**Introduction**

Skyline Plaza is a large complex located in Bailey’s Crossroads, Virginia which includes eight apartment buildings, six office buildings, a hotel, and a shopping center. In the midst of construction on March 2, 1973, one apartment building (A-4) and the parking garage adjoining it collapsed. The incident occurred at around 2:30 in the afternoon and resulted in the death of 14 construction workers and the injury of 34 others.
It is concluded that the improper removal of forms supporting the 23rd floor resulted in increased shear force around the columns. The recently poured concrete had not yet reached its full strength capacity and was unable to withstand these increased forces. Therefore, the trigger mechanism of the collapse was shear failure around a number of columns on the 23rd story. Without the support of these columns, other columns on that story were overstressed which ultimately led to the collapse of the entire 23rd floor slab onto the floor below. The increased loading on the 22nd floor from the weight of the collapsed floors above was too great and led to a progressive collapse all the way to the ground level. (Delatte 2009) Figure 1 shows the damage following the collapse. Cicled numbers indicate floor levels and hexagons indicate column line locations.

Project Team

Owner: Charles E. Smith Companies
Architect: Weihe Black Jeffries & Strassman
Design and Construction

Building A-4 was a reinforced concrete structure with flat plate floor slabs. It was designed as a 26 story apartment complex with a four-story basement and a penthouse level. All floor slabs were 8” thick and the floor-to-floor height was 9’-0”. A typical floor plan for building A-4 can be seen in Figure 2 above. The design strength of the concrete columns and floor slab is listed in Figure 3 below. Both the engineer and architect specified to the contractor that each slab being poured must be shored at least two stories below.

Concrete Design Strength (psi)
Floor: Base-7th 7th-17th 17th-Top
Columns 5000 4000 3000
Since the building was 336'-0" long, each floor slab was to be poured in four sections. The stages of the pour are shown in Figure 3 below. At the time of the collapse, sections 1 and 2 of the 24th floor had been poured. Section 3 was in the process of being poured and section 4 had not yet been poured. Based upon statements from construction workers and photographs taken, it was concluded that the forms for section 1 and 2 on the 22nd story had been removed prior to the collapse. The forms for section 3 on level 22, two floors below the location of the current pour, were being removed at the time of the collapse.

Figure 4: Extent of slab pour at time of collapse (Leyendecker 1977). Image provided by National Bureau of Standards.

Two cranes were used to erect the building, one in section 2 and the other in section 4. At the beginning of construction, these cranes were supported on the 4'-0" thick mat foundation. As construction continued, both cranes were lifted in order to complete the upper levels of the building. On the day of the collapse, the base of the crane is section 2 was located on the 20th floor while the base of the crane is section four was located on the 14th floor. (Leyendecker 1977)

Analysis of Failure
On March 5, 1973, three days following the collapse, the Center for Building Technology of the National Bureau of Standards was called upon to investigate the collapse of Skyline Plaza and determined the cause of failure. A three-dimensional finite element analysis was conducted on the 22nd and 23rd floors to determine the magnitude of forces exerted on the floor slabs and whether the slabs could properly handle those forces. For completeness, an analysis of three separate cases was run in order to cover all possible conditions at the time of the collapse.

Case I: All forms on the 22nd story were removed before the collapse. This essentially means that the 23rd floor slab carried its own weight, the weight of the 24th floor slab, and the weight of the forms underneath the 24th floor slab. The strength of the concrete on the 23rd floor slab used for this calculation was 1200 psi.

Case II: Assumed that the concrete on the 23rd floor slab reached its design strength of 3000 psi.

Case III: Only some of the forms on the 22nd story were removed which results in both the 22nd and 23rd floor slabs sharing the load from above. The strength of concrete on the 22nd floor slab used for this calculation was 1340 psi.

Upon completion of the analysis, it was determined that moments in the column strips of the slab were not great enough to cause failure. On the other hand, the analysis did show that for case I and III column #67, 68, 83, and 84 all experienced shear stress greater than the shear capacity of the concrete slab. This indicates that the partial or complete removal of forms was a major contributing factor to the collapse. The analysis of case II shows that the shear stress in the slab did not exceed the design capacity. This result confirms that the strength of the 23rd floor slab was below the design strength of 3000 psi at the time of the collapse. (Leyendecker 1977)

This type of failure is highly undesirable because it usually strikes without much warning. It also very easily can lead to progressive failure which is defined by the Portland Cement Association as "the local failure of a primary structural component leading to collapse of adjoining members which in turn leads to additional collapse."(Polak 2005) In the case of Skyline Plaza, the failure of the local columns on the 23rd floor due to punching shear initiated the failure of floor slabs below it and eventually created a total failure much greater than the original failure.

Non-Compliance Issues

Along with performing a thorough analysis of the structure, the Center for Building Technology of the National Bureau of Standards was asked to assist in the determination of any non-compliance of OSHA regulations during the design and construction of
Skyline Plaza. Several infractions were discovered but not all were labeled as contributing factors to the collapse of the structure.

According to OSHA regulations, forms are required to be in place for a minimum of 10 days with temperatures greater than 50 degrees F for spans longer than 20'-0". 

Violation: Areas on the 22nd story had spans greater than 20'-0" but forms on that level were removed before 10 days at 50 degrees F.

According to OSHA regulations, concrete specimens are to be tested in order to confirm that the concrete has obtained the required strength to handle the loading placed upon it. 

Violation: No specimens were tested.

According to OSHA regulations, bracing and shoring must be designed to handle lateral loads. 

Violation: (2) nominal 3x4 braces at 16'-0" O.C. were not capable of handling the lateral loads.

According to OSHA regulations, shoring is only permitted to be out of plumb by 1/8" per 3'-0". All damaged or weakened shoring must be removed and replaced. 

Violation: Shoring on the 23rd and 24th floors exceeded this limitation. They were also not removed.

According to OSHA regulations, an inspection is required before, during, and after the placement of concrete. 

Violation: No inspection was completed.

According to OSHA regulations, the distance between top and bottom supports of the crane shall not exceed 18'-0". Standard tower sections used shall not exceed four as recommended by the manufacturer. 

Violation: Both cranes had a distance of 18'-4" between the top and bottom supports. Crane 2 used five tower sections causing it to exceed the height limitation of 81'-0". (Leyendecker 1977)

Conflicting Accounts

The only conflicting account surrounding the Skyline Plaza collapse is related to the complete removal of formwork on the 22nd floor at the time of collapse. Three different responses were obtained through interviews with workers on site. Some believed that all the forms were removed. Others thought that the forms were partially removed. A third group thought that no forms were removed at the time of the collapse. This discrepancy was accounted for by the different cases in the analysis. (Leyendecker 1977)
Preventing Failure and Lessons Learned

Based upon the analysis of case II it is concluded that this collapse could have been prevented if the shoring remained until the concrete reached its full design strength. (Leyendecker 1977)

The tragedy of Skyline Plaza taught the building industry some important lessons.

1. Redundancy within structural design is essential to prevent progressive collapse. (Feld 1997)
2. Construction loads must always be considered during design. There are many instances when these loads will control the design. (Ross 1984)
3. Formwork and shoring needs to be detailed by the contractor.
4. Concrete testing must be performed before the removal of shoring.
5. Inspections must verify that the contractor is properly shoring floors above and that poured concrete is meeting its design strength. (Kaminetzky 1991)

The prevention of progressive collapse is still a big concern for structural engineers. There are now design methods for concrete structures that can be employed to limit the possibility of universal progressive failure. In slab design, it is now encouraged to place rebar continuously through the slab-column intersection at the top and bottom of the slab. If the slab fails in punching shear, the bottom bars act as a catenary and prevent the collapse of the slab onto the structure below. Other provisions include casting the concrete slab monolithically with beams and not splicing reinforcement at midspan or end of slab. (Dusenberry 2007)

Responsibility

The Skyline Plaza collapse became a major landmark in the debate concerning responsibility during construction. At the end of the legal proceedings it was the design engineers and architects who were found guilty of negligence. The general contractor and concrete contractor had some liability but it is ultimately the responsibility of the designers to visit the job site and to warn the contractors of any possible pitfalls due to unforeseen environmental conditions. So even though the contractor did not comply with the shoring requirements specified in the construction documents, the collapse was still found to be the fault of the designers. (Feld 1997)

It is best if the engineer of record works alongside the contractor to develop a formwork
removal plan. This plan would include detailed instructions on the proper time for removal of formwork and how many floors need to be supported at any given time. The owner should also have some responsibility to make sure the plan is followed by the contractor (Peraza). This way, the entire team is on the same page and mistakes will be avoided.

Industry Response

Following this failure, the Portland Cement Association (PCA) and the Prestressed Concrete Institute both issued new design guides with provisions included to prevent progressive collapse. In November of 1974, the ACI Journal reinforced the importance of designing for construction loads as well as normal design loads. This accident also made designers aware of the importance of site inspections and that it is still the responsibility of the designer to make sure the building is constructed properly. All of these changes have helped to prevent the frequency of these kinds of failures. (Ross 1984)

Of course, there is always a chance that a massive failure like Skyline Plaza could happen again. Architects, engineers, and contractors must always be vigilant when it comes to the design and construction of buildings. It is never guaranteed that all legal requirements will be followed on a jobsite or in a design office. However, with improved codes and sound judgement these failures can be easily prevented.

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Binishell Domes

*Carl Hubben, BAE/MAE, Penn State 2010*

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**Introduction**

Binishell domes are thin-shelled reinforced concrete structures which became popular after the first was built in the mid 1960’s by the Italian architect Dante Bini. Since then, roughly 1600 Binishells have been built in 23 countries across the world (Levy, 39). The
domes were well accepted due to their quick construction time, low cost, high strength and reduced carbon footprint when compared to conventional construction.

Binishell Example
They range in height from roughly 36 to 115 feet and have been built to span up to 300 feet. Due to the versatile nature of Binishells, they have been used in the construction of schools, housing, sports arenas, shopping malls, storage buildings and silos (Binisystems). The domes are constructed by inflating a membrane which lifts reinforcement steel and wet concrete into the desired shape by varying the air pressure within the membrane. The unique construction needs to be followed accurately and if it isn’t the domes quickly lose their structural integrity. Due to faulty construction practices, there have been issues with the domes’ strength and in a few cases, Binishells have failed to the point of collapse.

Construction
Binishell construction is a successful and innovative application of pneumatic forming. Traditionally, pneumatic forming is done by fastening a membrane to the edges of a mold and then then formed with the assistance of heat and compressed air. This technique was used to mold small items such as sinks, bathtubs, and aircraft windows. Dante Bini simply applied the theory on a much larger scale.
1.) Placing the springs and reinforcement

Construction of Binishell domes begins with placing a concrete anchorage beam circular in plan with a molded recess. A slab on grade is also poured at this time to act as the floor of the dome. The anchorage device of the inflatable neoprene-coated nylon membrane is placed within the recession in the beam. Steel springs of various strengths are then stretched across the membrane at which time reinforcing bars are inserted inside the springs.

2.) Pouring the concrete

A thin layer of concrete is then poured over the membrane, covering the springs and reinforcement. The concrete is then covered with light PVC sheeting which is also secured to the anchorage. Once the sheeting is in place, the spool and roller vibrators are placed at the center of the membrane.
3.) Inflating the membrane

Now, the membrane is inflated through the ducts under the membrane. Although the pressure in the membrane is very low, the entire mass of the wet concrete, springs and reinforcement rise. As the pneumatic form fills with air, the different tensions in the springs control the lifting speed and shape of the dome. The springs also ensure the location of the reinforcement as the dome is lifting. This process takes roughly 1-3 hours.

4.) Using the roller vibrators

When the dome has reached the desired height and shape, the rolling vibrators are pulled over the exterior of the surface to re-consolidate the concrete. When this is completed, the height of the dome is held constant by maintaining the pressure in the dome for 1-3 days depending on the size of the structure. During this time the PVC sheeting can be removed. Once the dome has reached sufficient strength, the form is deflated, leaving the 3 day old Binishell to support itself.

The construction process has been further developed and patented by Binisystems. Certain techniques have been implemented to have a more accurate structure in an effort to ensure the strength of the completed domes.
Failures and Causes

The unconventional and tedious construction of Binishells has caused some domes to fail and in a few, collapse. Two failures are covered in detail in this report along with one which were not well documented. The Fairvale High School and the Pittwater High School failure generated a large amount of interest because Dante Bini had been an architectural consultant during the design and construction of both of these domes. These collapses resulted in the investigation of Binishells with similar geometry. Some of these investigations were responsible for the deconstruction of standing Binishells.

In the poorly documented cases, the dome was located in Australia and was going to be used as a school. The first collapse occurred just two days after the inflation of the balloon and nearly immediately after the balloon was removed.

Structural strength of Binishells depends on four factors during construction. These are: air pressure of the balloon, location of reinforcing steel, concrete weight and distribution, and air temperature during while the concrete is curing. Improper responses to complications of these factors experienced during construction are the causes of the Binishell collapses. Luckily, there was no loss of life in any of the collapses.

Fairvale High School

West Fairfield, Sydney, Australia

Introduction

On January 4th, 1975 the center of the Fairvale High School Binishell collapsed, leaving the perimeter approximately 3-4 meters (10-13 feet) high standing and undamaged. No one was injured in the collapse because damage to the dome was noticed the day before, allowing time for the building to be evacuated. The Fairvale Binishell was a 36 m (118 ft) radius dome, built as part of the 1973 Department of Public Works program. To help manage the program, which was responsible for building 15 Binishells in roughly 4 years, Dante Bini was hired as an architectural consultant. As a result of the Fairvale collapse, an investigation was conducted to determine the strength of the other 36 m domes constructed under the program.

Collapse

On January 2nd, 1975 there were unusually high temperatures in Fairfield, reaching a maximum of 41 C (106 F). Around 4:30 in the evening, an intense rainstorm moved
through the area, causing a drop in temperature of 9 C (48 F) accompanied by strong,
gusting winds. When the storm was analyzed after the collapse, it was estimated that it
caused a temperature gradient of roughly 25 C (77 F) through the shell.

8:30 the next morning, significant cracking was noticed on the crown of the dome in an
area of a circle with a 6 m (20 ft) radius. Additionally, there was spalling on the
underside of the dome, which was beginning to sag at the center in an uneven way. These
warning signs were taken very seriously and the building was evacuated. The very next
day, January 4th, the center of the dome collapsed. When the dome fell, it left a 3-4 m
high perimeter wall which was completely undamaged and was described as very gentle
as it came to rest on tables and blockwork, which showed little evidence of a heavy
impact.

Causes
The collapse of the dome was a surprise to everyone involved. Although the school was
still under construction, the dome standing since October 15th, 1975 and was constructed
using the same techniques as the other Binishells in the area. In response to the Fairvale
collapse, an investigation of the physical properties of the Fairvale dome and the other
program domes was made. The investigation covered dome shape, thickness, concrete
strength, rod reinforcement, and temperature gradient. Despite a few minor areas where
Fairvale differed from the other Binishells, the investigation concluded that the Fairvale
shell collapse was caused by the presence of severe bending moments induced by a large
thermal gradient.

The severe weather on January 2nd set off a chain of events responsible for collapsing the
Fairvale dome. Based on a study performed by the N.S.W. Department of Public Works,
a temperature gradient of 12 C can overstress a 36 m dome like the Fairvale Binishell.
Fairvale experienced a gradient of 25 C, well over the necessary temperature difference.
The bending moments caused by the large thermal gradient resulted in deflections in the
domes shape. This caused a further increase in the dead and live load moments due to the
departure of the Binishells elliptical cross section.

Buckling was the failure state reached by the Fairvale Binishell collapse. When large
bending moments and deflection occur in a member, the compressive stress can be
increased to a level greater than what can be supported. When concrete is overstressed, it
can begin to creep, a deformation of the original shape which disrupts the load path of the
structure. In the gradual failure of Fairvale, concrete which was creeping was relied on to
support the structure. In a dome, the strength of the structure is directly related to the
shape and any departure from the intended geometry is extremely dangerous. The uneven
sagging of the dome the day before the collapse was a dead giveaway that the concrete
was beginning to creep and that failure was imminent. The concrete membrane resisted
the collapse of the dome until the critical state of creep was reached, at which point the
dome fell. The buckling failure in this case was gradual but it has the potential to occur
instantly which can be much more dangerous due to the lack of warning signs.
Lessons Learned
The Fairvale Binishell was still under construction at the time of the collapse. Although the dome had been standing under its own strength for some months, additions and modifications of the shell were still being made during the January storm. Since construction was still taking place, no insulation or weather proofing had been installed on the exterior of the dome.

The insulation is vital to the dome because it prevents possible thermal gradients to occur through the shell of the dome. Prior to the Fairvale collapse, installing the insulation was one of the final steps of the construction process. As a result of the Fairvale collapse, construction procedures were modified to suggest insulating the dome as soon as possible to prevent any effects that inclimate weather could have.

Pittwater High School

Northern Beaches, Sydney, Australia

Introduction
As part of the North South Wales Department of Public Works program responsible for constructing the Fairvale dome, a 36 m Binishell was built and used as the Pittwater High School. The dome at Pittwater was constructed a few months before Fairvale and in response to the collapse, the Pittwater dome was tested and determined to have appropriate strength. However, on August 4th, 1986, roughly 10 years after construction was complete, the Pittwater High School Binishell collapsed, only minutes after the area had been occupied by students. Thankfully only person was reported as injured. The collapse at Pittwater resulted in every Binishell across Australia being temporarily closed and inspected. Following these inspections, Binishells across the country either had supports added or were taken down.

THE CHICAGO CITY POST OFFICE
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Key Words
Abstract
Summary of Events
Causes Of Failure
Possible Prevention and Learning From the Collapse
Consequences

Team 9 Members:
Erin Brobson
Mike Harrison
Andrew Recco
Kate Rosen

Key Words
Abstract

On November 3, 1993, a portion of what was to be Chicago's new main post office, collapsed on the corner of Polk and Canal Street (Lou, July 26 1993). During the building's construction, the structure collapsed, killing 2 ironworkers and injuring 5 others. At the time, workers were laying beams in place before fastening them. One of the insecure beams caused the collapse by creating a chain reaction and pulling down between 60 and 70 other steel beams. Supposedly, there was a miscommunication between the engineer and construction workers when the construction design was changed into a more simplified version (Kendall and Talalay, November 4 1993).

Summary of Events

The Chicago Post Office collapse of 1993 was a devastation to the city and its people. Many workers were injured and some even died because of someone’s ignorance for following the required design process. The architect of the project designed the structure of the building as well as the connections; however, the fabricator made the call on how it would be assembled. Instead of following the architect’s beam to column plan of assembly, the fabricator decided on a quicker, simpler route. The original plan was to erect the columns first using thirty, one inch diameter bolts to secure it to the web. Next, using an erection angle the beams would be connected to the columns using two, one inch diameter bolts. Finally connection plates would be welded to secure the beam and column together. With the design changes made by the fabricator, solely for ease of design, the beams were placed one quarter inch further away from the columns then they were supposed to. This change made the use of the one inch diameter bolts impossible. Instead, workers had to switch to using three quarter inch bolts to secure the beam. The change in hardware led to a series of weak connections, some without nuts even holding on the bolts. The connection point where the collapse began, unsurprisingly, was a connection where a nut was not used. After investigation it turned out that many of the other connections were the same. The fabricator was quickly sued and faced many criminal charges. (Field 1997)(Clark 2009)
Causes Of Failure

The source of the failure has been traced to a temporary connection that failed, which in turn led to collapse of as many as 70 additional members that had already been secured. The actual piece that failed was one of the temporary erection angle pieces, which were used to hold the connections together until they would be permanently connected at a later time. The beams that collapsed due to the connection failure were 32 feet long, some weighing as much as four and a half tons. The area that failed was over 4,900 square feet, and there were over a dozen workers on the area at the time of the collapse. Also, the failure took place on an area of construction that had been built up to the third story with steel members and corrugated metal. At over 50 feet above the ground, there was definitely the possibility of more fatalities. Many of the construction workers who were working on the scene had to “ride” the beams to the ground as they were falling all around them. All of this devastation took place based on the failure of one temporary connection member, that was not even to be part of the final construction. (Clark 2009)

Possible Prevention and Learning From the Collapse
This is one failure where the cause of the collapse as been easily identified and the reason for the collapse occurring has been identified as a miscommunication between designer and steel erector during the design process. The most devastating aspect of this collapse is the fact that it was human error, and a lack of solid teamwork and communication that took the lives of two men, and injured others. The collapse could have been easily averted had the design been thoroughly and comprehensively evaluated by all parties after changes to simplify the connections were made during the design process. Had there been a better understanding of what the design changes were, the collapse never would have happened. (Clark 2009)

A very serious error occurred during the building of the Chicago Post Office, and not only can the profession learn from the disaster, but it is a great example for students to examine. Had more consideration been put into the communication of the design to all members involved, the collapse most likely would have never occurred. There was no issue with the temporary erection angle pieces, there was just an issue in how they were utilized at this particular situation and under the given conditions. Several important lessons were learned from the tragic collapse:

- Make sure there is clear communication between the engineer, fabricator, and construction workers
- Construction workers need to follow the specific construction methods and make sure these methods have not changed
- When laying down beams, secure the bolts with a nut right away

(Nazario 2000)

There remains room for improvement in the construction industry. The Chicago Post Office collapse stood out as an example of why construction work is dangerous. Even now, years after the collapse, the construction industry still has the highest number of fatalities out of all industry sectors (Meyer and Pegula 2004). Though improvements have been made to the equipment used by the construction workers to make them safer, improvements still need to be made to the design phases of many buildings. Equipment, technology, and skills have all been improved, but problems still lie in areas of design and communication. A 1994 study of the UK’s construction industry shows a link between design decisions and safe construction. In fact, 60% of fatal accidents in construction are the result of decisions made before the site work begins. A large portion of construction accidents could have been avoided or reduced with proper design and planning. Ideally, safety should be taken into account during the design phase of a project. Unfortunately, in the US, worker safety is not usually addressed until the construction phase. To better implement accident prevention, designers should receive training on construction safety fundamentals (Hecker 2005).

**Consequences**

The man in charge of construction for the post office was fined by the occupational safety and health administration. He was also charged by the US Department of Justice and US Department of Labor. The architect and engineer of the post office were not found liable
for the collapse. (Nazario 2000).

**BIBLIOGRAPHY**


This is a credible source, coming from the Chicago Sun-Times, which is Chicago’s oldest continuously published paper. The article describes the scene of the building collapse. It includes direct quotes from witnesses who experienced the collapse firsthand. This article also suggests the collapse was caused while workers were bolting beams, which had been hung but not yet welded, in place.


This site gives a brief description of the accident on the Chicago post office and what we learned from the collapse.


The Chicago Tribune, one of Chicago’s primary newspapers, published this article in July, months before construction on the site began. This article gives a brief history on Chicago’s post system, spanning from the 1800s to the 1931 Chicago Post Office. It then describes plans for the future post office, to be finished in 1995 as a small, sleek, and efficient office run with the support of an advanced computer system.


This article concentrates on the individuals in the accident and the cause of it. After two men had been killed and multiple others were hospitalized, investigation began on the reason of the collapse. Thoughts of human error by the engineer and production error by defective bolts were called into question. Many of the hurt workers have sued and are waiting for the investigators to decide whose fault it was.


This article provides details of the events that occurred through newspaper articles. It also explains the consequences of the incident and talks about what went wrong to cause the collapse.


This resource provides some statistics that relate to the collapse itself as well as the parts that failed.


This document explains and proposes ways in which construction safety is being and could be improved.


This article visualizes the collapse and the panic that it caused the workers. Some made it out; however, others were injured and two even killed. It reports the individual injuries of each of the
workers and records reactions of witnesses. The article also references reasons for the collapse; however, so far, the only noticed illegal act was the crane companies misuse of equipment which most likely had no direct effect on the actually collapse.

This article from the Chicago Sun-Times suggests that a sheared seat lug caused the accident. It also describes methods and goals of the officials working on the cleanup site, including precautions will take in dismantling the wreckage. They examine the size, type, and strength of bolts and brackets as well as the workers’ precision in following engineering instructions, the actual engineering design, and other physical evidence.

This article is from the credible Chicago Sun-Times newspaper. It was published a week after the collapse, so it includes further information on the incident, as well as plans for the site cleanup. The article uses credible sources, such as Bill Shook, a spokesman for the general contractor, Hyman-Power Construction Co., who confirmed that 20 representatives from the federal government, contractors and unions held two meetings in which they agreed to a plan for the site cleanup.

This article gives some information about the company in charge of the construction and talks about the charges filed against them.


Additional Resources
This documentary discusses 40 of the worst structural engineering disasters in American History.

Collapse

Inside Pittwater High School after failure
Immediately following the collapse of the Pittwater Binishell the NSW Public Works Department commissioned three engineering firms to determine the cause of the collapse. In one of the reports it was stated that the failure was “sudden and without warning”.
However, the dome had been showing signs of an eminent failure for some time before. In the same report, comments were made on the flattening of the crown of the dome and how it had been getting more and more noticeable during the months and weeks leading up to the collapse. On August 4th, 1986, the Pittwater dome fell in a sudden collapse only minutes after only over 100 students had been in the space.

Cause

Although some of the Pittwater collapse reports have different opinions on some of the engineering concepts responsible for the collapse, they all reach a common ground on what the cause of the failure was. Problems for the Pittwater Binishell began on the first night of construction and were never resolved during the 10 years the dome was standing.

During the first night shift of the construction, one of the blowers used to keep the dome static was accidentally shut off. This caused the partially cured concrete to develop internal buckling. This would cause the concrete to never reach its intended strength and large cracks were formed on the crown of the dome. The engineer on this project realized the problem that was developing and took remedial action. He decided to add a concrete stiffening cap over the entire crown as well as the cracks that had formed.
Despite realizing the problem and acting quickly, the engineer did not consider the bond between the partially cured original concrete and the wet concrete cap. No bonding agent was used and the repair work performed added very little strength to the compromised dome while effectively adding a large dead load. The additional dead load was too great for the pneumatic form to support and the membrane sagged, causing the dome to have a more flattened shape than intended. When the dome was finished, it had the larger than normal radius of curvature, giving it the flattest crown of any of the 36 m domes. Additional air pressure in the form would have supported the additional weight but it would have caused damage to the overall structure and the equipment. The dome was tested upon completion and it was determined to have adequate strength leaving everyone to believe the repair had served its purpose.

In addition to the problems related to the construction difficulties other factors contributing to the collapse were identified in the investigative reports. There was a loss of stiffness of the dome caused by cracking initiated by slight reinforcement corrosion. Additionally, cracking due to moments induced by a thermal gradient were identified. This discovery led to the suggestion that the thermal insulation had been moist prior to collapse, deeming it ineffective. Finally, tubular voids were found enclosed by the springs causing additional cracking. The voids suggest improper vibration techniques during the concrete pouring.

For 10 years, the crown of the Pittwater dome continued to flatten as the concrete began to creep. The strength of the dome weakened more and more as the shell continued to deform further from its intended shape. Finally on August 4th, 1986 years of creep caused the concrete to be overstressed and the dome inverted followed by an instantaneous and dramatic collapse.

**Lessons Learned**

The problems responsible for the Pittwater collapse began during construction when the pneumatic form was partially deflated. This mistake led to the Department of Public Works to employ a new monitoring device that which was not part of the traditional Bininishell equipment. The device was used during the construction of the Binishells constructed after Pittwater and no similar mishaps occurred.
Warning Signs

Several warning signs presented themselves throughout the duration of the Pittwater Binishell but they were either ignored or handled incorrectly. After the collapse of the Fairvale collapse, the Pittwater High School was subjected to a load test. Deflections of the Pittwater Binishell were three times larger than any of the other identical Binishells. This is a clear indication that the dome was not performing as expected but this was ignored and the report determined the dome had adequate strength.

The final warning signs that the Pittwater Binishell gave was the accelerated flattening of the crown leading up to the collapse. When it was observed that the dome was changing shape an engineer should have been notified. Shell structures depend on their shape for strength not the quantity of material in which they are constructed with. If a dome is rapidly changing its shape it is only a matter of time before the structure is over stressed and failure occurs.

Alternative Theories

There are some sources which claim the Pittwater High School collapse to be a result of lightning striking the structure in which the reinforcement steel was not properly grounded (Scott 35). There is little information available to support this theory.

Case A

In 1975, a 120 foot dome suffered a partial collapse of a Binishell just 2 days after construction began. There were uncommonly warm temperatures during the inflation of the pneumatic form. The warm temperature caused the air within the balloon to expand, reaching the desired pressure sooner than expected.
his would not have been an issue if the air temperature did not drop 50 F the next day due to a thunderstorm. The air pressure within the balloon went down with the temperature change and could no longer hold the desired shape of the dome. This resulted in the crown of the dome flattening out over a 40 foot diameter circle. The contractor for this project was inexperienced in Binishell construction and made errors when trying to correct the pressure loss. The contractor immediately began to re-pressurize the form but overcompensated for the loss of pressure and ended up over-inflating the balloon. Multiple iterations of lowering and raising the pressure attempting to reach the original shape resulted in crack forming around the top of the dome. When the form was deflated and removed, the top of the dome inverted and then sheared off the rest of the shell and fell to the ground. Although the middle of the dome collapsed, the exterior of the shell remained standing. (Levy 39)

Note: The majority of the content of this article was taken from the Engineers Australia article "Binishell Collapse - the Inventor's View" and the book published by the Department of Public Works New South Wales "Construction of Binishell Reinforced Concrete Domes New South Wales Australia".

Bibliography:

This is a letter written to Engineers Australia giving his point of view on the cause for the collapse of the Pittwater High School Binishell.

This article covers the information in the three engineering reports written on the causes for the Pittwater High School Binishell collapse.

This website contains information relevant to the research and development of automation in the construction of Binishell systems.

This book describes the techniques used in the early stages of the Binishell dome construction

Hamed, Ehab, Bradford, Mark, Gilbert R. Ian (September 2009). “Time-Dependent and thermal behavior of spherical shallow concrete domes.” Engineering Structures (1919-1929). This journal entry provides outcomes and insight to enhance the effective design and safe use of shallow concrete domes.

Latrobe City Council (2005). Latrobe City Heritage Study 2005: Places of Potential State Significance (1-4). This article describes the historical importance of the Binishell and how it has been implemented over the past 40+ years.


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Summary of Events
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Abstract

Beginning in 1990, Pittsburgh, PA begin the process of expanding their convention center in order to stay competitive in the region. In 1999, after a $750,000 international competition, the design proposed by Rafael Vinoly Architects was chosen from a pool of 25 ideas. A year later, ground was broken for the new David L. Lawrence Convention Center (DLCC). By 2003, the building officially opened even though it had already been used for a few convention events. At its opening, the DLCC was given the Gold LEED certification, making it the first "green" convention center in the world.

The convention center has about 1.5 million square feet of space. That means 1.5 million square feet of room for problems and errors. Multiple collapses occurred early in the building's life, with a truss collapse in 2002 and a lightly-loaded truck falling through the loading deck floor in 2007. Communication errors, lack of organization, and failure to check connections all led to these issues.
On February 12, 2002, a truss on the eastern end of the David L. Lawrence Convention Center collapsed, killing one worker and injuring two others. The truss was the 13th in the set of 15 north-south supports of Phase 3 of the convention center. Phase 1 was set to open February 23, 2002. The collapse killed 38-year-old ironworker Paul Corsi Jr., who was wearing a safety harness hooked to the truss. Though 2" nuts were intended to be used on each 90 foot truss, 1" nuts were used instead, becoming a major contributing factor to the collapse. The incident did not affect the opening of Phase 1, though another later structural failure occurred in February 2007 when a lightly-loaded truck fell through the floor of the loading deck.

Causes of the Failure

The principal north-south support (from Penn Avenue to the Allegheny River) for the convention center is comprised of 15 steel trusses.

During construction, plans were changed for how these trusses were to be supported. The change was to go from using a compression strut to using a tension strut. After this change was made, no clear instructions were put in to the shop drawings to accommodate this change. Because of this, no one on the construction site really knew how to install the trusses.
The Williams Form Engineering Corp. supplied fasteners and joinery for the $354 million construction of the convention center. In addition to the nearly 200 bolts that were provided to connect the steel trusses, Williams supplied two different types of nuts. The heat treated, hardened steel anchoring nuts, which were 2” thick, were colored black and designed to fasten the truss bolts into place. The smaller, weaker locking nuts (of which there were two types), which were either ½” or 1” thick, were silver in color and designed to lock the black anchor nuts into place, nothing more.

On the 13th truss (also called line 13), the locking nuts, in contradiction to their design capabilities, were used to fasten the truss bolts instead of the anchoring nuts due to the lack of clear instructions and direction for which nuts to use, see Figure 3. Furthermore, there was insufficient inspection of the nut-and-bolt assemblies. The only inspection of the assembly was the crew foreman Rogers who inspected them by simply doing a hand check to make sure that the bolts were in securely. He did not check to see if the right nuts and bolts were used.

On February 12, 2002, the misused locking nuts finally gave in to the weight of the structure and came off causing the truss to fall to the ground killing ironworker Paul Corsi Jr. and injuring two others, see Figures 4 and 5.

Prevention
Construction workers used silver, weaker 1" nuts that had been placed onsite instead of the stronger, black 2" nuts; the nuts were not similar in size or color. After the collapse, no one could say why those nuts had even been onsite where Truss 13 was located. Lack of communication between the workers, Dick Corp. management, and Solar Testing Lab was the overarching cause of this failure.

All parties involved claimed that checking the nuts were not their job. Officials from Dick Corp. claimed that inspecting the nuts was the job of Solar Testing Lab, which was hired by Sports & Exhibition Authority (SEA) to monitor the construction project. Even the crew's foreman said that Solar Testing Lab was supposed to test the nuts and bolts, saying, "I'm not a bolt inspector." (Law Office of Michael C. George, "Collapse Theory Gains Support"). However, when a material tes

Figure 5: Onlookers by Truss 13; Courtesy of the Pittsburgh Tribune - Review
ter for Solar Testing Lab, Patrick McKelvey, was interviewed he claimed that no one had told him which nut was correct for the trusses and that he "probably wouldn't have noticed whether the proper nut was used," (Barnes, Tom. (June 26, 2002). "Fasteners Blamed in Collapse Convention Center Supplier Says Wrong Nuts Were Used." Pittsburgh Post-Gazette.).

If there had simply been more communication between the many different workers and companies, this failure could have easily been prevented. The workers would have known exactly what nuts to use and the companies would have known when to deliver what parts. This includes the architects and engineers providing the right documents and the companies distributing these documents so all stages and parts of the process are clear for all parties involved. "Proper and clear erection notes and procedures should have been provided in order to direct the erector on all stages of erection. However, these were clearly missing from shop drawings," (Lavon, Benjamin. Goodrich, Andrew. (November 11, 2009). "Convention Center Steel Truss Collapse." American Society of Civil Engineers Publications.)

Even if communication had stayed the same, someone knowledgeable inspecting the nuts would have caught the blaring mistake. The nuts that were supposed to be used were 2" black nuts as opposed to the nuts that caused the failure, which were smaller, 1" silver nuts. If one of the managers had come through with construction documents to check the truss who knew which nuts were correct, it would have been easy to see the wrong nuts and get them replaced before the collapse ever occured.
Construction sites are complex. There are many different people, parts, and things on one site working to complete one job. The companies and workers involved need to stay informed and communicate with each other in order for the project to be completed efficiently and without mistakes. If policies, laws, and practices are put into place to force everyone to stay connected and informed as to the materials used on each part, failures like this can easily be prevented.

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Lessons Learned From the Failure

Figure 6: Workers by the collapsed truss; Courtesy of the Pittsburgh Tribune-Review

Communication between officers and companies involved in the construction of the DLCC was the overarching cause of the 2002 truss failure. During the investigations many officials realized that nobody had been charged with the job of inspecting the connections. The officials of Solar Testing Labs didn't want to contradict the professionals at Dick Corp. while the officials at Dick Corp. assumed that Solar Testing Labs was doing all the inspections as their crews worked.

Solar Testing Labs, the company hired by SEA to oversee and monitor the project, claimed that it was not their policy to inspect every individual ground connection. In addition, they didn't inspect the connections until months after assembly; they only issued a final "all-clear." Because of the failure and the ensuing investigation, Solar Testing Labs now inspects every connection before issuing the final approval (Franken, Stephanie. (June 26, 2002). "Incorrect use of nuts blamed for collapse." Pittsburgh Tribune-Review.).
No government action was taken after this failure to ensure companies on construction sites communicate better. However, Solar Testing Lab's policy change is a step in the right direction. In order to prevent more of these types of failures from occurring, companies must take the initiative to communicate with their peers and within their own company. However, the public will not know if this occurring or not until another failure occurs from lack of communication.

On the construction side, better bracing of the truss would have helped take the load off the incorrect nuts. This would have given the workers more time to notice their mistake. This could have involved crane hoist lines, guy wires, or other bracing. Also, those erecting the structure must be properly trained. According to Benjamin Lavon and Andrew Goodrich, this training should make sure erectors know how to avoid: "lack of, or confusing erection procedure, unbalanced loads (created) during erection, erection violations of OSHA provisions, lack of proper training of its employees, lack of proper supervision of its employees, lack of proper inspections of its work, use of improper inspections of its work, use of improper hardware for critical connections, lack of proper engineering review, and design or fabrication changes that impact erection procedure," (Lavon, Benjamin. Goodrich, Andrew. (November 11, 2009). "Convention Center Steel Truss Collapse." American Society of Civil Engineers Publications.)

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**Barnes, Jonathan (September 02, 2002). "Inquest Calls For Homicide Charge." Engineering News-Record.**
This article talks about the Pittsburgh coroner's decision to recommend that the death of Paul Corsi Jr. during the truss collapse be changed from accidental to homicide. It further details the charges, to be held against Dick Corporation, the David L. Lawrence Convention Center's steel erection subcontractor, and the rebuttle by Dick Corp.

**Barnes, Jonathan (July 08, 2002). "Inspector Says Contractor Responsible For Checking Nuts." Engineering News-Record.**
This article details testimony during the investigation into the truss collapse at the David L. Lawrence Convention Center. It also deals with who was responsible for putting the smaller nuts on the truss and who was responsible for not checking those nuts.

**Barnes, Jonathan. (July 29, 2002). "Steel Collapse Causes Cited." Engineering News-Record.**
This article details causes of the collapse, including the wrong size nuts and weaker anchor bolts used. It also talks about problems with inspections and installation of the materials.
This newspaper article talks about authority figures trying to calm the public after a truss failure in the new David L. Lawrence Convention Center. The convention center was set to open two weeks after the truss fell, killing one and injuring two others. The article reveals the truss collapse was in a separate phase than the already completed space set to open in a couple weeks. It also reassures the public that the truss failure was not caused by workers being rushed to complete the building.

This newspaper article states that the theory about wrong-sized nuts being the cause of the David L. Lawrence truss collapse is gaining popularity as more information comes out. It details unanswered questions and whether or not charges will be filed in relation to a worker's death.

This newspaper article explains that silver colored nuts that were either 1/2" or 1" were used to install the 15 trusses. The nuts that were supposed to be used were black colored and were 2". The choice should have been made based on plans but no one is really sure as to who should be blamed for the mistake and no one takes responsibility for being in charge.

This article from the Pittsburgh-Post Gazette marks a decisive point in the investigation of the truss collapse, going into further detail about the main cause. The article also covers the questioning of Robert Elmendorf and Matthew Abate, a metallurgist and ironworker, on the collapse as well as the legal issues presented by the death of Paul Corsi Jr.

This site contains detailed specifications and floor plans of the David L. Lawrence Convention Center.

This article talks about the use of the incorrect "jam" nuts and includes interviews with officials from the two main companies involved, Dick Corp. and Solar Testing Labs.

This article talks about the lack of answers found to explain the fatal truss collapse only a few weeks earlier. It summarizes the events and states that the accident will not delay the opening of the separate Phase One section of the Convention Center.


This article is an overview of the failure, starting with information about the new building, continuing with information about the trusses and the accident.

Staff Writer. (February 16, 2002). "Cause of collapse unknown, official says." Pittsburgh Tribune-Review.
This article reveals that faulty anchor bolts might have caused the truss collapse in the David L. Lawrence Convention Center. It also details some aspects of the U.S. Occupational Safety & Health Administration's investigation, such as the fact they have six months to complete the investigation and that they may require authorities to test trusses in other areas of the convention center.

This report details an investigation into the death of Paul Corsi and who is responsible for not checking the bolts and nuts on Truss 13.

Unknown Author. (August 12, 2002). "Two Firms Fined $19,000 Each in Fatal Pittsburgh Collapse." Engineering News-Record.
This article focuses on the U.S. Occupational Safety and Health Administration's decision to fine both Dick Corporation and ADF International Incorporated because of the fatal truss collapse in February 2002. The article details reasons for the fine and what happened during the inquest.
Unknown Author. (September 09, 2002). "OSHA Agrees to Reduce Charges Against Dick Corp. in Collapse." Engineering News-Record.

This article, following the one on August 12, 2002, talks about the U.S. Occupational Safety and Health Administration's lowering of Dick Corporation's fine. The fine was lowered from $12,000 to $19,000 because of insignificant evidence that one of the three safety violations occurred.

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- General Description of the Building
- Construction Activities at the Time of Collapse
- Collapse Summary
- Investigation and Findings
- Lessons Learned
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- Annotated Bibliography

#### 2000 Commonwealth Avenue, Boston Massachusetts (January 25, 1971)

*Kevin T. Wigton, BAE/MAE, Penn State, 2009
*Additional Peer Review in Progress*

## Introduction

The structural failure and eventual collapse of the 2000 Commonwealth Avenue building occurred January, 25 1971. Failure occurred in three stages beginning with punching shear in the uppermost floor of the 16 story building. This was followed by the collapse of the roof slab and the eventual progressive collapse of the east side of the building. Four workers were killed and 20 injured as a result of this failure (Kaminetzky 1991). Immediately after the collapse occurred, rescue efforts were delayed due to uncertainty of structural stability for the remaining building as well as other concerns for rescue workers safety (Blake 1971).

Cause of the building collapse was unknown for some time with three possibilities initially suspected. Formwork collapsing on the upper floors under the weight of freshly poured concrete was one of the first suspected causes of the building collapse. Second, a witness to the building failure named a welding compressor the trigger mechanism to the collapse when it fell from a crane onto an upper floor. Finally, the third theory was that the strength of concrete previously placed in the building was much lower than specified due to the cold weather that Boston had been experiencing in the days prior to January 25 (Cause 1971).

It would eventually be determined that punching shear of the floor slab at an upper level column was the initial trigger mechanism for the collapse (King, Delatte 2004). Many factors
played into the weakened shear strength at this location. Improper shoring of floors, low concrete strength and improper concrete detailing all contributed to the punching shear failure. From the early stages of this building project, there were numerous procedural errors that could have signaled the building's eventual collapse. Followed by flaws in construction procedure, the disregard for standard building practices became one of the major contributors to the collapse. Only one employee was present on site for the general contractor resulting in nearly no quality control or inspection of work. The project suffered greatly from changes in ownership and design professionals during early stages of the project. The lack of controls and monitoring throughout all stages of the building process is attributed to being the underlying cause of the collapse at 2000 Commonwealth Avenue.

![Figure 1: Floor Plan at Failure Location](image)

**Keywords**

Punching Shear, Flat Plate, Construction Failure, Coordination, Progressive Collapse, Condominium, Boston

**General Description of the Building**

When construction began at 2000 Commonwealth Avenue plans were for a new multi story apartment building. The building was to be constructed as cast in place reinforced concrete project with flat slab floors. The building would include 16 upper level floors with a mechanical penthouse, 5ft above the roof slab, and two below grade parking levels (Heger 1972).

This final iteration of the building varied substantially from earlier building plans at this site. Early attempts for the building on this site included a seven story apartment building and then later a 14 story apartment building (King, Delatte 2004). The first permit lapsed due to delays in construction and the second project was never issued a permit for construction due to insufficient information on the application (King, Delatte 2004). With no permit being issued this would later be deemed an abandoned project by the building department. After numerous
changes in ownership a building permit is granted in September of 1969 and construction begins. Lack of consistent designers and owners through this process as well as the irregularities in the permitting process were an early sign of the construction failure that would occur just over a year later.

Construction Activities at the Time of Collapse

Structural concrete work had been completed through floor 16 including the main roof level. Exterior wall masonry work was being performed on the 16th floor. On the morning of January 25th concrete was being placed for the mechanical penthouse floor slab with placement proceeding from west end and moving toward the east (Heger 1972). For an overview of the building floor plan at the failure location see Figure 1.

Collapse Summary

After investigation and interviews conducted by the mayor's Investigation Commission of eye witnesses, it was agreed that collapse of the building happened in three phases. The first phase was the punching shear of the main roof slab at column E5 (King, Delatte 2004). The initial failure was observed by several of the workers at the upper levels. They watched as the sag in the slab around column E5 slowly increased. This was followed by the complete collapse of the east side of the roof slab. See Figure 2 for the initial punching shear failure location. The structure remained in this condition for approximately 10 to 20 minutes, which allowed many of the workers to exit the building or reach safety (Heger 1972). This was followed by the total progressive collapse of the east side of the building, which left the floor plates stacked in the basement of the structure. Figure 3 shows the extent of the collapse on the building's east side.
Investigation and Findings

The primary investigation of the disaster was called for by the Mayor of Boston to find the cause of the fatal collapse. The mayor’s investigating commission enlisted the help of Professor William Litle, Associate Professor of Structural Design at M.I.T. to assist with the structural assessment. An additional notable investigator of the failure was Simpson Gumppertz and Heger Inc. (SGH) primarily conducted by Frank Heger. The SGH report was originally prepared for an insurance adjustor involved in the project.

The mayor’s investigating committee issued their report identifying a number of flaws contributing to the building failure and collapse. Key issues are summarized below.

As previously mentioned there were a number of irregularities associated with the permitting process for the building. The architect or the structural engineer of record did not seal any of the drawings for the project (King, Delatte 2004). Many of these issues can be attributed back to the numerous changes in ownership and designers on the project.
Procedural issues were largely to blame on this project. There was a general contractor for the project but they only had one employee on site during construction. Many of the contracts were issued from the owner directly to the subcontractors (King, Delatte 2004). Given this situation there was next to no inspections or quality control measures on site. The confusion over various design and construction responsibilities was a considerable issue on this project, considered by many to be a contributing factor to the collapse.

There were many deficiencies associated with the concrete mix, reinforcement, placement, and 28 day compressive strength. The mix designs did not meet the necessary prequalification required by the Boston Building Code (King, Delatte, 2004). During the investigation it was found that reinforcement was not correctly placed in many areas and may have contributed to the final collapse of the building (Heger, 1972). After concrete was placed no protection against cold was provided. The roof slab was placed in early December and in the days following the average temperature was 25 degrees Fahrenheit (Heger 1972). As a result of these contributing factors the 28 day compressive strength of concrete was as low as 700 psi (Feld 1997). A final straw was missing special shoring at the area of the initial collapse (Heger 1972).

The design of the project was determined to have been adequate to carry the structural loads of the building. Slab thickness did not meet the deflection criteria but all strength requirements were met (King, Delatte 2004). The investigating committee deemed that the failure would not have occurred if the necessary regulations and procedures were followed (Kaminetzky 1991).

Lessons Learned

Unfortunately, 2000 Commonwealth Avenue would not be the last project that suffered from faulty construction practices leading to a punching shear failure and progressive collapse of the building. Skyline Plaza was one of the first major failures to follow Commonwealth Avenue. Similar to the collapse in Boston this 30 story concrete structure failed due to early removal of shores, insufficient concrete strength, and improper construction planning. Harbour Cay Condominium was another case of a building project that resulted in a collapse during construction. Again procedural errors were largely to blame leading to a punching shear failure and progressive collapse during construction. Immediately following the collapse a summary of the failure at Commonwealth Avenue was available but the details were not widely known. Skyline Plaza and Harbour Cay both could have benefitted from the timely dissemination of information about this failure. For a further discussion of similar failures please review Concrete System Collapses and Failures During Construction.

In recent years the public has questioned if engineers have learned from the mistakes of improper construction practices (Daley 2006). The Commonwealth Avenue collapse would not be the last flawed construction project in Boston. The John Hancock Tower drew scrutiny when the windows did not perform as expected and began to fall from the building. Again in 2006 a major construction project failure struck Boston. A concrete panel fell from a tunnel ceiling on Interstate 90 as a result of a connection failure. In both cases improper detailing lead to poor
Conclusions

A variety of lessons can be learned from the 2000 Commonwealth Avenue collapse. The project, which was flawed from the beginning, went through many stages where deficiencies could have been caught. The culmination of errors on the project from the initial permitting of the building through construction procedures resulted in a major building failure. This failure brought to light the importance of construction controls and inspections on a project. If measures were taken to ensure that the original design was carried out the collapse would not have occurred. The Boston Building Department became aware of the pivotal importance of their role in a construction project with this collapse. There were a number of opportunities for deficiencies to be spotted and all of them went unnoticed.

Annotated Bibliography

Delatte, N. J. (2008). Beyond Failure, forensic case studies for civil engineers, ASCE, Reston, VA. (133-144)

A synopsis of the building failure that reviews causes, collapse details, and implications on the construction industry.


Three chapters from there book are useful in researching the collapse. Failures in flat plate construction as well as those resulting from formwork and low material strengths are discussed here.


The original article which appears agin in Beyond Failure. This article contains additional information regarding the construction proceedings.


This book Discusses the collapse as well as design and responsibility issues on the project.

This article appeared shortly after the collapse and summarizes the three likely causes that were suspected at that time.


The front page headline of *The Boston Globe* which gave the first reports of the building failure


This is an extensive report by Frank Heger that was prepared for an insurance company during the investigation that followed the collapse.


This is a short summary by William Litle which was made public shortly after the collapse and gives the main causes of the building failure.


In this news article, several Boston construction failures are discussed along with the engineering community response that follows.

Additional Resources:


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- Abstract
- Key Words
- Brief Summary of the Events Leading Up to the Collapse
- Causes of the Failure
- Conflicting Accounts
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Abstract

The collapse of Harbour Cay Condominium, a five-story flat-plate reinforced concrete residential building in Cocoa Beach, Florida, occurred around 3:00 PM on March 27, 1981, due to a punching shear failure. Workers were placing the concrete for the roof slab when failure of the slab at one column initiated a collapse of the entire 5th floor. The 5th floor slab fell onto the floors below it, which led to the progressive collapse of the entire structure. Eleven workers were killed and 23 were injured (Lew et al. 1982, p. vii-ix).

Investigation teams concluded that both errors in design and construction led to the tragic failure. On the design side, it was found that the slab thickness fell short of ACI code requirements. The slab was 8 inches thick whereas the minimum slab thickness required by code to account for punching shear was 11 inches. Punching shear was simply not considered in the design (Lew et al. 1982, p. ix). The structural engineer was a retired NASA engineer who neglected to perform several other necessary concrete calculations as well, including checks for deflection, beam shear, and column reinforcement spacing (Delatte 2009, p. 153). Steel reinforcement in the columns was over-congested, which made it difficult for the concrete to flow around the rebar (Feld 1997, p. 273). On the construction side, the chairs used to support the top reinforcement in the slab were too short, which reduced the effective slab depth and hence decreased the slab punching shear strength (Lew et al. 1982, p. 65). Studies found that both the design and construction errors contributed fairly equally to the failure and that the collapse probably would not have occurred had only one of these errors been made (Delatte 2009, p. 153).

Severe cracking of the floors and excessive deflections were brought to the attention of the structural engineer prior to the collapse, but he still confirmed that the structure had sufficient strength (Kaminetzky 1991, p. 74). Construction should have stopped immediately after the warning signs of “spider-web-type cracks” and deflections up to 1 ¾”. Improper reshoring procedures may have also contributed to the structural failure. Additionally, testing showed that the shear strength of the slab near columns was exceeded at numerous locations, and some of the concrete used had a nonuniform consistency (Lew et al. 1982, p. iii-vii). The analysis of the structure concluded that a
punching shear failure occurred on the 5th floor at one of the interior columns, which led to a progressive collapse of the entire slab and hence the entire structure (Lew et al. 1982, p. ix).

Figure 1: Harbour Cay Condominium Before the Collapse (Image courtesy of National Bureau of Standards)

Key Words

Punching shear, flat plate, progressive collapse, concrete strength, failure during construction

Brief Summary of the Events Leading Up to the Collapse

Harbour Cay Condominium was a relatively low-rise building with a length of 242 feet and width of 58 feet (Kaminetzky 1991, p. 72). Figure 1 shows an aerial view of the building before the collapse occurred. When completed, the condominium complex was to feature two five-story buildings, each with forty-five units and twenty-eight
townhouses (Wright 2006, p. 113). Flying forms made up of preassembled plywood decks on aluminum trusses were used to construct the 8-inch-thick floor slabs (Lew et al. 1982, p. 1). The maximum span of the slabs was 27'-8” and 22'-2” in the two directions. A structurally detached elevator tower was located at the east end of the building, and stairwells were located at the north and south ends. Typical exterior columns were 10 in. x 12 in., and typical interior columns were 10 in. x 18 in (Delatte 2009, p. 152). The columns along the exterior ends of the stairwells were 8 in. x 12 in. Story heights were 8’-8” from top of slab to top of slab (Lew et al. 1982, p. 5). All above-grade concrete was specified to have a compressive strength of 4,000 psi (Delatte 2009, p. 152). Pile caps with two to nine piles each supported the columns, and continuous wall footings connected the exterior pile caps (Lew et al. 1982, p. 5). While the roof was planned to be cast in one day, floor slabs were cast in two parts with each comprising one-half of the area of the entire floor (Lew et al. 1982, p. 1). Floors were cast at a rate of approximately one floor per week (Lew et al. 1982, p. 11).

The structure was almost complete at the time of the collapse. Concrete had already been placed on seven of the nine total flying forms for the roof (Lew et al. 1982, p. vii). Workers were finishing the roof slab when they heard a loud crack that sounded like wood splitting. Some eyewitnesses claimed that the center of the fifth floor came down first, while others reported the fourth floor as falling first with the fifth floor collapsing afterward (Kaminetzky 1991, p. 72). The top floors then fell onto the floors below, causing a progressive collapse of the entire structure.

Days prior to the collapse, workers observed and reported several spider-web-type cracks in the floor slabs and excessive deflections of up to 1 ¾”. The structural engineer was informed of the problems and was asked to recheck his design, which he did. However, he still ensured that his design was adequate (Kaminetzky 1991, p. 74). Work on the project proceeded despite these warning signs of a potential failure.
At the time of the collapse, the exterior walls for the first two floors at the south end of the building were in place, and masonry units were piled up on the third floor. Masonry walls for the first three floors in the north end of the building had been constructed, and masonry units were piled up on the fourth floor (Lew et al. 1982, p. 11). When the structure collapsed, reshores were located in the second, third, and fourth stories, but the exact number of reshores present was questionable (Lew et al. 1982, p. 12). The only reshores on the first floor were a few peripheral reshores under the walkways and balconies (Lew et al. 1982, p. 9).

There were 36 workers located throughout the building when the structure collapsed. Figure 2 shows the location of the workers in the building at the time of collapse. Two workers on the ground floor were installing window unit framing, while most of the workers on the third floor were completing masonry walls. One worker in the fourth story was cleaning up debris and two others were making leveling adjustments in the formwork. Workers on the roof were finishing the placement of the concrete slab and were located near the north two bays which had not been poured yet. Workers stated that at the time of the collapse there was no concrete being delivered to the roof (Lew et al. 1982, p. 11).

**Causes of the Failure**
Design Errors

The collapse of Harbour Cay Condominium was attributed to both design errors and construction errors. On the design side, the 8-inch slab thickness fell short of the minimum slab thickness of 11 in. required by ACI code to resist punching shear for the given loads, spans, and column sizes (Kaminetzky 1991, p. 75). Punching shear is the most common mode of failure for flat-plate structures, yet punching shear calculations were simply omitted from the design (Feld 1997, p. 274). A calculation for minimum slab thickness was also not performed. After the collapse, many of the columns remained standing with the floor slabs stacked on top of each other on the ground. This showed further strong evidence of a punching shear failure (Kaminetzky 1991, p. 75). Images of Harbour Cay Condominium after the collapse can be seen in Figure 3 (right) and Figure 5 (below). The only significant loads on the structure at the time of collapse were gravity loads (Feld 1997, p. 274). No evidence of overturning or sidesway of the building was found (Lew et al. 1982, p. 36).

“A punching shear failure happens when the concrete floor slab cracks and breaks away from its column connection. It’s as though you are poking the column through the floor slab like a pencil through a piece of paper” (Modern Marvels 2004). The punching shear strength of a flat slab for a simplified case of an interior column is:

\[ V_c = 4(f'c)^{(1/2)}(b0)(d) \]
\( f'c = \) 28-day cylinder compressive strength of the concrete  
\( d = \) depth of slab (measured from the bottom of the slab to the reinforcing steel location)  
\( b_0 = \) the perimeter of the failure surface around the column measured at distance “\( d \)” from the face of the column  
(Delatte 2009, p. 140)

To increase punching shear strength, the shear perimeter should be increased by using larger columns or column capitals. Also, some of the top (negative-moment tension) steel of slabs should pass through columns in all punching shear cases (Nawy 2008, p. 8-27).

The structural engineer was a retired NASA engineer who hired another retired NASA engineer to perform the calculations (Feld 1997, p. 274). Delatte made a point that “…structural engineering isn’t rocket science. Evidently, it is considerably more difficult” (Delatte 2009, p. 153).

Overall, design errors included:
- There were no calculations for deflection or minimum thickness provisions.
- There were no calculations for punching shear or beam shear.
- There were no code checks for column reinforcement spacing.
- Calculations used Grade 40 steel whereas the structural drawings specified Grade 60 steel.
- There were no actual calculations for the effective depth of slab flexural reinforcement. A constant multiplier of computed moments was used instead (Lew et al. 1982, p. 6).
- Congested column reinforcement prevented concrete from flowing around the steel bars and thus caused a deficient bond between the reinforcement and concrete (Feld 1997, p. 273).
Construction Errors

On the construction side, the top reinforcement steel was placed too low, which reduced the effective slab depth and hence the punching shear capacity of the slab. Figure 4 shows the configuration of chairs and top reinforcement in a column strip within the building. The top reinforcement bars were placed on chairs that were only 4 1/2” high, which reduced the effective slab depth “d” from 6.3 in. to 5.3 in. Hence, the top cover was increased to 1 5/8” whereas it was designed to be 3/4” (Delatte 2009, p. 153). The NBS investigation also found that bottom slab bars were not placed through many columns and that the slabs broke away from the columns where the slabs and columns met. In addition, some vertical reinforcement was found to have been severely bent during fabrication (Lew et al. 1982, p. 32). Laboratory-cured test cylinders were used instead of field-cured test cylinders to determine the actual strength of slabs prior to the stripping of formwork (Kaminetzky 1991, p. 76).

The NBS report also included interviews with workers and witnesses who were present at the time of the collapse. Many disagreements exist as to the location and amount of reshores present in the structure when it collapsed. Some workers said they saw bowed reshores, and others even said they saw reshores break when concrete was being placed on the roof. Due to the numerous discrepancies in the workers’ accounts, it is basically impossible to determine the exact layout of reshores at the time of collapse (Lew et al. 1982, p. 33). Many workers stated that the spider-web-type cracks were noticed once the flying forms were removed. Most cracks were located near midspans and around columns, and some were said to have extended 4 to 5 inches into the floor slabs.
Excessive deflections were reported once the forms were removed. A 1 ¾” deflection was noted in the end apartment on the north side of the building on the second floor (Lew et al. 1982, p. 34). Workers also noted that some of the concrete from the on-site batch plant had a non-uniform consistency and was difficult to finish (Lew et al. 1982, p. 37). One worker stated, “Twenty-two years I’ve been pouring concrete and they’ve never pulled the forms in two days like they did here. They usually set there for a week or 10 days” (Montgomery 1981).

Some investigators wondered why the structure had not collapsed earlier. The shoring and reshoring methods used provided the answer. Shores and reshores initially supported the dead loads of the structure and transferred the loads to the ground. Once the reshores below the first floor level were removed, the concrete slabs were forced to carry the weight of the structure through their punching shear capacity at the columns (Kaminetzky 1991, p. 75).

**Conflicting Accounts**

There were not really any major conflicting accounts of the failure or dissenting opinions of the cause of the failure. Most investigators agree that the main cause of the Harbour Cay Condominium collapse was a punching shear failure which led to a progressive collapse of the entire structure. Since many of the columns were still standing after the collapse, it was a clear indication that the structure collapsed due to a punching shear failure. Investigators agree that punching shear was not accounted for in the design of the structure, and that reinforcing bars were placed too low in the slab. One small argument relates to the concept of design error versus construction error. The National Bureau of Standards concluded that both the design error of the 8-inch slab and the construction error of the inadequate effective depth of reinforcing steel contributed fairly equally to the collapse of the structure. The NBS determined that the structure would have probably not collapsed had only one of these errors been made (Delatte 2009, p. 153). Feld and Carper, however, state “Some construction deficiencies were noted, but the design error related to punching shear alone was clearly sufficient to bring about the collapse” (Feld 1997, p. 274). Basically, Feld and Carper claim that the punching shear design error would have caused the collapse itself, whereas the NBS stated that the building would have stayed standing had only the design error occurred. Also, as previously discussed, eyewitness accounts also offer several dissenting opinions as to the location and number of reshores in the building at the time of collapse and whether the fourth floor or fifth floor came down first.

**Prevention of the Failure**
It is evident that the failure could have easily been prevented. Had the simple routine concrete design checks for punching shear and minimum slab thickness been made, the punching shear failure could have been avoided. The most economical way to increase the punching shear capacity of the slabs would have been to increase the size of the columns. This would also have created more space for casting concrete between the vertical column reinforcement bars (Kaminetzky 1991, p. 75). Increasing the thickness of the slab would have required much more concrete than increasing the size of the columns. Hence, increasing the column sizes would have provided a more economical solution (Delatte 2009, p. 155). Delatte also mentions how the Harbour Cay Condominium failure could have been prevented had the engineers learned the lessons of the 2000 Commonwealth Avenue collapse, which was also caused by a punching shear failure (Delatte 2009, p. 154).

In addition, paying attention to warning signs of a potential collapse is critical. All work on the building should have stopped after the excessive deflections and spider-web-type cracks had formed. Instead, work on the building continued without properly addressing these obvious signs of possible failure.

As mentioned above, the NBS investigators found that if only the main design error or the main construction error had been made, the structure would probably not have collapsed. Therefore, had one of the major errors involved with the structure been eliminated, the building may possibly still be standing.

Lessons Learned

Several lessons can be learned from the collapse of Harbour Cay Condominium. First, punching shear strength must be checked when designing flat slabs, for punching shear is the most common mode of failure for concrete flat slabs. Second, minimum depth of a flat slab must be checked to account for deflection and strength requirements. Next, it is crucial to place reinforcing bars directly within the column periphery to help prevent progressive collapse. This can be done at no additional cost. Furthermore, proper design of formwork, shoring and reshoring plans and schedules, and procedures to verify minimum stripping strength of the concrete by professionals are essential for successful field construction control. Another important lesson is that all work on a project must be stopped if warning signs of potential failure are encountered. Workers should evacuate the building immediately, and professional evaluation of the problems must be performed before work can be resumed. Finally, it is important to use proper test methods to determine the in-place strength of concrete in cold weather. Field-cured test cylinders should be used instead of laboratory-cured test cylinders. The level of construction carelessness also increases during the winter (Kaminetzky 1991, pp. 77-78).

The building industry can also learn the consequences of a catastrophic failure like the
Harbour Cay Condominium collapse. The primary structural engineer on the project, Harold Meeler, surrendered his license and said he would never practice again. Meeler said he would pay the maximum fine of $3,000 to avoid a hearing on the collapse of the structure (Engineer 1981). The other structural engineer also surrendered his license and will never practice in the state of Florida again. The Florida Department of Professional Regulation charged five of the parties involved in the project with negligence. Additionally, two contractors were disciplined, and the architect was suspended from practicing in Florida for ten years. We must remember that major failures in low-rise projects are still possible despite all of the knowledge available to avoid them (Feld 1997, p. 274).

Changes Made in the Industry

The state of Florida strengthened its safety laws after the Harbour Cay Condominium collapse, requiring more on-site inspections by engineers and more scrutiny of construction plans (Modern Marvels 2004). The failure also raised awareness that punching shear failures are the most common type of failure of concrete flat slabs, and accounting for punching shear during the design stage is crucial. The collapse also demonstrated that major catastrophes can still occur with low-rise buildings and not just high-rise structures.

Figure 5: Harbour Cay Condominium Collapse (Image courtesy of National Bureau of Standards)

Could This Collapse Occur Again?
It seems that making a major design error such as omitting punching shear calculations for a concrete flat slab would be rather unlikely today. However, punching shear failures can still occur, especially if formwork is removed before the concrete gains full strength or improper shoring and reshoring is used during construction. The 2000 Commonwealth Avenue collapse and the Bailey’s Crossroads collapse were similar case studies that were both caused by punching shear failures (Delatte 2009, p. 144). Fourteen years after the Harbour Cay catastrophe, the Sampoong Department Store in Seoul, South Korea, collapsed due to a punching shear failure, killing 502 people and injuring more than 900 others. Like the Harbour Cay structure, the store had flat slabs and reinforced concrete columns. Another floor was added to the four-story building in 1989, and the structure collapsed in 1995 after ten tons of air conditioning units were added to the roof. Like the Harbour Cay building, cracks in the concrete were brought to the Sampoong Store owner’s attention days before the collapse, but the warning signs were ignored (Modern Marvels 2004). All of these case studies demonstrate the importance of punching shear capacity and how a punching shear failure can cause a progressive collapse. Adequate slab thickness, concrete strength, and proper placement of reinforcement are essential to prevent such a failure (Delatte 2009, p. 144). The importance of not ignoring warning signs of a potential disaster is evident as well.

**Conclusion**

The Harbour Cay Condominium collapse demonstrates the consequences of improper design and construction procedures. A punching shear failure on the fifth floor initiated a progressive collapse of the entire structure. Punching shear calculations were omitted by the structural engineer when the structure was designed. Reinforcement bars were placed too low in the concrete slabs, which reduced the effective depth of the slabs and hence reduced the overall strength of the slabs as well. Warning signs of a potential failure were brought to the attention of supervisors and the structural engineer but were basically ignored. The Harbour Cay disaster could have easily been prevented had simple design checks and careful construction techniques been performed.

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L'Ambiance Plaza (April 23, 1987)

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Introduction

On April 23, 1987 the L’Ambiance Plaza building, in Bridgeport, Connecticut, collapsed during construction. This collapse spurred a large scale, eight day, rescue attempt and ultimately left 28 workers dead.(Moncarz, 1992) This 16 story building, 13 apartment levels over 3 parking levels, was being constructed using the lift-slab method. The lift-slab method consists of casting post-tensioned floor slabs, one on-top of another, at ground level and then hydraulically jacking each level into place. At approximately 1:30pm on April 23rd a loud bang was heard and within the next 2-10 seconds the entire building crashed to the ground. The collapse launched several investigations but was quickly settled out of court ending all investigations and leaving the exact cause of the collapse unknown. Although the exact cause of the collapse remains unknown, five viable theories have been proposed in the years since the collapse by various experts on building failures.(Schribner, 1988)

Description of Building

L’Ambiance Plaza was planned to have two virtually identical towers, with floor plans measuring approximately 63 ft by 112ft, and a neighboring parking garage. The parking garage had not begun construction and was not involved in the collapse. (Korman, 1988) The towers were separated by 4 ft and would have been joined by cast-in-place concrete during the final stage of construction. The structural system consisted of steel columns and 7” thick two-way unbonded post-tensioned concrete flat plates with shearwalls at
four perimeter and four interior locations (Culver, 1987). Generally the Youzt-Slick method of lift-slab construction, which was employed on L'Ambiance Plaza, is used in the design of two to five story buildings. However, there have been taller structures built using this method. (Cuoco, 1992) The towers are designated the east and west buildings and will be referred to as such throughout this case study.

The Lift-Slab process used at L'Ambiance Plaza consisted of casting floor slabs at ground level, raising those floor slabs to the desired elevation with hydraulic jacks, and fixing the slabs in position mechanically. By casting the floor slabs on grade, the need for shoring or formwork underneath each slab was eliminated, and only side forms were required. Slabs were raised in packages and parked at designated levels by a prescribed schedule of construction. The packages were "parked" at a level by 3 workers, one operating the jack, one monitoring the level from underneath, and a third placing in steel wedges and tack welding them in place. Packages of floor slabs were "parked" at various levels until shearwalls could be cast to provide stability, and steel erection could continue to higher levels. (Culver, 1987).

Figure 2: Typical Floor Slab Plan (Culver, 1987)

Day of the Collapse

The post-tensioned floor slabs were grouped in packages of 2-3 floors each, which were to be jacked to various levels during the construction period before being set at their final assigned level. These positions were preplanned in accordance with stages of construction. The morning of the collapse the building was in Stage IV of construction. At this point in Stage IV, the west building package containing floor slabs 9-11 were jacked into position at level six approximately 8" below the package with the slabs for floor 12 and the roof (Cuoco, 1992). The ironworkers were working to tack weld wedges under the 9-11 package to temporarily hold it in place. They were working on this through out the day. After lunch at approximately 1:00pm workers used a 12-ton
horizontal jack between the two towers to plumb the West building (Masih, 1995). Around 1:30 pm the ironworker, Kenneth Shepard (Martin, Delatte, 2000), who was installing wedges in the west building, heard the first loud bang and looked up to see the concrete above him "cracking like ice breaking." (Cuoco, 1992) The floor slab above him the proceeded to collapse onto the levels below. Within 2-10 seconds both towers had collapsed completely. The west building fell first, followed shortly by the east building. Both buildings collapsed in a similar, "pancake" manner settling almost entirely with in the plan of the building towards their respective centers.

Figure 3: Slab Locations Prior to Jacking of West Tower 9/10/11 Package courtesy of NIST

Causes of Collapse
Theory 1: Instability of the wedges supporting the 12th floor and roof package

- Thornton Tomasetti Engineers

Thornton-Tomasetti Engineers' concluded the epicenter of the collapse was a core column, 3E, of the west building. Wedges supporting the 12th floor and roof package at the column were unstable and started the collapse. They state in their "Collapse Scenario" that a wedge supporting the 12/R package rolled out leaving the shearhead at this level supported by a single wedge. The horizontal load from the jack used to plumb the building caused the remaining wedge to roll slightly as evidenced by rounding and bending in the west weld block of the shearhead. Additional movement of the slabs may have caused the remaining wedge to roll completely out. (Cuoco, 1992)

Figure 4: Structure Geometry Prior to Collapse - Drawn by Liam McNamara based on description from Culver, 1987
In the absence of both wedges the package would have dropped the 8 in. onto the top of the package containing floors 9-11. The loud bang was heard when the lifting nuts initially supporting only floors 9-11, slipped out under the additional load and impacted the web of column 3E. The slabs and shearheads then began to slide down the column, impacting the floor slabs below, ripping them from the column, and progressively collapsing the west tower. The catenary action of the post-tensioned cables then pulled the remaining columns towards column 3E. The east building then collapsed from horizontal forces transmitted through the pour strips or horizontal jack, or impact of the west tower debris. (Cuoco, 1992)

The physical evidence supporting their theory included the discovery of abnormal tack welds on the wedges supporting the 12/roof package as well as shearhead gaps on columns 3E and 3.8E(0.628 in.) that were much larger than those on the rest of the building (0.233 - 0.327 in.) and other buildings built using the lift-slab method (0.250-0.375in). These large gaps as well as the presence of hydraulic fuel would have reduced the friction normally depended upon to hold the wedges in place until they can be completely welded.(Cuoco, 1992)

**Theory 2: Jack rod and lifting nut slipped out due to a deformation of an overloaded steel angle welded to a shear head arm channel**

-National Bureau of Standards (NBS)

The NBS concluded in their investigation that the failure began at the building's most heavily loaded column, E4.8 or the adjacent column, E3.8, as a result of a lifting assembly failure. At each column the shearhead reinforces the concrete slab, transfers vertical load from the slab into the column, and provides a place of attachment for the lifting assembly. Steel channels are cast in the slab, allowing room for the lifting angle. Lifting rods, raised by hydraulic jacks above them, are passed through holes in the lifting angle and fastened with lifting nuts. (Scribner, 1988)
Figure 5: Shearhead and Hydraulic Jack (Martin, Delatte 2000)
NBS testing determined that when the shearhead and lifting angles were loaded with forces nearing 80 tons, they had a tendency to twist. This was due to a lack of stiffness, not strength. During the lifting process the shearheads and lifting angles were loaded close to their maximum capacity. The angles deformed under the excess force of the three 320 ton slabs, causing the jack rod and lifting nut to slip out of the angle and hit the column. This contact would have produced the loud bang heard by Kenneth Shepard. (Martin, Delatte, 2000)

The building then collapsed in the same method as described by T-T after the lifting nuts impacted the column.

**Theory 3: Improper design of post-tensioning tendons**

-Schupack Suarez Engineers, Inc.

Schupack Suarez Engineers examined the unusual layout of the post-tensioning tendons in the west building. The east building’s tendons were run in a typical two-way banded layout, uniform tendons running North - South carry the slab load to the East- West Column line, where the E-W banded tendons then "pick up" the load and transfer it to the columns. However, at column 4.8E in the west building, the E-W tendons split around the column line. The absence of tendons in Line E due to the split at the column added
increased load into the structure. The design details also did not include the location of the shear walls or the openings for the walls at Columns 11A, 8A, and 2H (Poston, 1991).

Finite-element analysis determined that the tensile stresses along Column Line E, east of Column 4.8E, exceeded the cracking strength of the concrete. By this reasoning once a crack was initiated it would spread immediately to Column 4.8E. The finite-element analysis also showed that even under ideal lifting circumstances column 2H would have had unsuitably high compressive and punching shear stresses. (Poston, 1991)

Theory 4: Substandard welds and questionable weld details
-Occupational Safety and Health Administration (OSHA)

OSHA stated that two weld design details were questionable. These welds were the 1/2 in. single-bevel-groove weld between the arm channel and the lifting angle in the shearheads, and the one-sided square-groove weld connecting the header bar and header channel at the shearhead. Both welds were of unknown and unspecified penetration depth and therefore had an unpredictable strength. The single-bevel-groove weld could also have been further weakened from flush grinding. The one-sided square-groove welds are even more questionable due to the fact that they were not among the American Welding Society prequalified joints. (McGuire, 1992)

OSHA investigators found a shearhead with a failed single-bevel-groove separated completely from the arm channel. This shearhead belonged to column E3.8 previously identified as a possible epicenter for the collapse. Upon further investigation by OSHA hired consultant firm, Neal S. Moreton and Associates, it was determined that of 30 welds investigated at Column E3.8, levels 7, 8, and 10; 17 welds did not meet industry standards. (McGuire, 1992)

Theory 5: Global instability caused by lateral displacement
-Failure Analysis Associates, Inc. (FaAA)

FaAA consultants focused on the response of lateral loading and overall torsional instability. The shearhead connection is rotationally stiff when the concrete slab is temporarily resting on the wedges and when it is fully welded in its final position it becomes a rigid connection. However, when the slab is lifted off the wedge it can rotate freely. In the absence of lateral loading the building would be completely stable. In the presence of lateral loading or displacement, such as that from the horizontal jacking just after lunch, the slab could be lifted off a wedge and the building would become laterally flexible. FaAA used 3D computer modeling (ANSYS) and nonlinear stability modeling to investigate this possibility. Upon analysis of their modeling FaAA concluded that
lateral instability was the cause of collapse for both the west and east buildings.(Moncarz, 1992)

**Conclusion**

Due to the relatively quick settlement of the case, many possible lessons will never be learned from this terrible collapse. The lift-slab method whose use is responsible for approximately 45,000,000 sqft of safe buildings, suffered greatly from this collapse, never making a come-back in the United States (Cuoco, 1992). The collapse calls into question why critical connection details don't get enough attention from engineers, as well as revealing a need for greater attention to temporary construction load. This failure shows the need for clear connection details during the design phase, as well as a need for lateral bracing during construction, and strict following of the specified construction phases (Korman, 1987). Collapses such as L'Ambiance Plaza should be used as building tools for future construction, the building industry must continue to learn from its mistakes so as to prevent them in the future.

A lack of communication between several subcontractors and the engineer was also a major issue in the L'Ambiance Plaza collapse. Responsibility for design was fragmented among so many subcontractors that several design deficiencies went undetected. If the engineer of record had taken responsibility for the overall design of the building or a second engineer had reviewed the design plans, these defects probably would have been detected (Heger 1991). The L'Ambiance Plaza case was mediated by a two-judge panel who determined a univeral settlement between 100 parties, which closed the case. Twenty of more separate parties were found guilty of “widespread negligence, carelessness, sloppy practices, and complacency.” All parties contributed to the $41,000,000 settlement fund in various amounts. The families of those killed in the collapse, and the workers injured received $30,000,000 in the settlement (Martin, Delatte, 2000.)

Prior to the collapse of L'Ambiance Plaza, Connecticut had no provisions in their building regulations for an independent review of building structural designs. Generally, building authorities do not have an adequate staff to review structural designs, and there were no requirements for peer review. An improvement made in Connecticut as a result of the collapse was to require review of structural designs by an independent engineer accepted by the building authority (Heger, 1991).

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**Other Lift Slab Structures of Note:**

Although the collapse of L'Ambiance Plaza effectively ended the use of lift slab
construction in the US, a number of structures had been built prior to April 23, 1987. Many of these were successful and are still in service today. Others experienced problems such as column-wedge failures, global instability (sidesway failure mode) or related construction issues that resulted in collapse or remediation during construction. Fortunately there was no loss of life with these cases, however there were injuries and significant economic losses. Examples of several problem Lift-Slab are noted below:

- On July 15, 1954, the Junipero Serra High School Roof in San Mateo, California collapsed due to sidesway instability. This 16 ft. tall one-story building was approximately 65 ft. x 70 ft. in plan and used 6 inch diameter steel pipe columns for support. Attempts by the contractor to stabilize the leaning structure with guy wires on one side of the frame backfired as the contractor apparently overcompensated and failed the building in sidesway in the direction opposite the original lean (Zallen and Peraza 2003, pg 24)

- A structure based on the Canadian wedge system (a system that relies on frictional resistance between the column and wedges) collapsed under construction in Marion, Indiana in 1962 (Delatte 2009 pp 120-121).

- Cleveland, Ohio experienced a near collapse on April 6, 1956 when the Pigeonhole Parking Garage (using the Youtz-Slick lifting system) nearly collapsed in winds of 35-65 miles per hour prior to the steel wedges being permanently welded. Fortunately the contractor was able to right the structure by pulling it back into plumb and finish the project without further incident even though the building was leaning as far as 7 feet out of plumb (Zallen and Peraza 2003, pg 24-25; Delatte 2009 pg 121). An expanded account of the Pigeonhole Parking Garage case, including dramatic original photographs of the leaning structure, can be found on the Failures Case Studies website of the (NSF MatDL) created by Dr. Norbert Dellatte.

**Early Lift Slab Construction:**

Photographs of what is believed to be one of the earliest, if not the first, lift slab structure constructed in the US, are shown below. Provided by former Professor Vincent L. Pass, P.E. of Penn State AE, the photographs are that of the Trinity University Library, San Antonio, Texas and were taken in April of 1951.
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- Construction collapse
- Crossbeam
- Formwork
- Lateral Force
- Diagonal Bracing Member
- Temporary Supporting Structure
- Shoring system

BRIEF OVERVIEW

The New York Coliseum was an exhibition hall built during the 1950's in New York City. During the construction in 1955, 10,000 square feet of the main hall collapsed (Kaminetzky, 1991). Several of the crossbeams connecting the posts of the formwork did not offer enough protection against the lateral forces. The workers used the crossbeams to transport concrete around the construction site via buggies. The live load of the buggies was too heavy for the crossbeams, causing the collapse. Unfortunately, one worker was killed and fifty others were injured as a result of the collapse. Interestingly enough, the formwork was built to withstand the live loads of the workers but was not reevaluated once the buggies were introduced.
HISTORY

Designed by architects Leon and Lionel Levy, the New York Coliseum was built by the Triborough Bridge and Tunnel Authority under the direction of city planner Robert Moses. Moses condemned the west side of the Columbus Circle after an earlier project had failed (Gray, 1992). He then got the Federal Government to pay the acquisition costs required to buy the remaining lots - old offices, tenements, and minor retail stores. The design of the Coliseum, essentially a low rectangular exhibition space attached to a taller block of office spaces, has often been criticized for being "utterly pedestrian" and not relating at all to the curve of Columbus Circle. The Coliseum was covered with unadorned white brick and metal paneling.

Construction began in 1954 with the collapse occurring in 1955. The Coliseum was finally finished in 1956 and opened with the capacity to house six different shows simultaneously. The opening exhibits were the New York International Auto Show, the National Photographic Show, and the Fifth International Philatelic Exhibition (Gray, 1992). In 2000 the New York Coliseum was demolished in order to build the AOL Time Warner Center (Gray, 1992).
CAUSES OF THE COLLAPSE

The New York Coliseum was a concrete structure, which consisted of twenty-inch think waffle slab floor (McKaig, 1962). The form of the floor was two stories high and supported by timber shores connected to a crossbeam. However, the crossbeams did not brace against lateral instability. Later, when twelve 3,000-pound power buggies were driven across the formwork at twelve miles per hour they imposed a lateral thrust onto the deck (Purva, 2010). The lack of diagonal bracing members to transfer the lateral force caused the structure to sway and fail without warning. In essence, there was an inadequacy for the temporary supporting structure to maintain the load of the buggies under the pressure of a horizontal or oblique thrust on the main exhibition floor. The movement and sudden stopping of the buggies created this thrust. Other activities such as dumping concrete on the floor also attributed to the structural failure.

POSSIBLE PREVENTION AND LEARNING FROM THE COLLAPSE

Unfortunately, this failure could have been prevented or at the very least held to minimal
damage, provided that they had appropriate lateral and horizontal bracing of the temporary support structure. From this failure, we learned that shoring systems should be well braced to support lateral dynamic loads and that we should always recalculate to account for the effects of moving power carts or related modern equipment on temporary framework. As a result of the Coliseum collapse, the industry tightened formwork regulation and increased the number of proper inspection facilities on construction sites. Specifically, the district attorney made recommendations to revise the building codes about temporary formwork structures.

Despite the tragedy of the New York Coliseum collapse, formwork failures still occur on large construction projects. For example, in Japan a temporary shoring system collapsed occurred during concrete placement in 1992 (Carper, 1997). Materials from the second floor fell into a ground floor swimming pool when 800 tons of material was being transported. This resulted in seven deaths and thirteen other workers sustained injuries. Despite the precedent set by the New York Coliseum, temporary formwork is not given as much care as the actual structure, even though formwork often represents over half the cost of reinforced concrete structures. Even with all the required codes in place, formwork collapses depend on how much time and attention the construction managers spend building and inspecting the temporary formwork.

RELATED LINKS

Concrete System Collapses and Failures During Construction

Tropacana Casino Parking Garage Collapse

Indian Bridge Collapse (December 2009): A bridge construction in India recently collapsed killing 30 people and trapping 20. This more modern example shows how construction collapses can occur even now if the proper attention is not given during early construction work.

BIBLIOGRAPHY


This book provided information about other similar structural failures. We used an example of a Japanese construction collapse similar to the New York Coliseum to show how temporary formwork problems still can occur. <http://books.google.com/books?id=-jnlb-oJxcEC&printsec=frontcover&dq=Construction+Failure&hl=en&ei=VdjiS5a8HYWBlAf75oGVAg&sa=X&oi=book_result&ct=result&resnum=1&ved=0CDgQ6AEwA>

This publication provides historical context and forensic engineering analysis to give a new point of view on the civil engineering failures of the New York Coliseum.


This Times article provides a brief written history on the New York Coliseum. It includes details about how the project was started and what has happened to the building since the construction finished. It also provides some criticism of the buildings utilitarian purposes and aesthetic functions.


This book provides information about the technical details of the construction. Also, it provided some solutions on how to prevent similar failures.


This book provides a detailed technical look at the formwork structure that collapsed during the construction stage. As well as providing dimensioning and sizes, the book also paraphrases the official statements made about the collapse.


This article talks about the injuries sustained by the workers during the construction collapse and the causes of the structural failure. It also provides a brief overview of the lessons learned from the building failure.

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Introduction

The Rosemont Horizon Arena collapsed during construction on August 13th, 1979. Located directly under a flight path to Chicago's O'Hare Airport, the arena's design employed a wood roof to dampen the resultant noise pollution. The roof was comprised of sixteen glue laminated arches spanning 288 feet, split into three trusses, joined by metal and braced laterally by wooden purlins. The arches were poorly aligned and varied as much as twelve inches laterally across a single span. This alignment issue prevented the purlins from being adequately connected to the arches; "Over 53 percent of the required connection bolts were missing from the building's roof" (Nazario). The general
instability of the incomplete wooden roof frame combined with a gust of wind brought the construction to the ground, resulting in the deaths of five laborers and the injury of sixteen others. The arena was swiftly rebuilt using a different contractor and adhering to the connection requirements as originally specified by the architect and engineer. The Horizon Arena (now the Allstate Arena) has served the greater Chicago area for nearly 30 years, proving that it was construction techniques, not design failure, that brought the original structure to the ground.

**Keywords**: timber roof, construction oversights, wind, Rosemont Horizon Arena

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**Structure**

The Horizon Arena was constructed with a timber roof comprised of sixteen 288' glue-laminated arches, laterally braced by wood purlins. These large arches were 6' deep by 1' wide and were made up of three separate pieces and erected in three different stages. Connecting the arches laterally to each other were smaller wooden girders, spanning approximately 20'. The roof was supported and buttressed by concrete columns situated at the ends of the arches. As construction workers moved along, they didn't place all the nuts and bolts in position, because they wanted to let the natural deflection of the wood set in before doing so. By making this decision, each new arch became progressively off center, and in combination with being ill-supported, the first arch fell and caused a domino effect that accordingly brought the other arches down as well.

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**Collapse**

A westward gust of wind perpendicular to the arches was responsible for starting the collapse. The workers had begun work on the sixteenth and final arch, which was not tied laterally to the previous fifteen. As the wind brought the first arch down, a domino effect ensued, and within seconds the roof structure was on the ground (figure 2).
Causes of Failure

Construction Oversight
The timber roof was dependent on the fasteners, as wood tends to rotate, bend, and warp with time. In fact, in the initial stages of construction, workers noted that the void between the arches varied as much as twelve inches across a single span. This presented additional difficulty in attaching the lateral purlins, which were all cut to a uniform size. Many improvised connection methods were used to make up for this shortcoming; the use of drift pins in place of the specified bolts, cutting new holes for the connections with torches, and even omitting bolts where exceptional difficulty was noted. Although too late now, any on site decisions should have been checked by at least an engineer and perhaps the designer as well.

Fasteners
The difficulties of alignment had a direct effect on how the structure was assembled. The supervising contractor made the decision to omit fasteners to allow the wood structure to deflect downward as much as possible before adding the final connections (see unfamiliarity subsection for rationale). Of the 966 specified bolts, over 54% were missing. Out of the 444 bolts that were installed, 338 did not have nuts (Carper). This leaves a total of 106 properly assembled fasteners, only 11% of the intended amount.

Unfamiliarity
The contractors hired for the Horizon Arena were from the Chicago area. Builders in this region are accustomed to working with steel structures. It was common practice to leave out some of the bolts during construction of steel structures to account for the stretching and bending of the steel as loads are placed on it. The contractor's unfamiliarity with the demands of a wood structure played a key role in the collapse, however, it is no excuse. The contractor should have at the very least discussed this with a structural engineer and/or done research into it as well.

**Additional Causes**
A twenty mile per hour westward gust of wind caused the final arch to fall. The arch fell to the west, causing the other fifteen arches to collapse with it. Following the incident, an OSHA investigation led to the discovery of another factor in the failure; the additional weight of construction materials improperly stockpiled on the top of the incomplete roof. There has also been speculation that perhaps the rapid and continued plane flights overhead may have in some way have affected the building.

**Consequences**
The roof collapse resulted in five deaths and sixteen injuries. Financially, the collapse added $3 million dollars to the total cost of the arena, which was initially projected at $8 million dollars. OSHA levied fines on the contractor for improper construction practices and the architect and subcontractors for irregularities in the building as it was built and the building as it was designed and approved. Additionally, the engineering firm that was hired to investigate the collapse was fined for exposing their employees to the hazards found on the building site (Nazario).

**Lessons Learned**
Communication is of utmost important on large projects such as this one. As was shown by the new contractors after the collapse, the original design was fine and structurally sound. However, it was the original construction crew that failed to build the roof as it was planned. Had the architect(s) and construction workers perhaps collaborated better, and checked each others work as they progressed, the original roof could have been a success.

**Conclusion**
Immediately following the collapse, the decision was made to rebuild the Horizon Arena. The architect and client hired a new contractor to displace the previous one, and the arena
was constructed as per the original designs. Consideration of the special needs of timber, as well as a steadfast adherence to the numerous connections in the roof design resulted in success. Today, the Rosemont Horizon Arena (now known as the Allstate Arena) still stands, having served the Chicago area, sports teams, and traveling musicians and their fans for nearly thirty years.

Bibliography

The supervising engineer approved the omission of bolts, hoping the arches would deflect under the imposed loads and allow the installation of the remaining bolts. These omissions, combined with inadequate bracing and the storage of construction materials on the roof, resulted in the collapse of the structure.

This article contains many facts and figures on the collapse, from the number of missing bolts to how many were out of place. It also mentions that the total cost of damage done was $3 million, and that the roof was later built to specification by a different contractor and it was successful.

3. "Designers assess reason for collapse of arena roof that killed 5 workmen."
*Eugene Register-Guard* 14 Aug. 1979: 5A.
An architect with the firm that designed the arena called the claim that a low-flying aircraft, which passed over the structure moments before its collapse, "wild and premature." A design consultant for the roof’s construction did not rule out the possible effects of wind on the structure, asserting that the unfinished roof could have acted as a sail.

Sixteen glue laminated arches spanning 288 feet were installed, in three pieces each, and were raised on precast concrete buttresses. The metal plates connecting the wood to the concrete did not have the proper amount of nuts and bolts installed.

Investigation shows that during construction the sixteen wood arches were not properly braced. Many bolts were missing and as more information was revealed, it became clear that the construction job was nothing less than faulty. This allowed a wind load to cause a collapse.

Having been written only two days after the collapse, this article first points out that
engineers were still investigating. It then begins to question the source of the problem. Although not yet known, Gorman notes that speculators think the high frequency of planes in the area could of had an effect. He also describes in detail the long arches that traversed the stadium.

The contracting firm, being from Chicago and therefore more accustomed to steel construction, did not feel it was necessary to utilize all the bolts in the connection between arches and purlins. In steel construction, it is generally accepted to omit some of the connecting bolts due to the inherent complexity of the structures. However, in a wooden construction, all connections are of an absolutely critical nature because the wooden arches needed the lateral support offered by the purlins.


Tropicana Casino Parking Garage
Atlantic City, New Jersey - October 30, 2003
Robyn H. Engel, Dustin G. Julius, Laura E. Rakiewicz, and Kaite E. Simmons (Wiki Group 10), AE 210, Spring 2010

I. INTRODUCTION

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At 10:40 a.m. EST on October 30, 2003, part of the Tropicana Casino Parking Garage in Atlantic City, New Jersey crashed to the ground (See Figure #1). As the name suggests, the structure served the purpose of a parking facility for the Tropicana Casino and Resort. The structure was incomplete at the time of the crash, yet had an expected completion date of March the following year. The project began in April of 2002 and its design was composed of various aspect that referenced Old Havana, Cuba. The crash came to a group of unsuspecting workers who were pouring concrete at the time. Of the 300-400 workers at the sight, over 20 were injured and 4 were found dead. Five of the building’s ten stories collapsed as a result of a failure to provide adequate temporary supports until the concrete dried. Also, the steel reinforcements in the concrete were not properly anchored to its supporting columns.

Figure #1: Image showing the front facade of the Tropicana Casino Parking Garage after the collapse (Courtesy of the D'Amato Law Firm).

II. KEY WORDS
• Fabi Construction
• Tropicana
• Casino
• Parking Garage
• Atlantic City
• hotel
• shoring
• rebar

III. EVENTS LEADING UP TO THE COLLAPSE

Parking Garage Background

The Tropicana Parking Garage collapse occurred during the construction of the 2,400-space lot (“4 Dead,” 2003, 2). The ten-story garage was part of a thirteen-story structure that included a 502-room hotel (Murphy, 2003, 1)(CTL)(Lipton “Workers,” 2003, 1). The construction included stay-in-place precast, pre-stressed formwork as well as a cast-in-place composite floor system. Cast-in-place columns and sheer walls were connected with mild steel reinforced rebar (CTL). The garage was a partially precast, pour-in-place reinforced concrete structure (Foley 1).

Hotel Expansion

The expansion of the Tropicana hotel was led by the Keating Construction Corporation (Murphy, 2003, 2). The project contractor was Fabi Construction Incorporated of Egg Harbor Township, NJ (Lipton “OSHA,” 2004, 1). The undertaking was intended to evoke images of Old Havana Cuba (“4 Dead,” 2003, 2). Building of the $245 million expansion began in April of 2002 and was expected to be completed in 2004 (“4 Dead,” 2003, 2)(Murphy, 2003, 2).

Design and Later Alterations

The original design was altered during construction, an event that has been pegged as a leading component of the collapse. The original design included 2.5” thick precast concrete panels reinforced with metal wire trusses (Foley 1). Styrofoam blocks were employed to create void spaces in the concrete (Foley 1). Assembly occurred one 44’ bay at a time. The design changes included individual rods of rebar being swapped for factory-made, 8’ mats of rebar (Lipton “Changes,” 2004, 2). Additionally, the beams were made shallower and wider (Lipton “Changes,” 2004, 2)

Just Before the Collapse

A group of between 300 and 400 workers were on the construction site Thursday October
30th working to pour concrete (Murphy, 2003, 1). Several days earlier, some of the workers had noticed a few issues with some of the construction that had already been completed. There were numerous cracks in some of the recently poured concrete floors and as well as some of the bent vertical poles that supported these floors (Hanley, 2003). These workers made the contractors aware of the dangerous construction flaws, however the issues were ignored (Lipton “OSHA,” 2004, 1).

![Figure #2: An axial view of the parking garage showing the top floor as it caved into the other floors yet remained hinged on one side (Courtesy of the D'Amato Law Firm).](image)

IV. CAUSES OF THE FAILURE
Collapsing Details:

A worker was using a crane-mounted hose to pour a 60 feet wide area of concrete floor. At this moment in construction, the supports are under the most intense amount of stress because the wet concrete cannot support itself and it also contains the added weight of the unevaporated water in the concrete. Wet concrete weighs about 160 pounds per square foot (Lipton “Changes,” 2004, 2). Half a dozen metal pogo-stick-like pole devices that temporarily hold up the concrete floors until they harden enough to support themselves
had somehow been bent out of shape. This implied that the floors were slightly moving (Lipton “Workers,” 2003, 1). After these poles snapped, after ten o’clock in the morning, the building started to collapse (“4 Dead,” 2003, 1) when a concrete floor on the top level began to fail (See Figure #2). Next, the complex experienced a domino effect. Several precast form works as well some of the composite floors began to fail on multiple levels of the building. In total, five stories of the parking garage collapsed. Only the columns and sheer walls around the perimeter remained standing (CTL). The collapse stopped at fifth floor and the walls below absorbed the force of the collapse (Foley 2).

Structural Failure:

The failure occurred somewhere between the outer walls and the wide slab panels (See Figure #3). The columns failed either at or below the floor being poured. As weight in the building transferred, the remaining column and floor connections sheared on the outer wall, causing a lean-to collapse pattern between the first and second bays (Foley 2).

Figure #3: The front facade of the existing casino complex and the adjacent site of the building failure (Courtesy of the D’Amato Law Firm).

V. CONFLICTING ACCOUNTS
The sources used in this article have been compared and compiled with one another. The CNN.com article “4 Dead” claimed that the actual time of the crash was 10:30 am EST. The others, including Jarret Murphy’s article “4th Body Found In Garage Collapse,” said that the building fell at 10:40 am EST. The latter is assumed in this article.

VI. PREVENTION

The Tropicana Casino Parking Garage collapse could have easily been prevented. Several workers reported the initial splices in some of the concrete floors and columns (Lipton “OSHA,” 2004, 1). The contractors could have paid close attention to these warning signs and investigated the possibility that they could fail. Instead, these signs were merely ignored.

VII. CONSEQUENCES

Immediate Consequences

When the garage collapsed, the top floors sloped precariously. (Murphy, 2003, 1) The building shifted a total of 3 inches after the collapse. By the time it had settled enough for rescuers to enter, almost a week had passed. ("4 Dead," 2003, 1) 2 people were immediately found dead, while one was rushed to the hospital where he later passed away. A fourth victim was later found dead on the site. (Murphy, 2003, 1)

Secondary Consequences

The construction company was fined $119,500. These fines included 1 willful and 8 serious violations of safety standards. (Lipton "OSHA," 2004, 1) The Tropicana Casino remained open during and after the parking garage collapse. (Murphy, 2003, 2)

Tropicana Parking Garage Investigation
Violation Summary

<table>
<thead>
<tr>
<th>Company</th>
<th>Violation Description</th>
<th>Penalty</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fabi Construction</td>
<td>Willful Formwork not erected, supported, braced and maintained so that it would be capable of supporting without failure all vertical and lateral loads that may reasonably be anticipated to be applied to the formwork.</td>
<td>$70,000</td>
</tr>
<tr>
<td>Fabi Construction</td>
<td>Serious Reinforcing Steel was not properly installed to allow floors to be secured to columns and sheer wall.</td>
<td>$7,000</td>
</tr>
<tr>
<td>Fabi Construction</td>
<td>Serious Shoring plans were not available</td>
<td>$7,000</td>
</tr>
<tr>
<td>Fabi Construction</td>
<td>Serious No inspections of shoring and re-shoring prior to and</td>
<td></td>
</tr>
<tr>
<td>Company</td>
<td>Type</td>
<td>Description</td>
</tr>
<tr>
<td>--------------------------</td>
<td>----------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Fabi Construction</td>
<td>Serious</td>
<td>Shore heads not in firm contact with foundations and forms.</td>
</tr>
<tr>
<td>Fabi Construction</td>
<td>Serious</td>
<td>Proper test not performed to determine if concrete gained sufficient strength.</td>
</tr>
<tr>
<td>Keating Building</td>
<td>Serious</td>
<td>Formwork not erected, supported, braced and maintained so that it would be capable of supporting without failure all vertical and lateral loads that may reasonably be anticipated to be applied to the formwork.</td>
</tr>
<tr>
<td>Mitchell Bar Placement</td>
<td>Serious</td>
<td>Reinforcing steel and welded wire mesh not properly installed to allow floors to be secured to columns and sheer wall</td>
</tr>
<tr>
<td>Site Blauvelt Engineers</td>
<td>Serious</td>
<td>Inspectors did not ensure that reinforcing steel was installed properly</td>
</tr>
</tbody>
</table>

Figure #4: Close up of the crumbling levels (Courtesy of the D'Amato Law Firm).
VIII. CAUSES

The blame for the failure of the Tropicana Garage has been cast upon many aspects of the building project. The design underwent a series of changes that have been pinpointed as the cause of the failure (Lipton “Changes,” 2004, 1). For example, the columns were made smaller and support beams were moved in the design revisions (Lipton “Changes,” 2004, 2). However, Stephen V. DeSimone—a structural engineer—contended that the problem was in the execution of design, not in the design itself (Lipton “Changes,” 2004 1). In the end, OSHA found that the way the construction plan was executed was flawed, essentially causing the collapse (Lipton “OSHA,” 2004, 2).

The issue with the implementation of the design started with faulty installation of the concrete forms. Also, the way prefabricated steel reinforcement rods and an underlying beam in the garage floors were connected to six critical outer vertical columns made it destined to collapse (See Figure #4). Many of the steel connections were insufficient and therefore could not support the weight of the floor (Lipton “Changes,” 2004, 1). The rebar mats were not placed far enough into the columns and so they were not anchored securely. This problem occurred on all of the upper floors (Lipton “Changes,” 2004, 2). The horizontal reinforcing steel, or rebar mesh should have been pushed farther into the column. Also, more of these bars should have overlapped with the column line (Lipton “OSHA,” 2004, 2). Another problem dealt with the fact that there were insufficient amounts of shoring or support used to hold up the un-solidified concrete floors. Some of the workers noticed that the shoring was bowing and even cracking but were told to keep working (Lipton “Changes,” 2004, 2).

IX. WHAT DID THE INDUSTRY LEARN FROM THE EVENT

Earlier Corporation Failures

The Fabi Construction company had a history of incidence and fatal accidents related to their building endeavors. In June 1995, a worker was moving concrete slabs and fell 100 feet down an elevator shaft because the floor he was standing on collapsed – he died upon impact (Lipton “Changes,” 2004, 2). Again, in October 2002, several workers were injured when a prefabricated concrete panel collapsed (Murphy, 2003 2). On both these occasions, Fabi Construction was issued a series of fines. According to Jim Moran, director of the Philadelphia OSHA branch (Occupational Safety and Health Administration), the role of Fabi Construction in the Tropicana incident “…is even more egregious [because] it is a repeat offender. If you are going to keep fining people for killing other people, on its face, that is ridiculous” (Lipton “Changes,” 2004, 2). Although the guilt for this terrible accident should rest with Fabi Construction, the building
industry should learn as a whole to value both in mind and in practice human life. A potential industry improvement could be to better regulate construction companies who are repeat offenders in incidents involving fatalities. Unfortunately, there is no evidence of this change in practice.

Lessons NOT Learned

To this day, cost effectiveness is one of the top priorities during the design and construction of any building. In the case of the Tropicana Casino Parking Garage, the decision was made to switch from individual rebar rods to rebar mats (Lipton “Changes,” 2004, 2). This option—despite being more cost efficient—inevitably lead, in part, to the collapse. It is highly possible, therefore, that companies after this particular crash, as well as in the future, will make design decisions based more so on cost than on structural soundness and long-term stability.

X. CONCLUSION

The Tropicana Casino Parking Garage collapsed on October 30, 2003 at 10:40 EST as a result of poor construction techniques. The top five of the ten stories collapsed due to inadequate temporary supports and improperly anchored steel reinforcements in the concrete. The collapse caused 20 injuries and 4 deaths. Fabi Construction, Keating Building Construction, Mitchell Bar Placement, and Site Blauvelt Engineers were charged with a total of $119,500 in violations. One must balance cost effectiveness against structural stability in order to prevent tragedy. The fact that Fabi Construction had a negative record regarding fatal collapses in the past should have set off a red flag to the client. The design, construction, and eventual collapse of the parking garage collectively prove how important each aspect building a structure really is.

XI. ANNOTATED BIBLIOGRAPHY

CTL Group. “Tropicana Reinforced Concrete Parking Garage Collapse Evaluation.” <http://www.c-t-l.com/template_project.asp?topic=2603>. Although this source is not lengthy, it provides a solid structural overview of the collapse site. It includes the structure, focus, investigative approach, as well as the findings of the analysis. The source company, CTLGroup, was retained to perform an investigation and identify the collapse cause.

corporation that was cited for violations and the amount that they were fined.

**Foley, James M. “Parking Garage Under Construction Collapses.” Fire Engineering.**

Although this article is mainly about the efforts of the fire rescue team after the collapse, it also describes the way the floors fell when building collapsed and the exact location of the garage in Atlantic City. Furthermore it describes the exact way each of the floors of the parking garage were constructed and what happened to cause the structure to collapse.


Even as this article is not a technical source, it provides information in regards to the implications of a building failure. It also gives specific facts about the collapse, including that “too few metal bracing poles were in place beneath the 10th floor to support the weight of the fresh concrete.”


This article is a good resource for facts about the construction components prior to the collapse of the garage. It dictates the changes in the design, including a partial analysis of the amount of reinforcement between the concrete slabs and the columns in the garage.


OSHA concluded that intentional disregard or plain indifference by the main subcontractors of the resort led to the deaths. They were charged close to 100,000 dollars for “willful neglect”. They said that the mistakes made were part of “Engineering 101” and could have been easily avoided.


This article shows that signs of the collapse of the parking garage were present when the construction workers were laying the concrete slabs. It also says that there may have been OSHA violations and that workers may not have had the proper training for the job.


In this article, Harold Simmons is interviewed. He was in the building when it collapsed
and he says that “It sounded like an earthquake” The whole building was shaking…” The article also says that a fourth body was recovered from the rubble. Stacey Strasky was walking by the building before the collapse when she noticed strange noises coming from the building, but when she notified a security guard, the issue was dismissed.

(October 30, 2003). “4 Dead in N.J. parking garage collapse.” CNN.com. <http://www.cnn.com/2003/US/Northeast/10/30/garage.collapse/index.html>. This article was written shortly after the collapse and states that two people were found dead and two more were found in the rubble. One of the trapped men was able to be rescued, while they were waiting to acquire the other body because the area in which it was located was deemed unstable. When they talked with a woman at Aztar Cop., she did not give them an answer as to if construction would continue on the project.

XII. ADDITIONAL SOURCES


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Historicist: The Collapse of the Union Carbide Building

By Jamie Bradburn

Every Saturday at noon, Historicist looks back at the events, places, and characters—good and bad—that have shaped Toronto into the city we know today.
Last night’s wind storm may have caused power outages and scattered branches along city streets and yards, but it didn’t cause any major construction projects to tumble down. That wasn’t the case back in 1958, when blustery conditions during a late summer storm caused the steel frame of the Union Carbide building under construction on Eglinton Avenue to collapse. That the incident didn’t go down in the history books as a fatal disaster was due to timing and the skill of a bus driver.
The building slated to occupy 123 Eglinton Avenue East was designed by the firm of Shore and Moffat (which evolved into Shore Tilbe Perkins+Will), who would receive a Massey Medal that year for designing a research centre for Imperial Oil in Mississauga. The Globe and Mail indicated that the structure would consist of “contemporary modular design featuring glass and stainless steel with impressive black columns on the facade.” To maximize interior space in the 180,000 square foot building, no interior columns were to be built. Management of Union Carbide’s Canadian operations and its subsidiaries, including Bakelite, would take up two-thirds of the space, while the rest was slated to be rented out.

Installation of the steel frame began in mid-June 1958. By September 5, nearly all of the welding was finished except for the top two floors. Temporary bracing was put in that Friday to hold the unfinished sections in place for the weekend, with all signs pointing to the welding being completed at the start of the new work week. But Mother Nature had other ideas. A severe thunderstorm hit Toronto on September 6, which brought along winds that local weather stations reported were gusting up to 90 km/h. Around 6:20 p.m., due to the wind and possibly a lightning strike, the frame of the building swayed, then
collapsed in a scene that newspaper accounts compared to a falling house of sticks and a folding accordion. The roar of over 1,850 tons of falling steel was described in ways ranging from the sound of a jet squadron to a tornado.

*Globe and Mail* reporter Robert Gowe was at home a block east on Brownlow Avenue when his son Bob screamed “Dad! There’s a building falling!” Gowe quickly went to the front of the house to see what was happening:

At the top southwest corner it was already swaying downward. There was a noise like two freight trains colliding at full speed and the frame buckled and crashed to the ground with the shattering impact of a high explosive bomb. Sparks flew as steel crashed on steel in the sickening dive. People could be heard screaming from houses nearby and in a moment everybody seemed to be out on the street and hurrying to the scene…By the time neighbours reached places where they could see the spot…it was gone. It took minutes for many to realize that it could have really happened. And, after seeing it, I am not sure yet that I believe it.

*Photo caption: “One steel beam that fell onto Eglinton Avenue shattered this car owned by Charles Boomer of Cottingham Street, who was in a restaurant with his wife and*
daughter. Several other cars were flattened like pancakes as the girders crashed on top of them.” The Telegram, September 8, 1958.

Owners of five vehicles crushed by the falling steel were quick to believe what had happened. Frank Fielding and his wife were dining on Eglinton when the lights inside the restaurant flickered. “I told my wife to wait while I went for the car,” he told the Globe and Mail. “When I got there I couldn’t even see it. The steel had buried it completely.” One home on Redpath Avenue was damaged by both a beam that fell by it and a maple tree that was forced into the kitchen.

If the evening had a hero, it was bus driver Joseph Kelly, whose prompt action saved the lives of forty-five passengers in his vehicle. Kelly was at the wheel of a westbound bus on Eglinton when he noticed two men running along the north side the road in a state of panic. “When I looked up and saw that steel swaying,” he told the Telegram, “my heart stopped.” Certain that the structure was going to slam down onto Eglinton, Kelly put his foot down on the gas pedal and swung north onto Redpath. “Just as I stopped, about 150 feet from the corner, there was a tremendous vibration. I looked back and saw that the building had fallen not on Eglinton, but on Redpath behind us.” Kelly’s quick turn north and the sight of the tilting structure caused his passengers to panic and throw themselves to the floor of the bus. Among the grateful riders (“he saved our lives”) was Mrs. Douglas Bolt, who gave her account to the Star:

The bus slowed up and I heard this terrible rumbling and looked up to see what looked like smoke coming from the first floor. Then the bus suddenly lurched and went around the corner as some debris was hitting the side of the bus…I was watching the top part of the falling building to see if it was going to land on us. I thought we were all going to be killed and might have been if it hadn’t been for the quick thinking of the bus driver. Some of the passengers were screaming and threw themselves on the floor, I saw a woman lying on her 10-year-old son. Other passengers were holding on to each other and screaming “let me off, let me off.” Some ran up and down the aisles on verge of panic.

Despite admitting that he had never been so scared in his life (he still shook three hours later), Kelly kept his cool and walked through the bus to check on the passengers.

No One Killed or Injured

11-Story Toronto Steel Skeleton Collapses During Thunderstorm

Headline, the Globe and Mail, September 8, 1958.

When police detective David Williamson heard the sound as he cruised along Mount Pleasant Road, he quickly put in a call to “rush all utilities.” The site was quickly cordoned off as police, hydro workers, firefighters (who quickly put out a small blaze caused by a fallen gas can) and even a few priests rushed over. Final rites were not required that evening, as there were no fatalities—the only worker that had been on the
site was a teenaged night watchman who luckily had been on a separate part of the grounds when the collapse occurred. Toronto chief building inspector John Payne felt it was a miracle that the incident happened on the weekend, as a normal day might have seen high fatalities among workmen and those stuck in traffic by the site. Investigations into the collapse were carried out by the city, insurance companies, and consultants hired by Union Carbide. All agreed that the temporary bracing was insufficient to withstand the high winds. A report presented to Union Carbide determined that the architectural design was still sound, but to ensure another collapse didn’t happen it was recommended that deep horizontal trusses between the columns of each floor should be used for support.
Plans to rebuild went ahead. The first batch of office workers settled into their desks in July 1960, greeted by the stainless steel decor that dominated the building’s main floor. Union Carbide remained the main tenant of the International style complex until the early 1990s. Despite efforts to recognize the architectural significance of the building, it was razed in 1999 to make way for the condo that currently occupies the site.
March 2012

License Engineers and Certify Disciplines

In response to Timothy A. Lynch’s opinion in March’s Structural Forum, I agree with everything that Mr. Lynch said. I am licensed in several states and my area of expertise is civil engineering with a specialty in water systems and water-retaining structures. It is frustrating that, in the States of Illinois and Hawaii, I cannot perform the design of simple reinforced concrete structures because I am not a licensed SE. However, I know architects that can legally perform the services even though they don’t have a clue about water-retaining structures.

I developed my expertise through a career of specialty work and I would naturally limit my services to that expertise. I have no desire to attempt services outside of that expertise and I believe that the vast majority of other professional engineers work within their discipline. I support certain limitations, such as requiring an SE for critical structures such as hospitals, schools, or buildings over 13 stories high, but preventing me from performing a service that I handle on a daily basis is ridiculous.

Thank you,

David J. Peterson, P.E., SWD
President/CEO
Watershape Consulting, Inc.

February 2012

Responding to Forces of Nature

STRUCTURE magazine’s Editorial in the February 2012 issue raised important questions for consideration by the structural engineering profession. It makes a plea for structural engineers to become involved in reducing losses from natural disasters, by investigating building failures, strengthening building codes and working with jurisdictions, etc. While I agree with this premise, the call to action is overly broad and does not acknowledge what we already do in these areas. Indeed, structural engineers are already heavily involved in these activities. However, despite our best efforts, annual US economic losses from natural hazards continue to increase. The editorial also does not provide evidence why greater involvement would be more effective in reducing natural hazard losses.

I disagree with the author's premise that structural engineers have a “natural tendency after any major event (is) to call to strengthen the codes.” Our role as engineers has been
to create and advance scientific knowledge on building design through careful study and effort in order to predict building failures under extreme loads.

For example, many engineers are involved in long-term tasks to learn from last year’s tornado losses. In 2011, non-engineered residential structures sustained enormous damage during tornado outbreaks that caused over $22 billion in damage, and the second highest number of annual tornado fatalities on record. Code changes would make little difference in the performance of those existing structures, unless the structural resilience of existing homes was also improved. STRUCTURE magazine (July 2011) reported on the Tuscaloosa tornado damage survey and ongoing analysis of the data. However, it is likely that costly tornado disasters will occur again, and we should admit that more engineering (i.e. better codes or enforcement alone) is not the answer.

According to a 2007 study by the US Nuclear Regulatory Commission, there is a very small chance of a tornado strike within the contiguous United States (probability of 1 in 100,000 per year of a tornado with estimated wind speeds of 160 mph or greater). However, the consequences of that event upon a community can be catastrophic, and should be factored into our design; currently this is not the case.

Natural disasters result from a complex interaction of the physical hazardous event, with the vulnerability of society, its infrastructure, economy and environment. That vulnerability is determined largely by human behavior and actions, and so it is within our control to reduce it. Ultimately, people decide where and in what type of structure they will live. However, any action to reduce disaster risks and damage must hinge on a community’s collective decision, based on its risk perceptions, which go beyond engineering.

The public may be unaware (or in denial) as to the structural inadequacies of the majority of their existing homes. Currently, very few homes are actually engineered. Yes, structural engineers must be part of the conversation (and education) with the public, but it is primarily the public who must decide the level of risk that is adequate for them, and establish acceptable quality of structures they are willing to pay for. The message to our membership should be to understand the latest thinking on vulnerability (social and economic), and be able to discuss risk reduction and structural design issues. In this way we could best advise the public regarding its vulnerability to natural hazard risks, and what should (could) be done to reduce it.

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November 2011

Alfred Pancoast Boller
The "Great Achievements" article by Frank Griggs on Alfred Pancoat Boller in the November issue of STRUCTURE magazine states, “In 1909, Boller and Hodge were appointed engineers for the Port Authority of New York and New Jersey.” This does not appear to be correct. The Port Authority (originally: The Port of New York Authority) was created on April 30, 1921, nine years after Mr. Boller’s death. The Hudson River crossings studied during the period from about 1900 to 1927 were presumably conducted by the New Jersey Interstate Bridge and Tunnel Commission and the New York State Bridge and Tunnel Commission, which were not part of nor rolled over into the Port Authority. Boller, Hodge & Baird was a consultant to the New York State Bridge and Tunnel Commission in 1913 that recommended a bridge at 57th Street in Manhattan to New Jersey. This plan was rejected primarily because of its substantially higher cost as compared to a tunnel (see The New York Times, 04/22/13). The Holland Tunnel project was advanced instead; it opened in 1927, and the Port Authority took over its operation in 1930.

Joseph Kelly

June 2011

STEM

I enjoyed your article, “STEM”, in STRUCTURE, June 2011. It is a subject that I have given some thought to, and I believe it to be timely. In my mind there are several facets to the problem, some appear addressable and others do not. As modern technology developed, it has become more subtle and difficult to comprehend. For example: a nuclear power plant is far too complex for most of us to comprehend, whereas the functioning of a waterwheel is quite comprehensible. Electricity makes sense with some study but solar cells require another level of knowledge, as do solid state devices and modern astrophysics. Most of us acknowledge their existence and enabling attributes with little comprehension or full appreciation.

In fact, most modern technology is beyond the ken of nearly all primary and secondary teachers. Thus, it is nearly impossible to teach or appreciate such technology in primary or secondary schools. This void in education leads directly to a lack of appreciation and interest by youth as well as adults.

When the technologies of chemistry and electricity were in their infancies in England, the "Royal Society" and other organizations gave presentations by such notables as Robert Boyle in chemistry and Michael Faraday in electricity. These lectures and demonstrations were extremely popular with the general public. Of course, they were forming the scientific-technological foundation for the Industrial Revolution that followed in England.

If present-day professors in the various technological areas could be enticed to give, pro bono, exciting lectures with demonstrations in their field of endeavor, the public might well be interested. Of course, technological luminaries from industry might present
similar lectures. These professionals are usually adept at presenting complicated matters to non-technical people when writing proposals and making presentations to boards of directors. The professional societies are the obvious initiators for such efforts, much as the Royal Society did over 200 years ago.

Youth should see that science and math are exciting and not all bad, and that one can find a challenging and financially rewarding career in science or technology.

Frankly, many educators teach, or at least imply, that to aspire to make money is simply greed and that technology pollutes the earth.

A deeper understanding of God's creation and having the ability to use it for the betterment of our civilization is not all bad, but is indeed something to appreciate and to which one may aspire.

_Dann H. Hall_
Coopersburg, PA

**June 2011**

Dear STRUCTURE Editorial Board,

Each month I receive numerous trade magazines and journals representing a variety of organizations in the Civil Engineering field. In many of these cases, the lifespan for these publications is barely long enough to breeze across my desk before they end up in the trash can. Occasionally, I will browse the headlines and articles to check for interesting stories but I am rarely successful in finding something to catch my eye.

However, my subscription to STRUCTURE is unquestionably the exception. Each month I look forward to receiving it and carefully comb through every article. I even maintain a library of past editions which I frequently reference back to from time to time. I thoroughly enjoy every aspect of reading your magazine and always appreciate the Structural Practices, Opinion, and Feature articles included in every copy. Your publication has allowed me to grow in my career and I am without a doubt a better structural engineer as a result of your efforts.

Thank you for your hard work and dedication to this wonderful publication. I just received my latest edition and have already carved out a few minutes today to jump into the articles!

Yours Respectfully,

_Michael Howell, P.E._
Structural Engineer
Austin Brockenbrough & Associates, LLP
March 2011

Bridge Sustainability

In the March 2011 issue of STRUCTURE, author Mark D. Webster points out that the production and fabrication of one ton of steel results in approximately one ton of carbon emissions released into the atmosphere. In other words, the carbon emissions are about equal to the amount of steel. This indicates a real need to reduce the amount of steel manufactured. In my work as a structural engineer for the City of Chicago, Bridge Division, I found two ways to increase the useful life of steel bridges, which reduces the need for new steel and thus reduces carbon emissions:

1) On bascule bridges we try to make the deck as light as possible to reduce the counter weight, foundation loads and operating power. The obvious solution was to use an open grid deck, which is much lighter than a concrete deck and could still easily carry heavy truck loads. Several years passed with no signs of trouble. Then, during a routine inspection, it was found that the flanges of the small beams supporting the open grid deck were badly corroded. Debris from the tires of thousands of vehicles crossing the bridge had landed on the flanges and retained water from rain and melting snow, causing the flanges to deteriorate. Is there any way to prevent this situation? The answer is to use an orthotropic steel deck. Closed ribs can be designed to be airtight, precluding corrosion of the inside of the ribs. An excellent book on the subject is Design Manual for Orthotropic Steel Plate Deck Bridges, published by the American Institute of Steel Construction. It contains design information and discusses practical considerations, such as corrosion prevention.

2) The use of salt to melt snow has caused deterioration of various steel parts of both fixed and movable bridges. The solution to this problem is to find deicing agents that have no corrosive components and use them on bridges and bridge approaches. Away from the bridges, the standard deicing chemicals can be used.

Peter Kocsis, SE, PE

February 2011

Fazlur Rahman Kahn

Your article on Dr. Fazlur Rahman, the great structural engineer, was very inspiring. I have always felt proud and honored of his Pakistani heritage. Your article was beautifully written, except for the small part about the political history on the creation of Bangladesh (East Pakistan). According to a book written by Pakistani military commander (General A. A. K. Niazi), the separation movement in East Pakistan was fully funded and supported by India. This has also been confirmed by Indian author Mr. Ashok Raina, in his book Inside RAW.
The separation of East Pakistan was a tragic period in the history of Pakistan. At its creation in 1947, the union of East and West Pakistan was considered unnatural by many international experts. These two East and West units of Pakistan were separated and located thousands of miles away from each other.

Historically, the people of that region (South East Asia) have lived together for centuries. In 1947, the British Empire divided this region into India and Pakistan. Later, Pakistan was split and Bangladesh was created. However, the people from that region are still connected above and beyond these boundaries. As a Pakistani American, I feel optimistic that one day the people of that region will put aside all their differences and will live in complete peace and harmony, just like people of the Americas and Europe.

Ahsan M. Sheikh, P.E., S.E.

February 2011

Fazlur Rahman Kahn

As past-President/Director of SEAOSC (and past SEAOC Director) it was my great pleasure to host you when you spoke at one of our dinner meetings in Los Angeles a couple of years ago. I have always enjoyed your articles and I liked your book Engineering Legends (I have a personally autographed copy thanks to my wife who also attended that dinner event). I almost always agree with your stated viewpoints about our profession in general and on civil/structural engineering education in particular. As for the great genius Dr. Fazlur Rahman Khan, I have always felt proud and honored that he was a South-Asian American like me and in fact got his start in the profession in Pakistan like I did after I graduated from the California Institute of Technology, way back then. Your article about Dr. Khan (STRUCTURE Magazine – Feb 2011 issue) was beautifully written, except for the small part about the political history of the region where his roots lie. I would like to offer some corrections and background facts about that important subject if I may.

It is true that, in 1947 (not 1971 as was written in the article), the Imperial British Colony of India attained its independence and was partitioned into two sovereign countries, India and Pakistan, generally along the lines of the wishes of the people who resided in those areas of the South Asian Subcontinent. The people of East Bengal chose (and struggled) to become part of Pakistan even though there were 1,500 miles of India in between the two wings of the newly formed country. Post 1947, the relationship between Pakistan and India soured, tragically, and was exploited by some vicious and deceptive political/ethnic groups in the region, including East and West Pakistanis and Indians, as well as by the protagonists of the Cold War which was at its zenith in the 1960s and 1970s. Pakistani government/political policy in East Pakistan often exacerbated the situation. 1971 was the year that another war broke out between India (supported by the USSR) and Pakistan (supported by the USA) leading to an invasion of East Pakistan by the Indian army, resulting in the breakup of the country into Bangladesh (“East Pakistan”) and Pakistan (“West Pakistan”).
It was a dark and painful chapter in the history of the Subcontinent, for which all sides must share blame. As a teenager living in Karachi, Pakistan during the 1971 war, I experienced firsthand the terror of bombs being dropped on my civilian residential neighborhood night after night. Even then I realized that government forces on our side had also committed atrocities in East Pakistan. It is my belief that for most of us the scars from that awful experience have healed. In general Pakistanis, Indians and Bangladeshis have moved on to discover that they have more in common than what divides them, and more to gain from respectful cooperation rather than hubristic confrontation. As is often the case, however, there continue to be some selfish and narrow-minded (and pretty powerful) detractors on all sides who would have us believe otherwise. It is not common for us engineers to delve into politics and history, especially in articles related to our great profession, but I thought it important to clarify this point in your otherwise stellar article about the great Fazlur Khan.

*Saif M. Hussain, MS, P.E., S.E., SECB, LEED® AP*

**Author Response**

Messrs. Ahsan Sheikh and Saif Hussain offer clarification and insight into the history of, and past conflicts between, Pakistan and India – as well as between East and West Pakistan. Mr. Sheikh additionally lists two books providing further information for anyone interested and Mr. Hussain stresses that in 1947 India and Pakistan were, indeed, partitioned into two sovereign countries, India and Pakistan, as was stated in my article.

It was during the 1971 conflict between West Pakistan and East Pakistan (Bangladesh) – with interference from and/or instigation by India – that Dr. Fazlur Khan founded the Bangladesh Emergency Welfare Appeal, a Chicago-based organization, to help the people in his homeland. It was a noble, gallant and caring effort.

It has long been my contention that engineering does not occur in a vacuum, nor do engineers perform their work in a vacuum. Nor are engineers merely technical experts. They are also real “honest-to-goodness” people with human traits and feelings. To tell the story of an engineer and make him (or her) come across as a person, you need to tell the whole story. Dr. Khan was an especially well-rounded individual with many facets to his personality, and accomplishments in his career. To tell his story without mentioning his humanitarian and community leadership activities – and how he fit into history and the life and times he lived in – would be short changing his greatness and what he was all about. He was much more than a mere technologist. He was an iconic engineer with a very broad perspective on life.

*Richard Weingardt*

**July 2010**

**Small Firm Experience**
I totally concur with your comments about BIM and LEED in your July 2010 Editorial. I currently provide structural engineering consultation, analysis, and design in the light construction sector. I have Architect associates that I partner with that still deliver their services utilizing hand drafting. When I first started my own practice, I purchased CAD software. However, I soon found that it was better for me to focus on engineering business and to instead sub-out drafting to a CAD consultant who delivered the drafting service cheaper, faster, and better then I could.

I do not want to sound like a dinosaur, but BIM and LEED do not represent an important development in the business that I am in. Only one of my architectural associates is pursuing BIM – at a high cost and with little profitable return. More are pursuing LEED, but none have reported to me about it being profitable for them, just time consuming and costly. These two emerging areas of the A/E industry are certainly important, but not to everyone.

Peter Cloudas, PE
Isidro-Cloudas LLC
Stamford, CT 06907

Author Response

I would like to thank Mr. Cloudas for his support of my comments on BIM and LEED. I believe that we need to follow the money to see who is really benefitting from these unsolicited additions to our project costs.

I also believe that, with respect to BIM, clients want a free system to run their facility. Although shrouded in the fog of these services being “beneficial to mankind”, there is a cost to everything of value. I trust that clients can be willing to pay those extra costs to the firms providing that additional end product.

Structural engineering firms have been delivering their services quite efficiently for a very long time (since the advent of the pencil!) without BIM or a LEED program. I am sure they would be happy to continue in the same manner until such time that clients make it worth their while to incorporate such a drastic change.

John Mercer, PE

July 2010

Learn, Adapt and Change…

John Mercer’s editorial was thought provoking. I agree that firms need to assess their survival potential, but to describe this problem as something we need to weather until things get back to normal is misguided. Engineers must learn to adapt to a continually
changing environment. What we need is not how to increase billings but rather how to learn, adapt and change.

Classifying firms as Finders, Minders, and Grinders is disturbing. How would you feel if your role was just to work harder and grind it out? This attitude contributes to the turnover in firms and the dearth of experienced engineers looking after the technical work.

The fact that the Grinders “typically include entry level engineering staff” should be of concern both for its impact on the quality of the work and on productivity. Too often senior staff is busy acting as Finders and Minders, and putting out fires, while entry level engineers are left to work through their problems. Much time is wasted. Less than desirable solutions are the result, often because there is not time to do it over again.

Technical oversight is often a problem because senior staff is overworked. In addition, because of their focus on finding and minding roles, senior staff has difficulty keeping up to date with new codes and new software. Thus, senior staff may be unable to fully manage junior staff that is doing the work.

If a firm wants to take advantage of opportunities to make improvements, they need to look beyond costs and revenues. Financial data is an important enabler but cannot substitute for vision and leadership.

We need to look at why firm are selected and how they can provide value to their clients. All too often engineers are seen as necessary evils and are given repeat business because they have not screwed up too bad on the last project. If we think that the answer is to work harder or more efficiently, we are in denial.

More importantly, firms need to understand how to implement and maintain constructive change. It is easy for management to issue a new policy memo, but all too often the change doesn’t persist. Firms need to learn how to discuss issues and build consensus with staff. Many firms have office standards which are routinely ignored. We need to adopt a culture of continual improvement where we learn from experience.

IT, BIM, and LEED will continue to be problems until we learn to manage them proactively. We need to learn how to talk about the issues, develop consensus, and implement change.

Firms that adopt a culture of continual improvement, and that can implement and sustain change, will have an advantage and will have the tools to survive and grow. Firms that continue as before will find it difficult to survive in the long run.

*Mark Gilligan S.E.*

**Author Response**
Thank you, Mr. Gilligan, for your response to my Editorial. My comments describing differences between principals, project managers, and new hires seem to have struck a chord. In my personal experience, however, these classifications ring true in many firms.

I could not agree more with his comment on the issue of technical oversight through overworked senior staff. Firms often take on more work than they can handle, leaving gaps in quality, or pushing the responsibility for project design downward onto the least experienced staff. I think the result is seen in higher cost projects or other undesirable outcome. The most undesirable, of course, being called to task in a lawsuit.

The importance of continuing education becomes apparent when principals cannot keep up with the technical changes in design codes. Once the limits of senior staff members are reached, the firm needs to make an intentional decision to meter back the workload. Using Mr. Gilligan’s words, senior staff needs to learn, adapt, and change. One way is by becoming active participants in one or more of our three organizations; CASE, SEI, and NCSEA.

My comments on BIM and LEED are targeted at the unsolicited added burden of time, money and education necessary beyond what firms had been adequately providing in their service mix. If the costs of these additional services are passed on to the client, management of these would become very simple. At the moment, the clients’ appetites are larger than their budgets.

Fluctuating economies require firms to continually learn, adapt, and change. The only change that I’ve personally seen is firms cutting staff and expenses wherever they can. The impact of that type of “change” is not only on the firm, but on the engineers and their families.

In keeping with his comment for firms to be looking at continual improvement, I assume Mr. Gilligan has ideas that can be implemented industry wide. I have openings on CASE committees that would allow him to join with other like minded structural engineers to improve our business practices and working environment for all. Do you have good ideas as well? If so, join Mr. Gilligan and others in debating the issues and crafting solutions.

John Mercer, PE

September 2010

Structural Contributions to LEED

A misrepresentation of structural steel’s recycled content appears in the article Structural Contributions to LEED, published in the September issue of STRUCTURE magazine. The article states:
“Credit MR 4.1 and MR 4.2 – Most structural steel shapes are made from 97% recycled material. Recycled content in steel plate is about 65%. HSS sections are typically not made with recycled steel and should be avoided on LEED projects.”

In actuality, the current industry average for hot-rolled structural shapes is 93.3%; all U.S.-made hot-rolled structural shapes are produced in electric-arc furnaces (EAFs) using steel scrap as the primary feedstock.

Plate is produced in either EAFs or basic oxygen furnaces (BOFs). The average recycled content for plate made via the EAF process is 93.3% and 32.7% for plate from the BOF process. No plate has an average recycled content of 65%.

HSS is a manufactured product made from coil steel. If the coil steel originated in an EAF process, the average recycled content of the HSS is, again, 93.3%; if the coil originated in a BOF process, the average recycled content of the HSS is 32.7%. As such, the statement that HSS is not typically made with recycled steel is incorrect. All HSS has significant recycled content.

In both the case of plate and HSS, the steel supplier can track the product back to the producer and determine the production process. If the material cannot be traced but is known to be of domestic origin, the BOF value can be used as a default. For non-domestic material, USGBC allows a default recycled content value of 25% to be used.

The statement that HSS is not made with recycled steel and that HSS should be avoided on LEED projects is damaging to HSS producers, and may result in decisions made by structural engineers that could increase, rather than reduce, the environmental impact of a building project.

Geoff Weisenberger
Director of Industry Sustainability
American Institute of Steel Construction

August 2010

Is Roof Eave Blocking Required To Transmit Wind/Seismic Forces?

Registered and/or Licensed Engineers, in all states of the US, are obligated to provide for the health, safety and welfare of the general public. With this responsibility, in our opinion, blocking is required between all roof rafters, all roof trusses, all floor joists and all floor trusses to transfer roof and floor horizontal force diaphragm loads to the designated shear wall resisting elements indicated on the appropriate contract structural drawing.

To advocate that the International Building Code (IBC) is not precise and allows the omission of blocking, together with the violation or omission of the continuous load path from the roof diaphragm and/or floor diaphragms to the foundation, is a violation of the
Building Code. Blocking and the continuous load paths are to be installed with a rational analysis in accordance with established Principles of Engineering Mechanics. Based upon our review of the current Building Code in effect in California, and presumably across the country, no such deletion is allowed.

In fact, the Building Code unequivocally references blocking between roof structural members, floor structural members and the principle of a continuous load path in numerous Building Code Regulations. Failure to install roof and/or floor blocking and to provide for a continuous load path will place the general public in life hazard safety situations for all designated Building Code lateral force conditions.

Metal hardware installation should not be an alternative for lateral force transfer loads as suggested in the subject article.

Arnold Bookbinder, Structural Engineer
Arnold Bookbinder & Associates

Author Response

My article attempted to arrest the growing practice of omitting eave blocking by reminding design engineers that although eave blocking is not prescriptively required by the ICBO, basic engineering mechanics and metal connector manufacturers’ specifications require its installation.

It was my intent to present a balanced analysis of all existing prescriptive and analytical requirements, but at no time did I advocate omitting the use of eave blocking.

Felix Martin, S.E.

July 2010

Engineers Are From Aristotle

Jon Schmidt draws interesting connections between Aristotle's metaphysics and the practice of structural engineering. In a specific example about change, Schmidt mentions that a steel billet has the potential to become a wide-flange beam in the future. True. But how many structural engineers really think of our profession in terms of material, matter, and the process of becoming? More so than in the past, I think, we often lose sight of the physical reality of our designs. I am sometimes asked basic questions by non-engineers about home construction, materials science, or how historic structures were built – and in response, I mumble something about moment diagrams. I’m embarrassed that I call myself a structural engineer in training (I'm unlicensed so far) without knowing the answers to these basic questions. I admit that it’s my own shortcoming, but it seems to be a common problem among younger structural engineers.
Many philosophers disdain or even deny the existence of the physical world, but Aristotle is one of few who value it ("In all things of nature there is something of the marvelous"). As engineers, we should be encouraged by Aristotle, and remember to educate ourselves not only in analysis and design but also in field practices, materials science, and other practical considerations.

A further note on Aristotle's teleological views: Schmidt says that Aristotle viewed "teleology as something that is present throughout the universe, not just confined to human endeavors." But I don't think this is true: non-living things do not have ends toward which they aim. Philosopher Martha Nussbaum writes that "...Aristotle neither applies teleology to non-living natural bodies nor gives any evidence of believing in a universal teleology of nature." (Aristotle's *de Motu Animalium*, 60)

It can be difficult for engineers and philosophers to find common ground, so thanks to Jon Schmidt for illuminating this connection. Also thanks for explaining the wonderful concept of *eudaimonia* or "human flourishing," a valuable idea of happiness and a vast improvement over the passive contentment or the momentary joy we wrongly call "happiness."

*Leigh Arber*

**Author Response**

I appreciate the feedback, although I stand by my statement about Aristotle’s view of teleology. Non-living natural things *do* have ends toward which they aim, albeit not consciously (obviously) or in quite the same way as living things or artifacts (e.g., as functions). In fact, that is the only way that our ordinary concepts of (efficient) cause and effect ultimately make any sense; there is something in the essence (formal cause) of a thing that directs it toward producing certain effects (final cause), and not others or none at all. In other words, every natural substance acts for the sake of ends that are determined by its own nature.

*Jon Schmidt*

**November 2009**

**“Podium” Slabs**

Thank you to Mr. Russillo for the informative article on “Podium” slabs (Nov. 2009). These structures were popular in California in the 1960s, even before the advent of post-tensioning. I am glad they have finally found their way to the Northeast.
However, let me register a small critique of the very title of this work. I believe “podium” is a misuse of the term. According to Webster, a podium is either a low wall of some sort, a continuous bench in a room, or a low platform for the conductor of an orchestra. I do not see anything like our structures in these definitions.

I suggest that we call them what they are: “platform slabs”.

Richard Martter, P.E.
Senior Structural Engineer / Founder / President Emeritus
Strand Systems Engineering, Inc.

Author Response

It is agreed that care be taken to ensure the proper use of words in order to provide the reader with the correct picture of what we are describing. The term podium was used (although the thesaurus also listed synonyms such as pedestal and platform) since it matched the term used in the Post-tensioning Manual, sixth edition by PTI which was referenced if the reader desired further information. In any case, this allows us the opportunity to reflect on the line from Shakespeare's Romeo and Juliet:

"What's in a name? That which we call a rose
By any other name would smell as sweet;"

Michael A. Russillo, P.E.
Senior Manager Special Products
Barker Steel LLC

March 2010

Welding Inspection and the New Chapter N of AISC 360-10

This letter is in response to the article “Quality Time” in the March 2010 issue of Modern Steel Construction (MSC). The Structural Engineers Association of California’s Construction Quality Assurance Committee would like engineers to be able to consider a different point of view than that presented in the MSC article, and to be aware that their own responsibility to specify the frequency of welding special inspection may be increasing.

The 2010 AISC Specification for Structural Steel Buildings (AISC360-10) has recently been completed, including a new Chapter N, Quality Control and Quality Assurance. Chapter N provides a complete and comprehensive system of quality control, quality
assurance and non-destructive testing for structural steel buildings and for steel elements of composite members.

A proposal to replace the structural steel special inspection requirements of the International Building Code (IBC) Chapter 17, with the quality assurance provisions of AISC 360 Chapter N, has been accepted by the IBC Structural Committee, and will be considered again at the Final Action Hearings in May of 2010.

The Structural Engineers Association of California (SEAOC), Construction Quality Assurance Committee (CQA), is concerned that the quality assurance provisions of Chapter N, as applied to welding special inspection of multipass fillet welds and groove welds, represent a substantial decrease in special inspection over what is in the IBC now.

We feel that it is important for structural engineers to be aware of these issues, to consider how welding special inspection is typically handled in their region, and to be prepared to augment the inspection requirements for their projects, if necessary.

Current IBC Special Inspection

Chapter 17 of IBC 2009 has inspection and testing provisions for all construction, and additional inspection and testing for high-seismic construction. IBC 2009 refers welding inspection and testing for high-seismic to AISC 341, which has provisions very similar to AISC 360-10, Chapter N. IBC 2009 references AWS D1.1 for welding inspection of all construction.

The level of special inspection effort required by the building codes (IBC, BOCA’s NBC, UBC) has long been described by the use of the terms “continuous” and “periodic”. The code’s distinction between continuous and periodic inspection, and the decision as to which work items the terms are applied, has always been based on whether the important aspects of the work are fully available for observation at completion (periodic is appropriate) or whether there are stages during the progress of the work or aspects of the work process that need to be inspected before continuing or during the process (continuous is required.)

Welding inspection is “periodic” except for CJP and PJP groove welds, multi pass fillet welds, fillet welds greater than 5/16”, and plug and slot welds, all of which require “continuous” inspection.

The distinction between continuous inspection and periodic inspection for welding can best be illustrated by the following excerpt from IBC 2009, Section 1704.3, Exception 2 (this is essentially a description of “periodic” welding inspection):

“The special inspector need not be continuously present during welding of the following items, provided the materials, welding procedures and qualifications of welders are verified prior to the start of the work, periodic inspections are made of
the work in progress, and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.”

Continuous inspection is described as “full-time observation” in the IBC. There is, however, quite a bit of variation in how this requirement is interpreted and enforced by engineers, testing and inspection agencies, and building officials across the country.

**AISC 360-10 Chapter N Special Inspection**

Chapter N takes a different approach. It presents an excellent breakdown of welding inspection tasks, and identifies each task as an Observe task or a Perform task for both the fabricator/erector’s Quality Control (QC) inspector, and for the project owner’s Quality Assurance (QA) inspector (the special inspector). Observe and Perform are defined as follows:

- **O** – Observe these items on a random basis. Operations need not be delayed pending these inspections.
- **P** – Perform these tasks for each welded joint.

The Observe designation is applied to almost all of the “before” welding tasks, and to all of the “during” welding tasks. The Perform designation is applied to all of the “after” welding tasks. Observe is comparable to “periodic” in the IBC and provides the same broad latitude regarding the frequency of inspection activities.

The terms “continuous” and “periodic” are not used, and there is no distinction made between types of welds, or whether a weld is single-pass or multi-pass.

The fabricator/erector’s quality control organization and function is detailed and the requirements are made explicit. Although AWS D1.1 has always required that visual inspection be made by QC of all welds, these provisions in Chapter N should help ensure that the QC inspections are made.

AISC has noted that the concept behind the use of the Observe level of inspection is that of random sampling, and has suggested that this use of the Observe level of inspection gives the inspector the flexibility to provide the inspections needed. It is fair to say that this corresponds to current practice for “periodic” inspection, and to some extent for “continuous” inspection, given the wide range of interpretation and enforcement of the “full-time” observations required by the IBC.

**SEAOC CQA Concerns**

The Observe level of inspection, when applied to virtually all of the “before” and “during” welding tasks, represents a significant decrease in special inspection for multipass fillet welds and all groove welds when compared to the continuous inspection requirements of 2009 IBC. Although AISC 360-10 was developed by a consensus process, we do not feel that sufficient reliability studies were performed to justify such a
decrease in quality assurance welding inspection. Improved quality control may help, and this may contribute to overall quality. Unfortunately the project owner has no control over the fabricator/erector’s quality control organization, and unless the special inspector is verifying the work, cannot be assured of the desired level of quality.

IBC references AWS D1.1 for all welding inspection. AWS D1.1, Section 6.5.2, Scope of Examinations, states:

The Inspector shall, at suitable intervals, observe joint preparation, assembly practice, the welding techniques, and performance of each welder, welding operator, and tack welder to ensure that the applicable requirements of this code are met.

We have maintained that the building code has defined these “suitable intervals” by the use of the terms “continuous” and “periodic” and has assigned the more stringent interval (continuous) to those welds where it is suitable (multi-pass fillet welds and all groove welds).

Under Chapter N, once the inspector has verified the materials, WPSs, welder qualifications and skills, etc, at the beginning of a project, complete penetration groove weld joints could be started (fit-up and root pass) and completed (filler passes) without any of the steps being observed by the welding inspector (either QA or QC), for any particular weld. This would not be an abuse by the welding inspector – the inspector would be simply following the intent of Chapter N. Our contention is that this represents a substantial decrease in scrutiny over the continuous inspection currently required for this type of weld, regardless of how loosely one interprets the term “continuous.”

Our concerns are not driven by objection to change. SEAOC CQA, which is comprised of both design engineers and engineers in charge of testing and inspection laboratories, has considered input from testing and inspection laboratories who report significant differences in the workmanship of fabricators and erectors, both AISC-certified and non-certified, when inspectors are actively inspecting the work.

**Engineers Take Notice**

AISC has also indicated that the intent of not defining a time component for the Observe level of inspection is so that the engineer can feel free to define the frequency of inspection. It is true that the Statement of Special Inspection required by IBC Chapter 17 is intended to be prepared by the engineer (or at least the structural items should be), and that the extent and frequency of each test or inspection is to be detailed in the statement. Thus the engineer is already empowered to determine these frequencies. However, under the 2009 IBC, an engineer who is not steeped in quality assurance issues (as we in CQA are) is able (and likely) to default to the “continuous” or “periodic” requirement already in the code. Once those terms are gone, if the engineer does not further detail the frequency of Observe level inspections in the project specifications and the Statement of Special Inspections, the default frequency will apparently be up to the welding inspector.
Our concern of course is that the building code is intended to be a minimum standard, and placing the quality assurance provisions of Chapter N into the building code is a reduction in that minimum standard.

SEAOC’s Construction Quality Assurance Committee and the local California member organization’s committees have worked to elevate the engineer’s understanding of the quality assurance requirements in the code, and to emphasize the importance of the engineer’s participation in construction quality issues. Engineers take notice – your responsibilities for detailing welding inspection activities may be increasing.

Art Dell, Chair
SEAOC Construction Quality Assurance Committee

February 2010

In reading Barry Arnold’s article “The Failure of the Five E’s” in the February 2010 issue of Structure, I am obliged to provide additional information concerning the “Examination” element of his article.

The current licensure process for many U.S. states and territories does not mandate that individuals must take an NCEES examination in the same discipline in which they were awarded their engineering degree. Many people end up in fields of engineering other than their degree based upon interest, job availability, economics, etc. In such cases, once licensed as a professional engineer, these individual are allowed to practice in any discipline of engineering for which they possess minimum competence based upon a combination of education and experience. Practicing outside one’s area of competence is not only a potential danger to the general public, but an action that is not tolerated by state boards and which can result in an individual’s license being revoked.

For many years, NCEES has offered both a Structural I and a Structural II exam. These exams, like all NCEES exams, are created after studies are conducted to find out what practicing professionals in the field believe are the skills and knowledge necessary to practice this discipline of engineering. It is from these surveys of professional engineers working in the specific discipline of engineering that specifications are determined for the knowledge areas that need to be tested. A template with these specifications is then developed and used to write items for the exam. All of the efforts to evaluate the skills and knowledge needed, the design, and the regular maintenance of the exam are done under the authority and oversight of a dedicated group of professional engineers who represent practitioners and educators in the field.

For some years, many states with issues of high seismicity have required candidates who intend to practice structural engineering to successfully complete the NCEES Structural II exam. Often times, there have been additional state-specific examinations that would also be required for anyone who wanted to practice structural engineering in a specific
jurisdiction. To bring more consistency to the examination process, the member boards of NCEES have voted to eliminate the Structural I and II exams and to provide a new structural exam with additional rigor to address the concerns of specific state boards. Effective with the April 2011 exam administration, NCEES will offer a new structural exam that is 16 hours in length. The exam will be divided into two distinct parts, vertical and lateral forces. Candidates must receive acceptable results on both portions of the exam in order to be considered for licensure as a professional engineer.

In designing each examination, NCEES adheres to national testing standards with the goal of providing a test that measures whether an individual has the minimum level of knowledge and skills needed to practice and be in responsible charge in a manner that will protect the general health, safety, and welfare of the public. We constantly evaluate the performance of each item offered on an exam and monitor the performance of candidates to ensure that the item is performing as anticipated and that we are measuring those elements that have been deemed as necessary for the practice of the profession.

Irrespective of how structural engineers may be licensed in the future, there will be the need for a psychometrically and legally defensible examination. NCEES is confident that those examinations currently exist and are used to the best interest of the general public.

Jerry T. Carter
NCEES Executive Director

April 2007

I have been loosely following the education committee and surrounding arguments, but recently came across the NCSEA Basic Education recommended curriculum and it provoked a few thoughts. I will say that I am generally in agreement that the typical bachelor’s degree recipient is underprepared to enter structural engineering. I would caution developing a stringent curriculum. If you force undergraduates to take more narrowly focused coursework, they will be denied the opportunity of taking important classes that aren’t strictly engineering courses. Even if you require a master’s degree the recommended curriculum could preclude some valuable engineering electives. I received a master’s degree and had many classes available outside of this curriculum (stability, bridge design, forensic engineering, reliability, not to mention research and thesis, etc) that would not be options if the recommended curriculum were followed. This is why I would be very careful in dictating an exact curriculum.

In increasing the basic education requirements, one must also consider the market for these services. There are many engineers with a bachelor's degree who are quite capable of performing certain design work and they get compensated based on their expertise. Likewise, engineers with advanced degrees are in high demand for more complex tasks and get compensated accordingly. Increasing the required level of education will increase the cost to enter the field without necessarily increasing the benefit in terms of consultant fees and salaries. I’ve read comparisons to the law, medical, and business professions as an impetus to increase the preparedness of structural engineers, but the flip side of that is
that all 3 post-graduate degrees (MD, JD, MBA) are all extremely expensive and are often cost-prohibitive. It is becoming increasingly difficult to justify the mammoth cost of going to medical school or business school when the end benefit (salary) isn’t growing fast enough to keep up with the cost.

I believe that either increased licensing requirements either through experience or examination and continuing education is the best way to advance the profession.

Sincerely,

Adam Johnson, P.E., LEED AP
Associate
WALTER P MOORE

November 2009

Mr. Hatem & Tuller,

I enjoyed reading your recent article in the November Issue of STRUCTURE. It was a well written article.

I noticed that you used the example of the design of stairs a number of times in the article to illustrate a point relative to design delegation. You may be interested to know that in the case of cast-in-place concrete stairs and exposed monumental stairs (structural steel or concrete) it is common practice for the Structural Engineer of Record (SER) to take responsibility for the direct design of the structure.

You may also be interested to know that the primary reason that a SER will typically delegate the design of structural steel stairs to the fabricator is because there is a multitude of different ways to frame out any given steel stair. As a result most SER’s avoid detailing one particular method over another because 9 times out of 10 the fabricator will want to design and detail the stair using a different framing method from that shown by the SER. Therefore, as a general rule, to avoid the wasted energy of designing and then reviewing an alternate method of framing, the design of stairs are delegated by the SER to others. To avoid change order requests for beams designed by the SER that directly support the stairs (i.e. the beams located around the perimeter of the stair opening) it is also not uncommon for the SER to make conservative, redundant loading assumptions that cover a multitude of different potential stair framing support scenarios.

Somewhat similar logic has also been used when it comes to delegating the design of structural steel connections, however, the primary reason this is commonly done in the industry is because the minuscule fee (typically 1/2% of the estimated construction cost) that most Architects allow their structural consulting engineers to charge barely covers
the cost of the structural design of the main frames, components and foundations, much less the steel connections. For years the AISC tried to push structural design professionals into providing the connection design as a part of the bid documents, however, recently the AISC finally accepted this common practice.

It was also interesting that your article mentioned peer reviews in the last paragraph. As a result of my own professional experience (both as the reviewer and the reviewed) I have taken a keen interest in peer reviews and have published a paper on the subject (see reprint in January 2007 STRUCTURE Magazine: http://structuremag.org/Archives/2007-1/p18-19C-ProfIssuesProjectSpecificPeerReviewGuidelinesJan-07.pdf) and a follow up article on the topic (see reprint in June 2007 STRUCTURE Magazine: http://structuremag.org/Archives/2007-6/C-Sforum-Peer-Review-Stuart.pdf). I am currently working with CASE (ACEC - Council of American Structural Engineers) towards the development of a guideline for project specific peer reviews that can be used for peer reviews that commonly occur outside the realm of legislated, mandated peer reviews in states such as Connecticut and Massachusetts or cities such as Chicago.

D. Matthew Stuart, P.E., S.E., F.ASCE, SECB

June 2008

I read your interesting article, published in Architecture Week. These systems are still in use in Spain, Cuba, Mexico and other Latin American countries.

As an architect in Cuba in 1987-1997 I was in charge of building a factory for the production of precast pre-tensioned beams (viguetas – small beams) for one of these structural systems. The factory included also a facility to produce the concrete block infill tiles. They are called bovedillas in Spanish, due to the fact that structurally they act as small vaults - bovedas. The equipment had been bought in Spain (Cataluña) by the Cuban Ministry of Construction Materials, for which I was working as an employee. (There is no private practice of architecture in Cuba). Once the factory was finished, we started using it in the construction of dwellings and small commercial/office buildings.

In Spain the system is very popular and used for residential and light commercial work, with spans of up to 21 feet, and cantilevered up to 9 feet. The Spanish Normas Basicas de Edificacion of that time used to have a lot of detail about this type of construction.

As for the older types of systems, working in Cuba in restoring older buildings gave us a lot of experience on them too. They were very popular from the 1910’s to the 1930’s, when cast in place concrete replace this kind of systems. Still in the 1950’s a Cuban architect re-introduced a similar type of construction system, with precast concrete beams and clay tile arched units. This system was used until the late 60’s with the trade name PEPSA.
In Mexico several systems of this type are used. You can see an example in http://www.losaryd.com.mx/sistema.htm (Mexico, with infill tiles made of polystyrene), http://vigatec-eirl.com/index.html (Peru), etc.

I hope this e-mail would be of interest to you. If you are interested in some more details about this subject, please do not hesitate to contact me.

Julio Guillén

Viguertas PEPSA:

Typical Spanish “vigueta”:
May 2009

To the Editor:

I was interested in the InFocus article by Jon Schmidt in the May 2009 issue, entitled “The Nature of Theory and Design”. My background is in structural engineering and engineering mechanics. I have not read the book referenced in the article, so I am commenting only on the article itself. I would like to focus on the second and third paragraphs.

They are a little vague to me, but I suppose the point is that theoretical analysis cannot predict exact stress-strain response and “how structures behave” refers to observations of real structures, either by experimental testing or by studying actual failures and successes. I suppose that “bridge the gap” refers to making theory so good that it includes the design details. What I find missing here is that there are certain insights into structural behavior - particularly in complex arrangements of components, sensitivity characteristics, and effects of complex loadings - that can only be obtained by theoretical analysis. With this, I offer the following comments regarding my understanding of theory and practice.

Theory of any field represents the fundamental knowledge of that field, the collection of all information that is considered to be known about the field. It is open-ended so that more information can be added as new information is found, and it is general in nature, making statements about a large class of systems. It seems to me that physics, which is mature and deep in knowledge, is the most advanced science with regard to theoretical mathematical models. Some of these theories have been formalized into axiomatic systems, which are about as high as you can go; e.g., theory of particle mechanics and theory of rigid body mechanics. The reason I bring this up is because theoretical models, no matter how sophisticated, are never intended to encompass details like those required for structural design, where everything has to be translated into a physical detail made to fit into a unique complex physical system. Design is uniquely dependent on the details for that specific structure. Theories speak to generality, to a class of systems, but they can still furnish related information for specific problems. In fact, all three of the independent models, “one for materials, one for individual components and their arrangement, and one for the loads,” are underlain by theory, though this is not necessarily obvious to the designer.

Prediction (specific deduction) is the function of any theory even though it is not exact. It is only as good as the assumptions (axioms) on which the theory is based. Improved assumptions based on additional information make it better, but it will always be
approximate without design details. It doesn’t matter whether steel as a material is linearly elastic or not; it is a logical first approximation to a solution to a complex problem for “small” deformations. If this does not provide sufficient accuracy, then nonlinear effects need to be included, and theory provides the limits of the assumptions and where to go from there when these limits are exceeded. The exact location of joints does not matter if the system is not sensitive to their location. Wind loads can be assumed to have any distribution you choose. The designer has to fill in the difficult details so that everything works together in a safe structure, but theory provides limits, guidelines, and framework for all of this work. I guess I am saying that I do not know how you separate theory from design on any level except within the individual’s focus when dealing with details.

I guess I agree with the statement that theory and design have distinct objectives and cannot be merged by making theory more exact. That is not the function of theory. But they certainly have an interaction that is essential for understanding complex structural behavior and providing an approximation to system response under a specified load for the designer to use. Keep those safety factors intact.

Thank you for your contributions to an interesting and important field.

Sincerely,

Arnold E. Somers, Jr.

January '09

 Commentary from David Shepherd pertaining to the article "Tools for Reducing Carbon Emission due to Cement Consumption"

My thanks to Dr. Kumar Mehta for helping to broaden the perspective on concrete in a sustainable context. (Tools for Reducing Carbon Emission due to Cement Consumption, Structure Magazine, Jan. 2009) In the not so distant past, I have read similar articles that seemed to imply that adding fly-ash was all it took to make concrete “green.” Sustainability encompasses a much wider range of perspective, and advances the concept of balance in business decisions across environmental, economic and social factors.

As the largest manufactured product in the world, the concrete industry has a big footprint, and an equally large responsibility - to make our products better and to help our customers optimize the value in the applications we offer. Dr. Mehta notes multiple strategies to improve the materials aspect of concrete through recycled content, efficiency in mix design, and durability. But let’s not think we can stop there.

Dramatic improvements to reducing the ecological impact of the built environment will require evolutionary and revolutionary changes in how we design, construct and use the structures we create. Life-cycle assessment studies reveal that the operation of a facility contributes 85% to 95% of the environmental impact over the total life cycle of the
building. Understanding this puts into perspective the real value that the design community can offer by specifying durable, energy efficient structures and envelopes. Consider the synergy derived from integration of daylighting and lighting controls for optimized illuminating performance. Significant savings can be achieved. Now imagine a design process where the structural and mechanical engineers work in concert, integrating the structure, envelope and HVAC system; storing energy in the mass, tightening exterior losses, and reducing floor space and structural loads through downsizing HVAC equipment and ductwork.

Also recognize that while climate change and greenhouse gasses are at the forefront of concerns today, sustainable development extends to indoor and outdoor air quality, land use and bio-diversity, water and material resources, urban design and density, and energy dependency among other issues. Dr. Mehta also briefly mentioned designing for disassembly, an excellent application for pre-cast concrete components and other modular products. This notion dovetails with the value provided by concrete as a highly durable material, further reducing the need for replacement materials, landfill space, or energy to re-process products.

Climate change, a struggling economy reliant on foreign energy, and global competitiveness are all compelling reasons to re-visit how we do things. Regardless of your perspective on global warming, sustainable development provides a framework for the challenges we face.

**December '08**

*Commentary from Donald C. McElfresh pertaining to Matthew Stuart's articles "Antiquated Structural Systems Series"

Dear Ms. Sloat:

I am not familiar with Mr. Casper and his working with Ken Bondy, of Atlas, during and after 1966 “when T. Y. Lin’s load balancing method was introduced to our profession…”

However, when I joined T. Y. Lin & Associates in November 1962, we used as our base design reference T. Y.’s book “Design of Prestressed Concrete Structures (1955). In Chapter 11, Slabs, Page 329, he speaks of Two-Way and Simple Flat Slabs, where he refers to “The only basis for their design is the design of reinforced-concrete two-way slabs, moment coefficients for which are available from building codes on reinforced concrete.” This reference is, of course, known to all of us from ACI 318.

In the June 1963 issue of the Journal of the American Concrete Institute, was published “Load-Balancing Method for Design and Analysis of Prestressed Concrete Structures” by T. Y. Lin. Internally, within the company it was already using load balancing from the day I started with TYLA.
Also in 1963 was published T. Y.’s 2nd Edition of “Design of Prestressed Concrete Structures.” Chapter 11, Page 339 was titled “load-balancing method.” Starting with Page 356, “11-5 Two-Dimensional Load Balancing”, T. Y. discusses tendon placement distribution in the middle and column bands. Later on Page 386, under “12-4 Flat Slabs, Theoretical Considerations” T. Y. states that “some theoretical problems till deserve further investigation…B. The proper distribution of the cables along the column and the middle strips. This can be studied by either the elastic theory for plates or the balanced-load explained in Chapter 11.”

Within the four TYLA offices (Van Nuys, CA; Chicago, IL; Dallas, TX; New York City, NY), and the T. Y. Lin International office (San Francisco, CA), during the early 1960’s, there were many inter-office discussions about two-way post-tensioned concrete slabs, column-middle band post-tensioning distribution ratios, possible problems, and how to create the best possible designs.

Possibly the above will shed some additional “light” on an “Antiquated Structural System.”

Sincerely,

Donald C. McElfresh, S.E., P.E.

July ’08

Commentary from Mr. Alfred Commins pertaining to Mr. Ronald Nelson's article "Another View About Shear Wall Hold-down Systems"

All five people who wrote, contributed to or reviewed the article are competitors of Commins Manufacturing with their own agendas and bias. The engineers are presented as if they are independent. They are not independent.

The Author, Mr. Rawn Nelson, is a paid consultant to Zone Four, a supplier of a competitive system that incorporates a ratcheting take-up device. Mr. Edward Chin is a principal of Earthbound Corporation, a supplier of a ratcheting take-up device. Mr. Rick Fine and Mr. William Nelson are paid consultants to Earthbound Corporation. Mr. Steven E. Pryor, S.E., is an employee of Simpson Strong-Tie.

I have three degrees, including one in Mechanical Engineering Technology. I am not a P.E., but I consult with qualified structural engineers as needed for specific tasks. I have a skill set and design experience substantially different from those of the typical structural engineer. I have been designing structural hardware for over 30 years. Products I have designed include the HDA, PAHD, HPAHD, Strap Ties, the Simpson StrongWall, double shear nailing, the SDS screw and many others, I have performed over 300 full-size shear wall tests, many to ICC-ES AC 130.
I have designed and tested many of the devices described in the article. These devices met the criteria in effect at the time they were introduced but may not work as expected during an event such as an extreme windstorm or earthquake.

**Promised Lateral Capacity**

The lateral capacity of shear resisting panels, commonly called shear walls, is defined in the code and can be traced back to ASTM E72. This testing protocol includes tying one end of the shear wall down with a plate and positioning a rod on either side. This hold-down is extremely stiff and very reliable. This testing is for rating the shear resisting capacity of sheathing, nailing and studs. The hold-down connection is to be designed by the engineer and may be a hold-down, building weight or a combination of the two. “Unsafe” is a claim made by Mr. Nelson. I do not know if these walls are “Unsafe,” but I do know that they are not providing the expected lateral performance.

Mr. Nelson is providing a red herring by taking my comment about the four factors required for shear walls to perform out of context. The article concerns tying the shear wall to the foundation or floor below with hold-downs. Other factors such as foundations or shear wall construction are not considered in this article, only vertical connections such as straps, hold-downs and rod systems, and their contribution to the performance of the shear wall.

I am a great fan of shear resisting panels, and in awe of the simplicity and performance that these shear walls provide. The purpose of these articles is to look at the contribution to the performance of shear walls that the hold-down provides and how it might impact the capacity of the whole.

**Are Shear Walls Needed?**

Mr. Nelson’s comment here is also out of context; just the opposite is true. Many engineers look at STRENGTH only but do not look at STRETCH, SHRINKAGE and RELIABILITY. The point is that we need stretch control, shrinkage compensation AND reliability in addition to strength. If elements are missing, the shear wall may not perform as expected.

**Shear Wall Failures**

1. Splitting of sill plates

Mr. Nelson is partially right. Sill plate splitting can occur at any load, but a tight, stiff hold-down can substantially raise the load at which the splitting will happen, sometimes by a factor of 5 or 10. Conversely, a “flexible” hold-down will allow the wall to rotate at a relatively low load, which burdens the bottom plate with a twisting load and can fail the shear wall at a load substantially below its rated capacity.

2. Splitting of vertical wood studs
Splitting of vertical wood studs is not due to wood compression. This splitting is commonly due to bolted eccentric holdowns. Reduced wood section coupled with an eccentrically loaded bolt will tend to split the post. As stated, the loads may be due to wind or seismic loading. But, if bolted hold-downs work so well, why are they tested on a steel jig and used with a 2½ safety factor and a further adjustment for wood? A test on wood would be more realistic.

The looseness associated with commonly available bolted hold-downs is one source of shear wall rotation. Keep in mind that the framer is allowed to drill the hole 1/16 inch oversize. The oversize bolt hole is additive to all other sources of stretch or looseness.

3. Nail pull-through, bending or breaking

The load at the corners that will cause the failure to precipitate is much lower with “loose” or “flexible” hold-downs than with tight, rigid hold-downs. An analogy is to compare a shear wall to a phone book when torn apart by a strong man. This is a commonly known parlor trick. When you try to tear a complete phone book, you can’t do it. But the strong man will bend the book in such a way that pages are loaded sequentially, and he easily tears the book apart. In a similar manner, a flexible hold-down will load some corner nails more than others and the wall will fail sequentially, nail by nail. A tight hold-down will tend to load the nails all at the same time, and the wall will perform much better.

During the research that led to the Simpson Strong-Wall, I discovered that the only way to "push" the wall capacity was to address each and every failure and failure mode as it occurred. The Simpson Strong-Wall has 2½ times the stiffness and strength of a similar wall (at least with a 4-foot x 8-foot wall). This cannot be achieved except with every item properly performing.

Section 3.5.3.1 in AC155 specifies a maximum displacement equal to 0.250 inch. Some manufacturers specify a design capacity at a maximum displacement of 0.282 inches. This 0.282 inch is additive to other system deflections. Uplift deflection is a function of all the component deflections added together. A rod system may include rod stretch, bearing plate crushing, shrinkage compensator deflection, hold-down deflection, and shrinkage. Shrinkage introduces uplift deflection without load. It adds to system deflection and must be included.

AC316 rates shrinkage compensators, but there is a problem. Some shrinkage compensators introduce substantial looseness or backlash. Backlash is a function of rod pitch and internal looseness. I have measured backlash at up to 0.190 inches. What good does it do to introduce a product that eliminates ¼ inch of shrinkage looseness, then adds back 0.190 inch?

Shear Wall Hold-down Checklist
In the last three years, I have had the opportunity to review hold-down systems for some 500 projects. Stretch limits are specified on perhaps one out of 100 projects. When I see stretch limits, I see only rod stretch and not total system stretch; of the 500 jobs, only one specified system stretch. Most engineers do not appear to include bearing plate crushing, and do not look at shrinkage compensator deflection.

1/8-Inch Stretch and Loose Shear Walls

The 1/8-inch deflection limit is a suggestion for what is needed to allow shear walls to perform to their potential. This number should include all tension items including rod, bearing plates, shrinkage compensators, hold-downs, etc. It does not include couplers, nuts and washers. There appears to be a threshold where shear walls held by tight connections perform. We do not know exactly where this threshold is, so the 1/8-inch limit is suggested as a starting point.

To quote the report: Report of a Testing Program of Light-Framed Walls with Wood Sheathed-Shear Panels

“Groups 31 and 32 investigated the effects of free movement at the hold-down anchorages. The free movement allowed was 0.2 inches for Group 31 and 0.4 inches for Group 32. The effects were not significantly different for the free movement allowed. (i.e. 0.2 and 0.4 performed in a similar manner). The Nominal Strength in shear was reduced to about 60 percent of the mean for similar panels. A similar effect on Elastic Shear Stiffness was found. The displacements at the YLS and SLS were relatively unchanged. The SLS shear was unchanged.” (emphasis added)

Several years ago, I was researching the performance of wood shear panels for Simpson Strong Tie. During one test, I loosened a bolt ¼ inch to mirror the effects of shrinkage. The result was a reduction in lateral capacity of about 40%. My experience is congruent with the results of the testing cited above.

Building Settling and Shrinkage

While called a “Shrinkage Table”, this table also includes settling due to misalignment of studs and other assembly issues. By definition, the shrinkage table overestimates settling by assuming worst-case conditions. Most tables look at averages. If averages are used, half the time the shrinkage will be underestimated, and half the time it will be overestimated. Perhaps half of our connections would be loose by using averages, so I took the safe way out. Normal shrinkage estimates assume a degree of precision that is only achieved in the laboratory. The table is not meant to usurp the authority of the engineer; it is only meant as a simple guide.

Shear Wall Designs

Hopefully both groups will include elongation from all sources that contribute to uplift movement including, but not limited to, rod elongation, plate compression (and bending), shrinkage compensator deflection and backlash, hold-down deflection, etc. My
experience with designers is that they tend to use an average, rather than a worst-case condition. Loose connections cannot be good.

**Rod Elongation**

The requirement by AISC that nominal area be used sets up an internal conflict. Since the net area, as Mr. Nelson states, can be only 75% of the nominal area (actually varies from 74% to 79% depending on the diameter and thread pitch), the rod can be overstressed, then to use the actual area for stretch introduces another variable. In this case Mr. Nelson is correct, but we see some manufacturers using one number and others using another. The City of San Francisco in its *Administrative Bulletin AB-084* (AB) on the subject specifies using the nominal area. This area needs clarification. Should ICC-ES ever provide an Acceptance Criteria for Complete Tie-Down Systems, this is an area that needs clarification.

**Strap, Hold-down, or Continuous Tie-down Systems and System Type Take-up Devices**

The article did not suggest that connections be designed to resist compression loads. If connections are degraded by compression, such as buckling of straps or rods, such degradation must be considered. If not considered, then just a few wind or seismic cycles can destroy the connection.

I have encountered a sketch showing a strap combined with a rod system. If the rod system has an allowance for shrinkage, why is a strap used at the top? Is there no shrinkage at the top of the building?

I checked with my sources at APA, and they stated that they have not tested a stacked hold-down system. As manager of research and development at Simpson Strong-Tie until 1997, I do not remember ever testing such a condition. Quite frankly, I never even considered it until three or four years ago. Please remember that the performance of the shear wall is at question here, not necessarily the hold-down itself.

**Shrinkage Control Devices**

Devices that comply with AC316 may be a good addition, unless they introduce extra looseness. Just as all engineers are not the same, even if they studied the same material and passed the same examination, all products are not the same, even if they pass the same Acceptance Criteria. Just as some hold-downs are superior to others, some shrinkage compensators are superior to others. There are differences. Some differences are critical to system performance.

AC316 is now in its fourth revision. It started as an acceptance criteria for screw type devices. It has been expanded to include ratcheting devices.
Evaluating agencies make errors, and it may take a while to correct them. In 2002, I noticed an error in a code report. I notified the manufacturer and ICBO (now ICC-ES). The manufacturer has corrected the error, but even today ICC-ES still publishes the same erroneous detail. Once an error is memorialized, it is extremely difficult to delete.

Every device in every photograph in the March 2008 article is incorrectly installed. I have played no favorites. The problem is that moving elements tend to freeze up during installation, during movement or over time unless extraordinary precautions are taken.

Backlash is a term not found in AC316, but it should be. Backlash, as defined by the American Heritage Dictionary, Second College Edition (1982), is “The play resulting from loose connections between gears or other mechanical elements.” Backlash is a term used by mechanical engineers and is not commonly used or understood by structural engineers.

In rod systems, backlash is the looseness caused by the thread pitch (0.090 inches for a 5/8-inch 11-tpi rod to 0.143 inches for a 1¼-inch 7-tpi rod) plus the movement of internal parts. For comparison, screw devices can have backlash as low as 0.001 inches.

The take-up deflection of 0.012 inches is an estimate based on a series of load-deflection tests. For most purposes, it is not significant.

There are at least five different rod ratcheting devices. There may be specific elements on one variant that are not included on the others. For example, the number of ratcheting elements varies from two to four in the samples that I have examined. Every device that I have inspected includes at least one spring.

According to the American Heritage Dictionary, 2nd edition, page 1178, a spring is a flexible elastic object used to store mechanical energy. The device that Mr. Nelson is associated with includes a spring clip that encloses four jaws and pulls the jaws together so that they can engage the rod. The circular clip is a spring in the classical sense.

In 1996, I discovered rod-ratcheting devices and believed that they would solve shrinkage problems. I set up a cyclic test with a wood shear wall and tested the devices in a system according to AC130 (actually a predecessor to AC130). After several cycles, the system released. An examination of the rod showed a stripping of the threads. The device failed the rod, but the device itself was still functional. While this was a rod failure, it was caused by a concentrated load from the device. Failure load was below the design load of the system.

At the time, I ignored the reason for the failure. Since then, I have tested other rod ratcheting devices and failed the rod in a similar manner. My conclusion: the devices “cross thread”. What is Cross Threading? The definition is to screw together two threaded pieces without aligning the threads correctly. The drawing below shows the problem and a correctly installed part.
Ratchet devices will do the same thing during advancement, and load one segment at a time. This overloads the thread on the rod. The devices and the rod do not stay in line with one another. A building will rock back and forth during an event, and the rod and device will also rock. During the rocking, one segment will ratchet onto the rod, but other segments will be out-of-square to the rod and thus not engage. One of the ratchet segments will lock onto the rod and carry the entire load. The overloaded rod segment will fail.

Of course, the point is that structural engineers are not trained in this area and may not understand the mechanics of moving parts. To my knowledge, none of the people involved with Mr. Nelson's article and none of the engineers at ICC-ES are mechanical engineers. Clearly this is outside their area of expertise.

The photograph above shows a rod with deformed threads. This rod was tested in a ratcheting take-up device, according to AC316. I stopped the test when the threads began to yield.

My experience with rotating devices (of which this is one) is that they tend to bind if the elements are not in line. The drive shaft of a car has universal joints to prevent binding. The prop shaft in a boat either has a universal joint or is aligned carefully to prevent binding. From a practical perspective, binding appears to be a problem, and I am not the only one who sees it this way. A competitor reviewed the product, prepared a nine-page analysis and presented it to ICC-ES in October 2007.

Conclusion
As a solution to this controversy, I suggest that all parties selling rod systems and hold-downs have an independent laboratory test complete shear wall systems under identical conditions. All interested parties, including ICC-ES, could witness the tests and share information.

January '08

I read Gerard Feldmann’s article *Non-Destructive Testing of Reinforced Concrete* that appeared on page 13 of the January issue with great interest, since for the past few years I have been heavily involved in the development of an instrument to perform that type of testing. The article is extremely informative and very comprehensive.

An additional method could be added to the many mentioned by Mr. Feldmann, ultrasound echo (UE). UE finds delaminated areas of concrete and has been shown to accurately identify delaminations when tested on a concrete slab purposely built with such defects. It utilizes two ultrasonic probes; one transmits a stress wave field into the concrete specimen, and the other receives a signal corresponding to the natural response of the concrete specimen in the time domain. A Fast Fourier Transform (FFT) is then applied to the received response so it can be examined in the Frequency Domain.

When testing a floor with the UE method, the frequency response of an area of satisfactory quality will be uniquely determined by the floor thickness (thickness mode of vibration). In this situation the response sensed by the receiver is from wave reflections occurring at the floor to subgrade interface and at the floor surface. Testing at a known thickness to calibrate the concrete wave speed is not required, as the wave speed is calibrated by measurement of the surface P-wave. At an area with a shallow delamination, the response will shift to a lower frequency and be determined by the existence of delamination (flexural mode of vibration).

Ed Pristov

Inspection Instruments, Inc.

Author's Response:

Thanks for the compliment on the article.

Yes, I have heard of the test method you mention, but did not include it in the article due to space limitations and my lack of experience with the method. It seems I should look into it a bit more.

Thanks again,

Jerry Feldmann

September '07
Thank you for using dwgs from the HAER collection for the West Baden Springs Hotel article in the September 2007 issue. I was HAER principal architect during the summer of 1973 when the West Baden Springs Hotel was recorded. The Historic Landmarks Foundation of Indiana and the Indiana Historical Society were cosponsors. It was the second of a two-summer project to record a selection of engineering and industrial sites in Indiana.

These were heady and exciting years for HAER (Est. 1969) as we were defining the new field of industrial archaelogy and engineering heritage by recording engineering phenomenon and industrial processes with measured and interpretive drawings, historical data, and large-format photography. Though I retired nearly four years ago, the program thrives under the capable leadership of Rich O’Connor.

SIA, the Society for Industrial Archeology, is in the process of scanning issues of IA: The Journal of the Society for Industrial Archeology, with The History Cooperative. Unfortunately, Vol. 25, No. 1, which is a theme issue on the first 30yrs of the HAER program, has not been scanned. You can find the journal in any descent university library or it can be ordered from SIA, www.siahq.org. See “HAER: 30 Years of Recording Our Engineering Heritage,” IA: The Journal of the Society for Industrial Archeology, Vol. 25, No.1, 1999.

Eric DeLony
Chief (Retired)
Historic American Engineering Record
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September ‘07

While I think that Larry Muir's article in the July 2007 issue, 5 Common Myths of Steel Design Debunked, raises excellent points, I must take issue with the net section discussion listed under Myth #1. Many members can be connected with an effective net section much greater than 75%, even when heavy truss chord splices are made with 4 rows of bolts (double gage) across the flanges, by simply using one or two "lead-in" rows of bolts in the flanges before the web bolts are started.

In addition, there are many instances where, due to different member actual depths, significant shimming is required between members and connection material. Take for example a heavy truss tension chord splice where the center segment is larger than the next outboard segment. If the outer member size is dictated by an arbitrary 75% net section capacity, a much larger (and more expensive) section will result. This is especially important in our current market state of high-cost raw steel.
So, while I am 100% in agreement with Larry's general concept that less material weight does not always equate to less cost, this is one instance where I believe that it does. I have witnessed engineers attempting to limit member stresses to accommodate the arbitrary AISC listed values without thinking about how the members may be connected and determining whether the full member capacity could indeed be used. Experienced designers working with competent fabricators need to make this type of decision considering many other criteria specific to the project.

Regarding Myth #5, Larry is 100% right on: Owners should only hire top-notch, experienced fabricators and erectors. Unfortunately, not all such firms have the professional credentials, capabilities and competence of Cives, so the Owners do not always have a choice … too bad for us designers.

Sincerely,

W. Steven Hofmeister, P.E., S.E.

Thornton Tomasetti

Kansas City, Missouri

Author's Response:

Steve brings up a good point. I did not intend that an engineer should always increase the member size to accommodate the reduced area due to bolt holes, but rather that the effects of the connection design should be considered on a case by case basis in the main member design. Least weight does not always equal least cost, but sometimes it can.

If the decision is made to reinforce a member rather than increase the member size, the reinforcing and associated welding should be indicated on the engineer’s drawings so that it can be accounted for in the bid. Having the engineer and fabricator work as a team early in the project often allows these issues to be more effectively addressed in the design phase, and can lead to increased economy for all parties.

Steve’s comment about the conservatism inherent in the 75% reduction reflected in the AISC Manual is also valid, and the Manual Committee is currently working to revise these tables to be more accurate and useful to the designer.

Larry S. Muir, P.E.

Cives Steel Corporation

Roswell, Georgia

August ’07
Dear Mr. Weingardt,

I just wanted you to know how much I enjoyed your profile of Timoshenko in the August 2007 STRUCTURE magazine (funny how those back issues pile up). I still refer to Timoshenko's books, and had the pleasure of learning under his co-author Jim Gere, but never knew the turbulent story of Timoshenko's life, or of his wit. Thank you for filling that gap in my technical education!

Leonard Martin Joseph, P.E., S.E.

Senior Vice President

Thornton Tomasetti

Irvine, CA

August '07

Thank you for the article in the August 2007 “Great Achievement” series of STRUCTURE ® about the “father of engineering mechanics,” Stephen P. Tymoshenko.

Among his many contributions to science worth noting was the creation of the Ukrainian Academy of Sciences in Kyiv in 1918, of which he was a founding member. A Ukrainian commemorative postage stamp of 1998 illustrating Mr. Weingardt’s biographical sketch witnesses reverence of his illustrious personality by Ukrainians at home and all over the world.

Roman Wolchuk

Fellow, ASCE

August '07

I would like to comment on the gable end article in your August 2007 issue. Gable end trusses are not allowed in Miami Dade County, and for a good reason. They leave the wall under them basically unbraced, unless the top of the wall under them has a structural element that can span the length of the wall and brace it for out-of-plane wind forces.

For taller walls (ever more present as homes get more and more expensive), there is basically no wood-framed bracing system that can be designed to transfer those forces back in the remaining of the roof structure unless fairly large members and bolted connections are utilized.
A gable end wall should have continuous framing from the foundation all the way to the top of the wall, where roof sheathing and blocking can resist the forces. If the walls are masonry, the cells should be reinforced as such. If they are wood framed, they should have studs continuous from the foundation to the roof sheathing.

Eugenio M. Santiago, P.E.

Chief Building Official

Key Biscayne, Florida

esantiago@keybiscayne.fl.gov

Response

Thank you for your comments regarding the article on Wood Truss Gable End Frames. The scope of this article was to provide basic and additional design considerations for gable end frames (i.e., vertical and lateral load considerations).

Page 55 of the article provides information related to gable end frames under lateral loads acting parallel and perpendicular to their plane. In addition, your comments concerning continuous framing are addressed on page 55 and 56; citing section 2304.3.4 of the FBC with regards to Gable End Wall Bracing. Both Jim and I conducted several claim investigations in Florida related to hurricane and tornado damages, and are familiar with building code requirements in Broward and Dade County.

Figures 6, 7, 8, 10 and 11 (full article) deal with gable end frames resisting lateral loads, stress and/or deflection concentrations and recommended details for gable end walls and/or frames. Figure 11 is taken from SSTD-10-99 (Standard for Hurricane Resistant Residential Construction), which is also provided in AF&PA’s WFCM (Wood Frame Construction Manual).

(The full article can be viewed in STRUCTURE’s online archives.)

Agron Gjinolli, P.E.

WTCA

Agjinolli@qualtim.com

July '07

The article The Case for an Engineer of Record for a Metal Building System in the March 2007 issue of STRUCTURE magazine contains good recommendations for Owners who are considering Metal Building Systems.
One item that is questionable from the Engineer of Record (EOR) perspective is the section on “Inspection Services”. For most other projects, the EOR agrees to “observe” the structure during construction to ascertain substantial compliance with the Contract Documents that the EOR prepared. According to the IBC, “inspections” are performed by the Special Inspector hired by the Owner.

It is an abdication of responsibility that the metal building manufacturer does not observe the metal building system during construction. The location of the manufacturer is not a valid excuse. The metal building steel erector is generally under the same contract as the metal building designer. The design, erection details, drawing quality, construction conformance with the drawings, and construction quality are all the responsibility of the metal building designer working with the erector.

While it is preferable to have the EOR review the Order Document, that is not always done. Metal building manufacturers should adhere to the current industry practice of inspection by the Special Inspector and site observation by the designer, rather than shift such responsibility to the EOR.

Sincerely,

Lawrence R. Chute, P.E.

DESAI/NASR CONSULTING ENGINEERS, INC.

Mr. Chute brings up an interesting point about whether a metal building manufacturer should be responsible for the observation or inspection of a metal building system during construction. We feel that the “special inspector” hired by the building owner should be someone who is independent from the erector/building manufacturer. The metal building manufacturer has a conflict of interest on top of the logistical problems associated with performing the inspection, because the erector is commonly the customer of the manufacturer. Some manufacturers do include onsite inspection as part of their contract, especially with regard to accepting warranty responsibilities on large or complex projects, but if a manufacturer does not include this service, it should not be considered an abdication of responsibility.

W. Lee Shoemaker, P.E., Ph.D.

July '07

Metal Building Systems and the EOR

Dr. Shoemaker's article on metal building system structures in the March 2007 Structural Forum column presents a comprehensive overview of the metal building system world. Metal building systems provide very efficient structures by keeping material costs at a minimum. As a structural engineer who has been responsible for the erection engineering of these structures, I have found that low material cost can require extreme efforts to
afford safe erection of a structure. Girders and rigid frames that are adequate, when all intermediate framing members are installed, can require elaborate analysis and special bracing to allow their erection. Anyone entertaining the use of a metal building system structure needs to be aware that the *Metal Building Systems Manual* excludes structure erection design from the manufacturer's responsibility unless specified otherwise by the purchaser. Metal building systems can provide durable, low-cost structures; their design should include a method for getting them erected, safely.

Alan D. Fisher, P.E.
Manager, Construction Structures Group
Cianbro Corporation

*Mr. Fisher is absolutely correct that metal buildings, like other types of construction, require careful planning for the safe sequencing of erection. Metal building erectors are no different from other steel erectors that are typically responsible for determining the best erection sequence and method based on their available equipment and experience. If the metal building erector requires additional special engineering to help determine the erection procedure, such as Mr. Fisher’s expertise, it seems more appropriate that the knowledge of the site-specific conditions be handled by someone other than the metal building manufacturer, because of logistical and practical constraints. Local design and oversight would seem to be the most realistic method to achieve the goal of safe erection.*

W. Lee Shoemaker, P.E., Ph.D.

July '07

**Communicating with CAD**

The author hit the nail squarely on the head with the InFocus column *Cad-How It Has Changed the Way We Think* (STRUCTURE ©, April 2007). I passed it to several others here, who immediately had the same reaction. Good job!!

Every one of your points were accurate: the lack of x-refs being bound to the transmitted files, the lack of dimensions and the Architect’s attitude – “just scale the cad drawing” and when you do it has some funky fraction at the end. All that stuff is the norm around here and a source of everyday frustration. Thanks for letting us know we are not alone in this gripe.

Stephen M Rudner PE
Robert Darvas Associates PC
440 South Main Street
Ann Arbor, Mi. 48104
I was reading the article titled “Is Four Years Enough?” in the April 2007 issue of the STURCTURE ® magazine and felt that some readers might be lead to a wrongful conclusion based upon a couple of sentences contained in the article.

The articles states “ASCE drafted a model registration law for consideration by the National Council of Examiners for Engineering and Surveying (NCEES). The model law incorporates the above education requirements.” My concern is that action taken last year by NCEES to amend its Model Law to require a Bachelor's Degree plus 30 additional credits would be attributed to a similar study that has been under review by ASCE during recent years.

NCEES was fully aware of ASCE’s review of this matter, and the development of a body of knowledge on what one might need to know in order to be qualified for the professional practice of engineering. NCEES has studied this matter separate and apart from ASCE, which included work by two distinct and separate NCEES task forces. After several years of study and review by these task forces, a motion was made and ultimately approved by NCEES to amend its Model Law to require, effective 2015, that candidates for the Principles and Practice exam must have a Bachelor's Degree in Engineering plus 30 additional credits. The NCEES Uniform Procedures and Legislative Guidelines (UPLG) Committee has been tasked this year with defining what constitutes 30 additional credits as satisfactory for pursuing licensure.

The point of clarification is that the action taken by NCEES during their 2006 Annual Meeting was not as the result of the ASCE study, but as the result of NCEES’ own study and findings.

Jerry Carter

Associate Executive Director

National Council of Examiners for Engineering and Surveying

May ’07

The debate between thermally Restrained versus thermally Unrestrained assemblies, has existed for over 30 years. Industry groups, such as the American Iron and Steel Institute (AISI) have sponsored studies, which lead to the conclusion that all structural steel frames, independent of the level of restraint, can be classified as Restrained assemblies.
Unfortunately, the conclusions drawn from these studies do not take into account the effect of the structural frame on the overall fire protection package.

Restrained Assembly beams generally exhibit significant deflections before the required hourly rating is met. A building’s fire protection package, in addition to the sprayed cementitious fireproofing, includes other important features, such as compartmentation, suppression systems and detection systems. While a Restrained structure, in the right situation, may transfer loads without collapse, excessive deflections will likely compromise the compartmentation offered by firestopping and smoke damper systems and jeopardize the effectiveness of sprinkler systems. Non-functioning compartmentation and suppression permit the quick spread of smoke and fire to areas outside of the point of origin.

Actual fire events have proven the effectiveness of Unrestrained classification. The Occidental Tower fire in Los Angeles (Nov 1976), State Office Building fire in Olympia Washington (Oct 1983), First Interstate Bank fire in Los Angeles (May 1988), and Union Bank Building fire in San Francisco are all examples of structures protected with cementitious fireproofing applied to Unrestrained thicknesses. In all cases, the damage to the structural steel was minor and the buildings were open for business shortly after the fire event.

Some failed fire protection packages resulting from Restrained classification or sprinkler tradeoffs are: The McCormack Place fire in Chicago (Jan 1967), One New York Plaza fire (Aug 1970), K Mart Distribution Center fire in Falls Township, PA (Jun 1982), and One Meridian Plaza fire in Philadelphia (Feb 1991). All sustained structural damage beyond repair resulting in major financial losses.

In short, Spray-on Fireproofing works and Unrestrained protection affords the best protection for structural steel framed buildings. It insures full compliance with building codes in all jurisdictions, without assuming the liability associated with designating a building as thermally restrained. The cost difference between fireproofing a building to Unrestrained versus a Restrained classification is generally less than 1% of the overall cost of the building. This amount can prove to be the difference between saving a structure with minimal loss of life and a catastrophic disaster.

Michael Giardinelli

W.R. Grace & Company

Author, Fireproofing Steel Structures (STRUCTURE, February 2007)

May '07

CASE and the members of the CASE Fire Protection Committee found the article, Fireproofing Steel Structures (STRUCTURE®, February 2007), to be potentially misleading. The premise of the article, that structural steel assemblies should be
considered to be thermally unrestrained, is not supported by engineering data. There has been significant engineering research over the past 30 years that suggests that structural steel assemblies behave as thermally restrained in almost all instances.

- Restained Fire Resistance Ratings in Structural Steel Buildings by Gewain and Troup (*see reference above*) states that most common types of steel-framed construction are classified as thermally restrained.
- Appendix X3 of ASTM E119 lists the few instances where individual steel beams and girders, or steel framed floor and roof assemblies, are classified as unrestrained.
- AISC Design Guide No. 19, *Fire Resistance of Structural Steel Framing*, clearly indicates that the position suggested in the article is incorrect.

It requires considerably more sprayed fire-resistant material to achieve an unrestrained fire rating, rather than a restrained fire rating. This appears to be a conservative approach that could present an economic burden to the project. These conditions must be carefully evaluated by the designer.

*CASE Executive Committee*

*Edward W. Pence, Jr., P.E., S.E., F.ASCE, Chair 2006/07*

May ’07

Much effort has been spent disseminating the research sponsored by the American Iron and Steel Institute (AISI) that confirmed the performance of steel framing and the use of restrained ratings in the selection of fire protection. The conclusion of that research remains valid – steel-framed structures can be considered thermally restrained.


The issue of restrained vs. unrestrained construction is unique to the United States. It has been a source of confusion since the concept’s introduction in 1970. To assist the design professional in determining this parameter, AISC has collected information demonstrating that steel framed construction qualifies for a restrained classification and makes it available so that the provisions of section 703.2.3 of the International Building Code can be satisfied. This being the case, the opinion expressed in the article serves to perpetuate unnecessary questions that have already been answered repeatedly.

*John L. Ruddy*

*Director of Building Design, AISC*
March '07

I am writing in response to a portion of Jon Schmidt’s InFocus article in the January 2007 issue of STRUCTURE®. Jon, stated, “I cannot help but wonder if the ‘commodization’ of engineering services is inevitable if we do not significantly raise the bar of entry into our profession. The moves toward specialty certification and, eventually, separate licensure are certainly steps in the right direction, but may not be enough in the end.” Perhaps I misunderstand the intent, but it seems to me that we are considering professional licensure as a means of job protection. We exist professionally to facilitate the construction of structures — safe structures. If additional licensure or educational requirements are needed to assure safe design, very well. If we begin, however, to consider these methods as some form of trade protection tariff, we are amiss.

Our age is a turbulent one. The ability to communicate globally has altered and is still altering our business fundamentally. This article touched not only on the educational future of engineering, but also on our own fears of our professional future. In a free market, when we perform useful, needed services, we are and will be compensated. If there are quicker, easier or cheaper means of providing those same services, the market will shift the demand to those other sources. If the services performed there are inferior, our services will command more demand and/or money, or at least we hope so!

Perhaps the real goal is not a free market goal. Perhaps the real goal also has an aim to create a union of sorts. I don't know, but I think our intentions ought to be clear at least to ourselves.

Bret Wickham

Contra Costa County, California

I appreciate Mr. Wick’s comments and do not necessarily disagree with them. When I talk about “raising the bar”, I do not mean making it more difficult per se, but rather making it more rigorous. Protecting the public is, indeed, the primary intent — not protecting our turf.

Response from Jon Schmidt, P.E., SECB

Feb ’07

Mr. Rouis’ December 2006 article, A Better Base, was well written and informative. I offer the attached Pin Wheel Isolation Joint Detail as an alternate to his Figure 7a. The Pin Wheel method of simultaneously isolating the column as a part of the installation of sawn control joints is used very often in warehouse distribution facilities in which the slab is cast after the erection of the steel.

An alternate to Mr. Rouis’ Figure 1 is the Pocket Form Isolator. Information on this pre-manufactured item can be found at www.isolationpocket.com.