for establishing CROSS-US were twofold. First, there has long been interest and, indeed, considerable activities, in the US in learning from the performance of our built environment. Some past and present activities in the US include creation of the Architecture and Engineering Performance Information Center (AEPIC) at the University of Maryland, establishment of the ASCE Forensic Engineering Division, publication of the ASCE Journal of Performance of Constructed Facilities, numerous Wikis and other web publications, as well as conferences, books, and university curricula. These efforts have been highly effective, and CROSS-US is not intended to replace them. CROSS-US, however, supplements these activities through a proven system for reporting and discussing issues, well-honed through fifteen years of experience and offering confidentiality and the expertise, integrity, and impartiality of a distinguished, expert Panel. We are concerned with both the technical and procedural causes of failures so as to improve public safety. Our goal is to make CROSS-US the go-to resource in the US for information on structural failures, incidents, and safety concerns. The second motivation for establishing CROSS-US is to join an international network of CROSS entities and lead the way in improving safety in the US.

CONTENTS

| US-6 | Rigid wall/flexible diaphragm roof collapse during an earthquake › 3 |
| US-5 | Collapse of tower cranes during dismantling › 4 |
| US-11 | Concern over use of standoff brackets with C-shaped cornice hooks for scaffolding support › 4 |
| US-3 | Failure to maintain roof drainage during re-roofing leads to ponding instability collapse › 5 |
| US-9 | Hartford Coliseum roof collapse (legacy report) › 6 |

HOW TO REPORT

For more information, please visit the How to Report› page.

If you have experienced a safety issue that you can share with CROSS-US, please Submit a CROSS-US Report›. If you want to submit a report by post, please send an email to administrator@cross-us.org› asking for instructions.

KEY

© CROSS-US Report
C CROSS-US Panel Comments
N News
I Information
M In Memoriam
> Denotes a hyperlink
Panels collaborated on this case. The third
The CROSS-US and CROSS-UK expert
to learn from each other across borders.
2019 during dismantling – one in the UK
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CROSS's use and value. The first case is
that we think demonstrate the breadth of
CROSS-US since its launch, but we have
Many reports have been submitted to
CROSS-US since its launch, but we have
chosen five cases for this newsletter
that we think demonstrate the breadth of
CROSS's use and value. The first case is
a roof collapse during an earthquake in
California. There has been much interest in
CROSS from the US seismic community. In
the second case the reporter notes concern
raised by two tower crane collapses in
2019 during dismantling – one in the UK
and one in the US. It is testimony to need
to learn from each other across borders.
The CROSS-UK and CROSS-US expert
Panels collaborated on this case. The third
report alerts the community to an unfolding
concern with the use of standoff brackets
for C-shaped cornice hooks for scaffolding
support. It is an excellent example of how
CROSS can be used to release timely
information about safety concerns. The
fourth report involves a building collapse
caused by inappropriate stockpiling of
construction debris during a reroofing
operation. This was not a news-making
collapse, and the lessons learned might
not otherwise be brought to light if not for
the existence of CROSS. The fifth is a
retrospective look at a legacy failure, the
catastrophic roof collapse of the Hartford
Coliseum in 1978. While processing of
legacy failures for which public information
exists was not the primary motivation for
establishing CROSS, we must not forget
past lessons learned. The concerns over
computer software and modeling errors
raised by this case are very much relevant
today. We intend to feed additional legacy
cases into CROSS-US over time.
CROSS-US will not be all that it can be
without your participation. We rely on
volunteers from all parts of the profession
and industry to submit reports. No concern
is too big or too small, and CROSS is
structured to make submitting reports
easy and confidential. Please take the time
to become a student of CROSS reports
and encourage others to do the same. We
welcome your submissions and suggestions
for improvements to CROSS-US via the
“Feedback” button at www.cross-us.org or
by email to either of us.
Glenn and Andy

Robert Ratay, PHD, P.E., F.ASCE

We are deeply saddened by the loss of our
friend Robert Ratay, who passed away on
January 3, 2020. A long-time supporter of
CROSS, Bob was an enthusiastic and dedi-
cated member of the CROSS-US expert
Panel.
Bob was a giant in the structural and
forensic engineering community. He was a
structural engineer in private practice and
Adjunct Professor at Columbia University
in New York. His practice and teaching
focused on the investigation and analysis
of structural and construction failures. His
four decades of professional experience in
structural engineering were divided equally
between full-time design practice and com-
bined teaching and consulting. He was a
consultant to architects, engineering firms,
contractors, corporate clients, law offices,
insurance companies, and government
agencies. He was lead expert in over 200
structural failure investigations.
Bob served on and chaired several national
and international technical committees
concerned with structural performance,
building science, construction methods and
safety, and design codes and standards.
He served on the SEI Board of Governors.
He was a Fellow of IABSE and chaired
its Working Group on Forensic Structural
Engineering. He was a member of the
Editorial Advisory Panel of the Forensic
Engineering Journal of the Institution of
Civil Engineers, UK (ICE).
This first CROSS-US newsletter and its
reports are dedicated to Bob.
US-6: Rigid wall/flexible diaphragm roof collapse during an earthquake, Report ID: 877

**REPORT**

This case involves the partial collapse of a roof of a one-story masonry building in California during a 2019 earthquake. The building structure has concrete masonry bearing walls with roof trusses of wood chords and steel diagonals. The roof diaphragm/deck is plywood. Original construction was from the early 1980's but the building had been expanded several times. The part of the building where the roof collapsed is believed to have been built in the mid-1980's. There was a seismic foreshock the day prior to the collapse event. The ground shaking during the collapse event was moderately high by current code standards for the site. In the failure, the roof trusses separated from the bearing walls, and the roof collapsed onto the ground floor. The reporter indicated that the truss bearing failure appeared to be from cross-grain tension failure of the timber ledger at the truss-bearing wall connection as illustrated in Figure US-6.

The reporter notes that cross-grain tension restrictions did exist in the California Building Code when the part of the building involved in the collapse was built in 1984.

**COMMENTS**

Good structural practice for robustness, especially in earthquake zones, is to ensure that parts are well tied together - as here, roof structure to supporting walls. Seismic action will produce large lateral loads from roof to walls, so the viability of the overall load path to ground must be assured by the designer, and that means checking the integrity of all components in the load path.

Problems with cross-grain tension and cross-grain flexure in timber-diaphragm-wall connections were identified in the 1971 San Fernando earthquake. These were addressed in the 1976 *Uniform Building Code*1 Para. 2312.D.3:

3. Special Requirements. A. Wood diaphragms providing lateral support for concrete or masonry walls. "Where wood diaphragms are used to laterally support concrete or masonry walls the anchorage shall conform to Section 2310. In Zones No. 2, No. 3 and No. 4 anchorage shall not be accomplished by the use of toe nails, or nails subject to withdrawal; nor shall wood framing be used in cross grain bending or cross grain tension."

Current codes and design standards continue to prohibit diaphragm-wall connections that place timber in cross grain flexure or tension, and some contain examples of good and bad connections. Examples are the National Design Specification2 Para. 4.1.5.11 and ASCE 7-163 Para. 12.14.7.5.2.

A second factor in this failure is the large displacements caused by a flexible diaphragm. When flexible diaphragms deflect laterally the tops of the walls to which they attach experience increased inertial forces, placing large demands on wall-diaphragm connections. Computation of demands on such connections must consider increased accelerations caused by the dynamics of flexible diaphragms.

![Figure US-6: Cross-grain tension failure of timber ledger on CMU wall](image-url)

**REPORT**
The reporter notes that there have been several instances over many years where tower cranes have fallen during erection/dismantling, citing two such events that occurred in 2019, one in the US and one in the UK.

In the US incident, the crane was being dismantled when it collapsed into the street, causing fatalities. At the time, gusty but not exceptional winds were reported in the area. The tower had been assembled in 20 ft (6 m) long, four-legged truss sections. An investigation by state authorities concluded that workers ignored the tower manufacturer’s dismantling procedures and prematurely removed the tower’s assembly pins.

The UK incident occurred at a construction site on an existing building. The crane was being dismantled when a 20m (66 ft) section of the crane separated and fell to the roof. Fortunately, no one was injured.

**COMMENTS**
The CROSS Panel offers no additional opinions regarding the specific causes of the two incidents cited by the reporter, but general observations regarding crane site safety follow.

In NYC between 2006 and 2015 operations related to lifting/jumping or dismantling tower cranes accounted only for 10% of crane incidents, but they produced by far the largest number of casualties. This included seven fatalities that occurred during the jumping of a crane on East 51st St. in Manhattan, the deadliest crane construction accident in recent memory. Investigations by OSHA and the NYC Department of Buildings determined that the cause of the accident was human error.4,5

During tower crane lifting and disassembly operations the temporary unbraced length of the tower crane is the determining engineering consideration. These operations need to occur in acceptably low wind conditions. In most instances crane jumping and installation of new ties to the building occur together and involve numerous parties/participants. Participants’ experience and coordination are essential. As a lesson learned from the accident on East 51st St., the NYC DOB introduced rules that require a pre – jump safety meeting and also several new responsibilities on the site – lift director and assembly/disassembly director. See NYC Building Code 2014 Sect 3319 and Local Law 14 of 20186,7. The risk associated with a crane collapse may be to adjacent infrastructure with disproportionate consequences such as equipment falling onto a railroad track in the path of a train. In the US this was examined in a 2011 report “Preventing catastrophic events in construction”8.

As always, key operations need to be under the control of qualified competent staff who work to procedures provided by crane suppliers. Correct procedures for erection and dismantling must be followed.

“Factors that affect safety of tower crane installation/dismantling in construction industry”9 investigates factors that have contributed to accidents during tower crane installation/dismantling in Korea. A total of 38 fatal accident cases involving tower cranes occurred between 2001 and 2011. Accidents occurring during installation/dismantling of tower cranes accounted for 68.4% of all fatal accidents. Accident analysis identified “not following work procedures” as one of the main causes of these accidents, followed by “unsafe acts of workers.” In 1994 the ASCE Board of Direction approved ASCE Policy Statement 424 on crane safety on construction site. In July 13, 2019 the Board adopted an update to Statement 42410.

US-11: Concern over use of standoff brackets with C-shaped cornice hooks for scaffolding support, Report ID: 909

**REPORT**
A reporter notes that using stand-off brackets with C-shaped cornice hooks (Figure US-11) for supporting suspended scaffolding have been contributing factors in safety incidents in certain circumstances, such as failure and falling of parapet coping stones. This has led to them being banned in 2019 as an interim measure in a major US city. A stand-off bracket is a rigid member that extends an assembly as shown in Figure US-11, and the problem can occur when there is an installation or use of a stand-off bracket attached to a cornice hook (C-hook) to provide a suspended scaffold additional outreach from the face of a parapet or wall.

The ban will remain until such time as the city is able to further study the use of such brackets and promulgate regulations to ensure their safe installation and use.

**COMMENTS**
C-shaped cornice hooks are commonly used to anchor suspended scaffold support lines to building cornices and parapets. Clearly the adequacy of the anchorage depends on the structural capacity of cornice or parapet to which the hook is attached. While such devices may be used in new construction of buildings or in the repair and maintenance of existing buildings, the concern is more acute with existing buildings as (1) the original construction may not be as robust in older construction as in new, (2) deterioration particularly of masonry parapets is common, and (3) it is generally more difficult to ascertain the as-built condition of old construction than new.

These devices will induce compressive, flexural-tensile, and lateral shear loads to various components of the cornice or parapet. In particular, older masonry construction may have little to no flexural-tensile or bond-shear capacity. The special concern with standoff brackets is that they exacerbate the loads imposed on the supported element, especially at the connection of the coping stone to the masonry, by increasing the eccentricity of the downward suspension cable load. The reporter
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must have an appreciation for the load-resisting mechanics and construction conditions involved in the ability to safely resist loads. We hope this report will serve to increase awareness of the important considerations involved with C-shaped cornice hooks for scaffolding support that employ standoff brackets.

In the United States the applicable regulation of such anchorages is commonly governed by OSHA 29 CFR 192611, Safety and Health Regulations for Construction, Subpart L or variations on this adopted by state or municipal authorities. These requirements include the design loads and factors of safety required, employee training, and pre-use evaluations by a competent person. 1926.451(d)(5) contains special requirements for suspension scaffold support devices such as cornice hooks, roof hooks, roof irons, parapet clamps, or similar devices.

Key to a safe and compliant installation is an evaluation by a competent person (1926.451(d)(3)(i)). During façade work, suspended scaffolds are frequently moved, so the evaluation must include all locations where the cornice hook may be moved. Where a competent person recognizes a hazard, he/she may need to seek support from a qualified person (1926.52(f) & (l)). Clearly such evaluations cannot and need not always require destructive evaluation, load testing, or calculations, but such evaluations must have an appreciation for the load-resisting mechanics and construction conditions involved in the ability to safely resist loads. We hope this report will serve to increase awareness of the important considerations involved with C-shaped cornice hooks for scaffolding support that employ standoff brackets.

Figure US-11: C-shaped Cornice Hook with Stand-off Bracket


US-3: Failure to maintain roof drainage during re-roofing leads to ponding instability collapse, Report ID: 864

REPORT
An existing building was being re-roofed, which involved the removal of existing roofing materials from atop the structural roof deck. During this process, construction debris was stockpiled along the low side of the roof and was periodically removed off the roof to a dumpster. During an afternoon, a rainstorm developed which dropped approximately 1/2 inch of rain at the project site.

Unfortunately, the stockpiled construction debris along the low side of the roof acted as a ‘dam’, preventing rainwater from running off the roof. This rainwater built up on the roof, leading to a ponding-stability collapse of the roof framing at the lowermost bay. This collapse could have been averted by simply periodically breaking the ‘dam’ of roofing debris along the low side of the building or by stockpiling debris at the high side of the roof instead. Roofing contractors need to be aware of how existing drainage performs and work to maintain the operability of roof drains and overflow drains during the re-roofing process.

COMMENTS
Roof collapses due to unintended overloads are commonplace. Some, like this case, are due to drainage impediment. Others are due to overloads from construction materials or debris storage, snow piles from plowing, dirt piles from amenity deck construction, and heavy trucks parked in unapproved areas. The NRCA 2019 Roofing Manual12, Chapter 9, gives considerable guidance on reroofing operations, including the importance of understanding existing conditions, evaluation of drainage, and loads that must be considered. The Manual quotes the following from the International Building Code - 2018:

“1511.2: Structural and construction loads: Structural roof components shall be capable of supporting the roof covering systems and the material and equipment loads that will be encountered during the installation of the system.”

The Manual goes on to say:

“The structural integrity of the roof assembly must be maintained during reroofing operations, including loading on the roof attributable to workers and material being present during this special period of time. The roof structure must be able to support all layers of roof-covering materials.”

Re-roofing project specifications frequently contain language such as:

Maintain roof drains in functioning condition

to ensure roof drainage at the end of each workday. Prevent debris from entering or blocking roof drains and conductors. and/or

If roof drains are temporarily blocked or unserviceable due to roofing system removal or partial installation of new membrane etc. provide alternative drainage method to remove water and eliminate ponding.

This case highlights the duties of a contractor to not overload the existing structure and to ensure proper drainage is maintained during re-roofing operations. The latter requires a careful understanding of how the roof surface is sloped and intended to drain. For projects of the scale and consequence that warrant written specifications, language requiring these duties are helpful. Independent inspection of the progress of the work also can reduce risk.


REPORT

The case study involves the January 18, 1978 collapse of the roof of the Hartford Coliseum in Hartford, CT. 11

The analysis and design of the space truss of the roof was an early application of computer simulation. The roof of this three-year-old structure collapsed at 4:15 AM on January 18, 1978 during a freezing rainstorm after a period of snow. Fortunately, there were no injuries sustained as a result of the collapse. The night before, there were over 5,000 people in the coliseum attending an event. Following several investigations of the collapse, it was determined that this was an instance primarily of inadequate structural design. This CROSS-US report contains findings of three investigating firms, called herein Investigator 1, Investigator 2, and Investigator 3.

A triangular lattice steel space grid, supported on four reinforced concrete pylon giving spans of 270 feet and 210 feet, was used to support the roof. Investigator 1 concluded that the interaction of top chord compression members and their bracing played an important role in the redistribution of load and the eventual collapse. They noted that certain compression members were braced against buckling only in one plane. As loads increased, these members buckled out of plane and redistributed the loads to other members. Over time, more chords buckled and fewer and fewer members carried the load. This situation worsened until the remaining members were unable to withstand the added stress due to the loads present that night, and the final, sudden collapse took place.

Background 14

"In order to save money [the Coliseum's designer] proposed an innovative design for the 300 by 360 ft space frame roof over the arena. The proposed roof consisted of two main layers arranged in 30 by 30-ft grids composed of horizontal steel bars 21 ft apart. 30-ft diagonal bars connected the nodes of the upper and lower layers, and in turn, were braced by a middle layer of horizontal bars. The 30 ft bars in the top layer were also braced at their midpoint by intermediate diagonal bars. The design departed from standard space frame roof designing procedures in five ways:

1. The configuration of the four steel angles did not provide good resistance to buckling. The cross-shaped built up section has a much smaller radius of gyration than either an I-section or a tube section.

2. The axes of the top horizontal bars and the diagonal bars did not intersect at a common working point. This induced bending in the members, making the roof especially susceptible to buckling.

3. The top layer of the roof did not support the roofing panels; the short posts on the nodes of the top layer did. Not only were these posts meant to eliminate bending stresses on the top layer bars, but their varied heights also allowed for positive drainage.

4. Four pylon legs positioned 45-ft inside of the edges of the roof supported it instead of boundary columns or walls.

5. The space frame was not cambered. Computer analysis predicted a downward deflection of 13-in at the midpoint of the roof and an upward deflection of 6-in at the corners." 15

"[Investigator 2] discovered that the roof began failing as soon as it was completed due to design deficiencies. A photograph taken during construction showed obvious bowing in two of the members in the top layer. Three major design errors coupled with the underestimation of the dead load by 20% (estimated frame weight = 18 psf, actual frame weight = 23 psf) allowed the weight of the accumulated snow to collapse the roof. 16 The load on the day of collapse was 66-73 psf, while the arena should have had a design capacity of at least 140 psf. The three design errors responsible for the collapse are listed below.

- The top layer's exterior compression members on the east and the west faces were overloaded by 852%.
- The top layer's exterior compression members on the north and the south faces were overloaded by 213%.
- The top layer's interior compression members in the east-west direction were overloaded by 72%.

"The most overstressed members in the top layer buckled under the added weight of the snow, causing the other members to buckle. This changed the forces acting on the lower layer from tension to compression, causing them to buckle also. Two major folds formed initiating the collapse."

Excessive deflections apparent during construction were brought to the engineer's attention multiple times. The engineer, confident in his design and the computer analysis which confirmed it, ignored these warnings and did not take the time to re-check their work. A conscientious engineer would pay close attention to unexpected deformations and investigate their causes. Excessive deflections often indicate structural deficiencies and should be investigated and corrected immediately. Unexpected deformations provide a clear signal that the structural behavior is different from that anticipated by the designer.

The joint of the truss members was incorrectly modeled in the computer as having no eccentricity. As a result of this inaccuracy, bending moments developed in the built structure, causing additional stresses in members. In post-failure investigation a nonlinear analysis was performed using correct joint

13 This is a legacy failure for which substantial public record exists. Hence de-identification of the incident is not necessary
14 This background section is largely drawn from an on-line article by Rachel Martin and available here:[https://eng-resources.uncc.edu/failurecasestudies/building-failure-cases/hartford-civic-center/](https://eng-resources.uncc.edu/failurecasestudies/building-failure-cases/hartford-civic-center/) and an article by Richard S. D'Ippolito and available here:[https://bcatless.ncl.ac.uk/Risks/8/81#subj6](https://bcatless.ncl.ac.uk/Risks/8/81#subj6). Passages taken directly from Martin's article are enclosed in quotation marks.
modeling, and the analysis predicted collapse at the loading at which it actually occurred. [Investigator 3] agreed with [Investigator 2] “that gross design errors were responsible for the progressive collapse of the roof, beginning the day that it was completed. They, however, believed that torsional buckling of the compression members, rather than the lateral buckling of top chords, instigated the collapse. Using computer analysis, [Investigator 3] found that the top truss members and the compression diagonals near the four support pylons were approaching their torsional buckling capacity the day before the collapse. An estimated 12 to 15 psf of live load would cause the roof to fail. The snow from the night before the collapse comprised a live load of 14 to 19 psf. Because torsional buckling is so uncommon, it is often an overlooked mode of failure.”

**COMMENTS**

The failure of the Hartford roof is classic example of the twin follies of firstly relying totally on an advanced structural analysis program without fully understanding the load resisting system, and secondly of a failure to verify that the model adequately represents reality. It is timely to resurrect this failure as warning to contemporary practice.

In current times, with advanced structures, it is increasingly common to place total reliability on ’The Model’ and this is especially of concern when such advanced structures are not amenable to traditional analysis methodology. There is clear danger in this if models are not verified as being truly reflective of expected behavior. However, in the US and the UK there has been much unease that the profession is becoming too disconnected with traditional understanding of structural behavior to the extent that the skill sets required for verifying models are not widespread. Prudence suggests that proper practice is to verify the modeling at the outset and to formally validate the output before proceeding to finalize designs (i.e., acceptance of member sizing).

Over-reliance on structural analysis software and poor modeling have long been a concern in the US structural community. Many papers have been published expressing this concern with recommendations on how to use analysis software prudently. Similar questions of modeling in general and determination of bracing and unbraced length were issues in the 1988 collapse of the Dallas Cowboys Practice Facility. This is a topic of active discussion in university curricula.

Related issues in this failure are a lack of independent checks (i.e., peer reviews) of structures of substantial risk or import and field observations by competent professionals. Since the Hartford Coliseum collapse, many US states have adopted mandatory project peer reviews for structures over prescribed threshold limits. The threshold limits and requirements for the review vary amongst jurisdictions. Also, since the Hartford collapse, many states and other jurisdictions have adopted mandatory special inspections, typically involving in some capacity, the structural engineer of record. Many structures exhibit warning signs prior to collapse, while others may collapse suddenly and without warning.

Questions regarding the sufficiency of computer modeling, the adequacy of peer review, and the role of the structural engineer in the field have been raised recently in the FIU bridge collapse. This will be the topic of an upcoming CROSS-US report.

In the UK the Standing Committee on Structural Safety (SCOSS) and CROSS have had a long-standing policy of endorsing third party checks for key structures. The rationale is to assure public safety. In 2016 SCOSS published a paper “Reflective thinking,”24 which looked at over-reliance on computer modelling and posed this set of questions for the designer:

- Is the model capable of satisfying the requirements? (the validation question)
- Is the model the most appropriate in the context?
- Has the software been validated and verified?
- Has the model been correctly implemented? (the verification question)

The paper referred to two classic structural failures - the Hartford Coliseum collapse and the failure of the Sleipner off-shore platform (1991), both of which, said the paper, were due to inadequate validation of analysis models. The Sleipner sank under a controlled ballasting operation off Norway, and the conclusion of the subsequent investigation was that the loss was caused by a failure in a cell wall. This was as a result of a combination of a serious error in the finite element analysis and insufficient anchorage of the reinforcement in a critical zone.

In addition to the modeling and design errors leading to the collapse, this case should be classified as a process or procedural failure in that multiple warnings were provided, or at least there were multiple opportunities to discover the problems (similar to the Kansas City Hyatt Regency) and appropriate action was not taken. The final committee report notes: “We further believe that the major design deficiencies as pointed out by [Investigator 2] and others should have been detected at one or more subsequent stages to the design phase and that cumulative errors in judgment account for the fact that they were not detected.”

Additional References:
- “Estimated Snow, Ice, and Rain Load Prior to the Collapse of the Hartford Civic Center Arena Roof.”
- “Another Look at Hartford Civic Center Coliseum Collapse.”