Avoiding Structural Failures During Construction

Part 1

By David B Peraza, P.E.

A large percentage of failures and collapses occur during the construction phase. Some of these failures involve the temporary structures and equipment that are used in the construction process, such as formwork and scaffolds. Others involve permanent structures. This article will present engineering principles and "lessons learned" that can be used by design professionals, owners, and construction managers to minimize the probability of these unfortunate events. These principles will be illustrated with case studies of well-known catastrophic events, as well as little-known cases.

Formwork

Premature removal of formwork is one of the most common causes of failures. This type of failure can lead to a progressive collapse, and is therefore extremely dangerous.

One of the most famous cases was the collapse of Baileys Crossroads in 1973. This 26 story residential tower in Fairfax, Virginia was almost topped out when disaster struck. Concrete was being placed on the 24th floor, and formwork was being removed on the 22nd floor. The fresh concrete was therefore supported entirely on the 23rd floor, where the concrete had been placed only 5 days before. The slabs collapsed, one on top of the other, all the way to ground, killing 14 construction workers and injuring 35 workers. *Figure 1* shows this building after the collapse.

Since removal of formwork is closely tied to the contractor's "means and methods," the contractor is in the best position to prevent this type of failure. However, the Engineer of Record (EOR) can assist by requiring that the contractor prepare, submit, and follow a project-specific formwork plan. This plan should include criteria for deciding when formwork can be removed, and the number of floors to be shored or reshored. The formwork plan should be developed by an engineer, retained by and in consultation with the contractor. Compliance needs to be closely monitored, preferably by the owner or a firm retained by the owner.

Premature loading of a slab on metal deck, although not as common, can occur in certain situations. A well known case, is that of a metal deck collapse at Worchester Polytechnic Institutein 1988 in Massachusetts. In this case, the Occupational Safety and Health Administration (OSHA) made multiple attempts to issue fines against the EOR. The case was eventually heard by the US Court of Appeals, which decided that the EOR did not have sufficient control of the jobsite to be under the jurisdiction of OSHA.

What may not be so well known about this case are the underlying technical facts. The area that collapsed was non-typical. It was an area of a 2nd floor slab that projected beyond the building face, creating a canopy for an entrance. As such, the structural slab was depressed to accommodate a layer of rigid insulation and a topping slab, as can be seen in *Figure* 2. The structural slab consisted of composite metal deck and concrete. The collapse occurred while workmen were placing concrete for the topping, after having placed concrete for the structural slab earlier in the day. One end of the deck slipped off its bearing, and the deck dropped open like a trapdoor, causing five workmen to fall 40 feet and sustain severe injuries.

Figure 1: The progressive collapse of the middle portion on this building in Baileys Crossroads, caused by premature formwork removal, resulted in 14 fatalities. Courtesy of the National Institute of Standards and Technology.

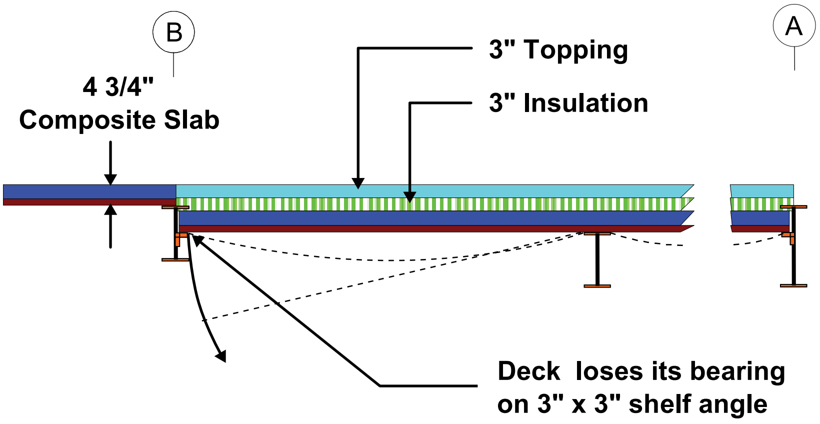

Figure 2: This metal deck collapsed while concrete for the topping was being placed over the newly-placed structural slab. Note also that depressing the structural slab to accommodate the insulation and topping slab introduced some constructability issues that affected the amount of bearing afforded to the metal deck.

There were numerous construction errors that were relevant. Most importantly, the topping was placed while the structural concrete was still fresh, and therefore, before it was acting compositely with the unshored metal deck. In addition, there was minimal bearing of the metal deck on the shelf angles and there were "half moon" puddle welds at the edge of the deck, no sidelap attachments, and no welding washers.

However, there were also errors on the part of the EOR. Most significantly, the EOR verbally approved the placement of the topping prior to curing of the composite slab. This erroneous information was given the morning of the accident, in response to a telephoned question from the contractor. Another unfortunate action was that the EOR noted on the shop drawings some – but not all – areas where metal deck needed shoring. In fact, the area that collapsed did need to be shored, but the EOR neglected to identify that. These gratuitous notations were made in spite of the fact that the specifications placed this responsibility squarely on the contractor. But once the EOR identified some of the areas that needed shoring, it was therefore not unreasonable for the contractor to assume that the EOR had identified ALL of the locations where the metal deck needed shoring. In addition, the depressed slab introduced constructability issues that led to minimal bearing and "half moon" welds at the very ends of the deck. Finally, neither the EOR nor the inspector noticed any of the metal deck installation deficiencies.

Lessons learned from this unfortunate incident include:

1) Non-typical areas require special attention, both during design and during construction. For this particular case, a constructability review would have probably triggered numerous questions.
2) Avoid giving gratuitous advice. For example, if the contractor is responsible for identifying where shoring is needed, avoid taking on that responsibility unless you are prepared to do it fully and completely.
3) Avoid, whenever possible, giving verbal directions to a contractor. If necessary, follow up verbal instructions with written memorandums immediately. This minimizes the possibility of misunderstandings. Often, the process of memorializing a decision triggers additional thought and clarity. Also, taking time to discuss the issue with a colleague may provide additional insight.
4) Competent and thorough inspections, of both typical and non-typical areas, are a vital part of the process.

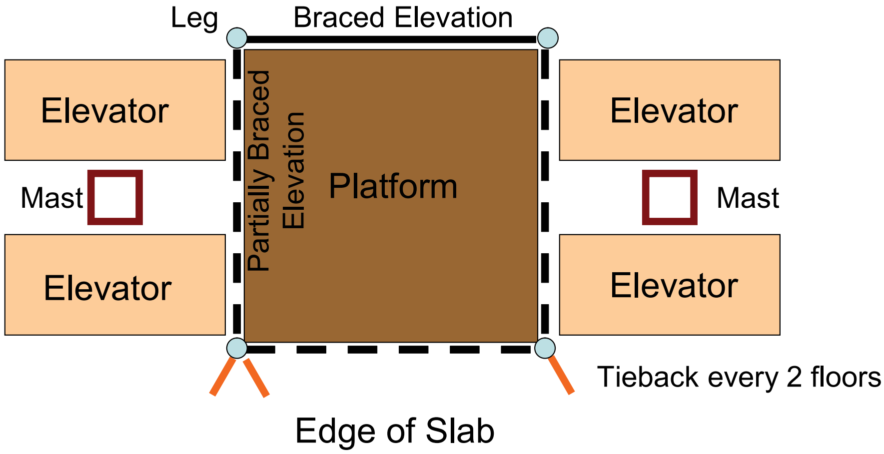

Figure 3a: A plan of a hypothetical platform for a hoist complex. The heavy line indicates a side that provides full bracing to the legs; the dashed lines indicate sides that provide partial bracing.

Column Buckling

Scaffolds and similar structures sometimes require significant engineering design. They are as tall as the high-rise building that they serve, and their structural behavior can be even more complex. Unfortunately, they do not always receive the level of engineering design that they need. One issue that deserves more attention is the buckling strength of scaffold legs.

Figures 3a and *3b* show a common scaffold-like system for a construction hoist complex. The restraint conditions for the legs are more complicated than that of most building columns. The landing trusses are typically moment-connected to the legs, providing the legs with rotational restraint; but the legs are usually spliced with a spigot connection that acts as a pinned connection; and filler bracing, which reduces the headroom to about 6 feet – 8 inches may or may not be effective as a brace, depending on its location. Furthermore, the entire system is typically braced against the building every two or three floors – not every floor.

A common assumption by scaffold engineers who design these structures is that the legs have an effective unbraced length of about 6 feet – 8 inches, which is the clear height between the filler braces and the floor members. However, this assumption can be unconservative, particularly in cases where the system is not braced at every floor, as is illustrated in *Figure 3b*. To determine the true strength of such a structure, it may be necessary to perform a sophisticated buckling analysis of the structure. This analysis would be geometrically non-linear and would include the actual restraint conditions provided to the legs.

In the wake of the 1998 collapse of the 700-foot tall back structure (a scaffold-like structure that connects the construction elevators and hoists to the building) at 4 Times Square, an investigation by the City of New York found that the structure was not built according to the plans, that braces were missing or disconnected, and that equipment was stored on the platforms. The incident also prompted changes in the code that governs the design of these structures.

Some construction managers, general contractors, and owners have learned from this incident and have taken proactive steps to minimize the probability of a similar failure happening on their projects. Those measures include:

- Perform a peer review of significant scaffold structures. Depending on the arrangement of the hoist complex, the analysis may be simple, or it may require a sophisticated buckling analysis.

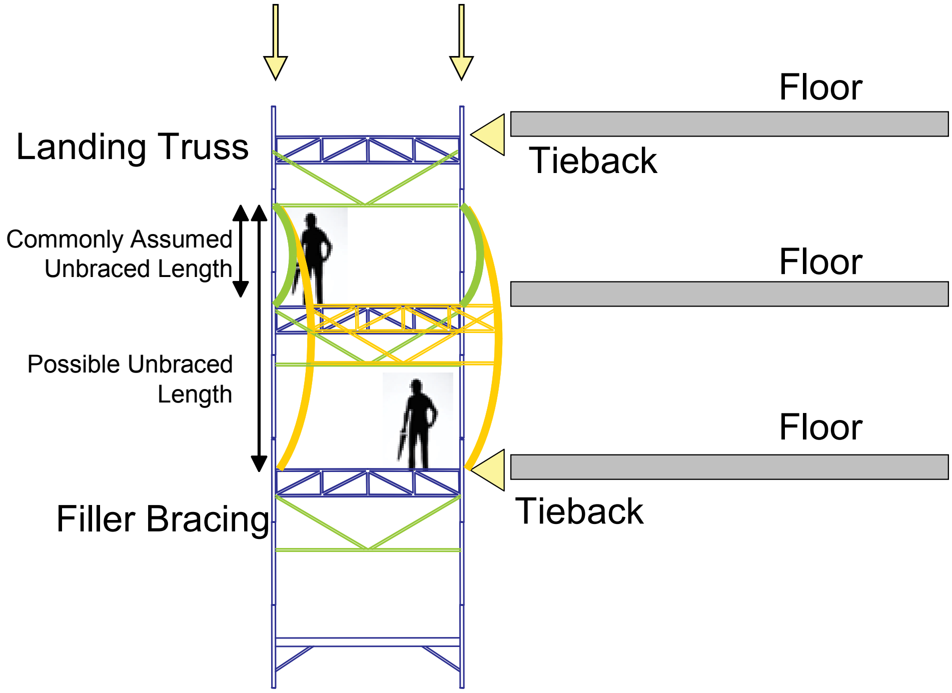

Figure 3b: An elevation through the hoist complex, showing partial bracing.

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- Retain an engineer to inspect the structure. The inspections are typically done once initially, then each time the structure is modified, and then once per month. The inspections help ensure that the structure is built according to the plans, that unauthorized modifications are not made, and that the structure is not misused.

Outside the Envelope

Some temporary structures are initially engineered, but then are modified or used in situations for which they were not designed. These structures are "outside the envelope" and, therefore, are not engineered for all practical purposes. The following case study exemplifies this situation.

Two workers were killed in 2006 when a work platform, cantilevered from the 20th floor of a building under construction, failed. The wood platform was supported by two steel outrigger

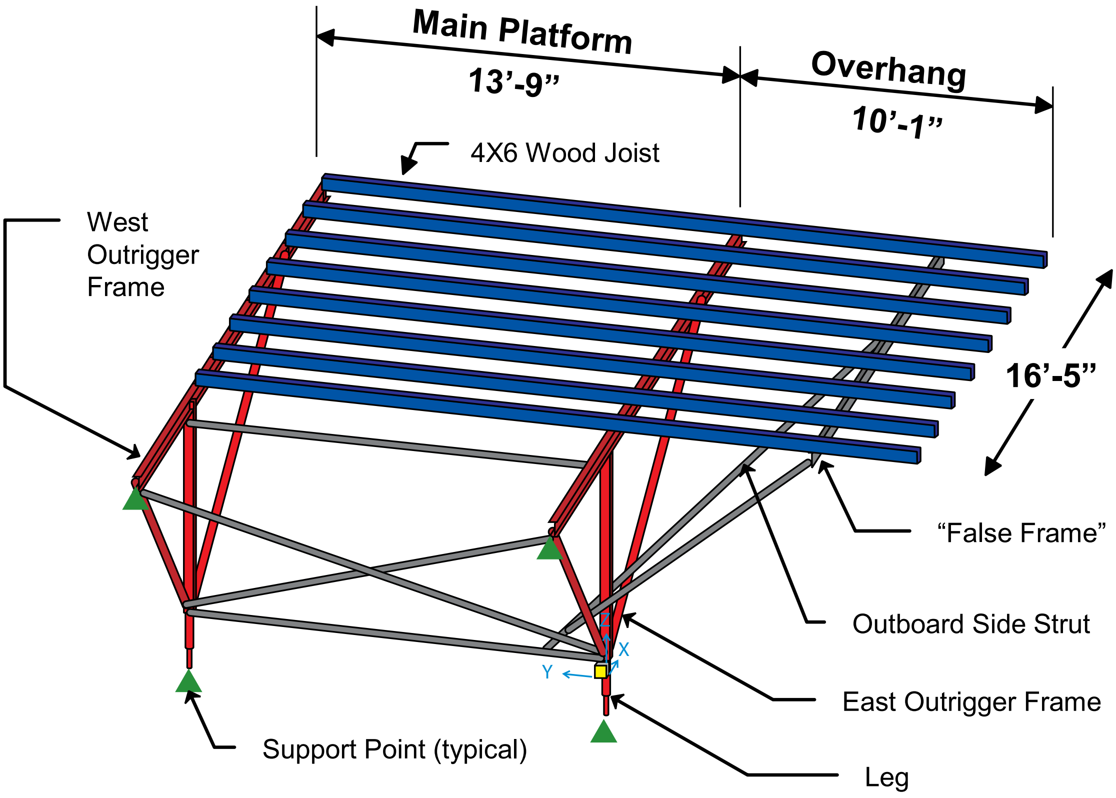

Figure 4a: This shows the construction of a work platform that failed.

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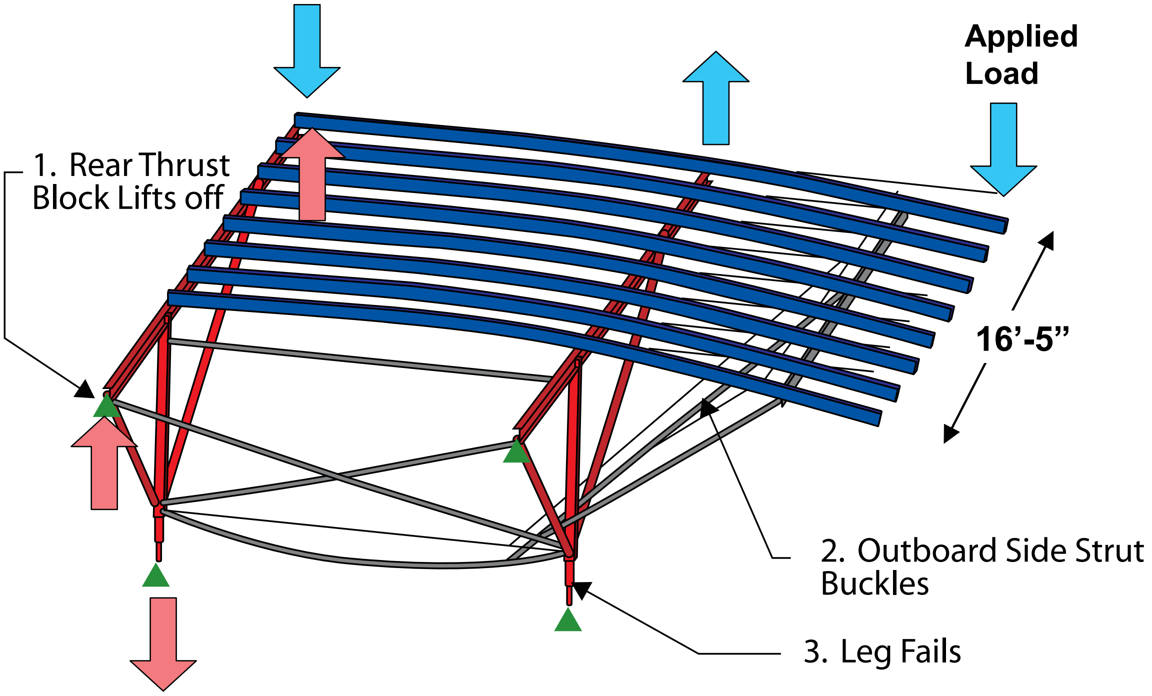

Figure 4b: This figure shows the failure mechanism of the work platform. An applied load near the corner of the overhang caused the far outrigger frame to disengage and become unstable. The blue arrows indicate forces on the wood joist; the pink arrows indicate needed reactions on the outrigger frame.

frames that were supported on the building floor slab and that reacted against the underside of the slab above. There was no mechanical attachment between the outrigger frames and the slabs. The platform that failed was non-typical compared to other platforms on the building. The typical platform was only slightly wider than the two outrigger trusses that supported it. The failed platform had a large overhang to one side. A "false frame," which consisted of a channel and two diagonal struts, apparently was added to support the overhang. *Figure 4a* is a three dimensional view of the platform structure.

Analysis showed that the most likely cause of the failure of the platform was inadequate structural design of the platform structure. The structural design of the platform did not meet the OSHA requirement that scaffolds support four times the intended load, without failure, nor did it meet the ANSI A10.9 requirement that flying forms be designed with a safety factor of 2.5. The platform was constructed substantially in accordance with the shop drawings, and there was no evidence that it was being misused.

The failure most likely initiated when the rear thrust block of the west outrigger frame lost contact with the underside of the 20th floor slab, making the structure unstable. *Figure 4b* shows the likely failure sequence. The main factors that enabled this to happen were the excessive length of the platform's side overhang coupled with the ineffectiveness of the false frame to support the overhang. These two factors allowed the wood joists to "see saw" about the east outrigger frame, so that modest loads applied on the overhang caused uplift on the tip of the west outrigger frame, which was not capable of resisting any uplift loads.

This platform system had been in use for decades. Apparently, considerable engineering was performed when the system was invented. But calculations for the original system were no longer available. Furthermore, project-specific design calculations, that addressed the unusual geometry of the failed platform, were not prepared. Thus, this platform was "outside the envelope", and was never engineered.

Similar failures can be minimized if simple guidelines are followed:

- Recognize that these structures need to be engineered.
- Require calculations, and signed and sealed drawings, for prototypical structures.
- Require project-specific calculations, and signed and sealed drawings, for non-typical or unusual structures

Conclusion

Many failures during construction are caused by lack of attention to fundamental engineering principles, and/or by not adhering to well known construction management principles. Although the EOR does not have direct control over the construction process, he or she can help define appropriate processes to be used in the construction phase, can help recognize temporary structures that require engineering, and can alert the construction manager to these situations.▪

References

Peraza, David B., "Metal Deck Collaps e – Professional Liability During Construction," Proceedings of First Forensic Engineering Congress, American Society of Civil Engineers, Puerto Rico, 1997

"Four Times Square--Board of Inquiry Report "New York City Department of Buildings, September 29, 1999

"Safety Requirements for Concrete and masonry Work – American National Standard for Construction and Demolition Operations," ANSI/ASSE A10.9-1997.

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ASCE 37 "Design Loads on Structures During Construction" is in the process of being revised, with suggestions actively being solicited. Suggestions can be submitted via the author at ***dperaza@exponent.com***, or directly to the committee chair, Dr. Robert Ratay at ***Structures@RobertRatay.com***.