

Failure Case Studies

Steel Structures



Forensic Engineering Division

Edited by

Navid Nastar, Ph.D., P.E., S.E.

Rui Liu, Ph.D., P.E.

ASCE

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Preface

This publication was developed by the Education Committee of the Forensic Engineering Division of the American Society of Civil Engineers (ASCE). The current document is the first in the *Failure Case Studies* series in Civil Engineering and presents eight case studies of failures observed in steel structures between 1970 and 2013.

Failures in Civil Engineering: Structural, Foundation and Geoenvironmental Case Studies was first published by ASCE in 1995. Edited by Robin Shepherd and J. David Frost, the publication collected short descriptions and relevant references for 43 failure case studies. Subsequently, the Education Committee of the Forensic Engineering Division of ASCE published a second edition of the document in 2013, retitled *Failure Case Studies in Civil Engineering: Structures, Foundations, and the Geoenvironment*, with updates and additional case studies.

The *Failure Case Studies* series is a follow-on project to previous efforts by the ASCE Forensic Engineering Division and is intended to promote learning from failures by disseminating information regarding previous failure cases. The purpose of the *Failure Case Studies* series and their predecessor documents is to promote failure literacy to improve the practice of civil engineering and to reduce risk to the public.

Each case study in this document presents a summary description of a documented civil engineering failure, followed by *lessons learned* from the failure and *references* for further study.

The reader is reminded that each case study only contains the findings of the research and literature review by the author(s) who directly contributed to that particular case study, based on the published results of failure investigations for each case. The contents of this document do not represent the professional or personal opinions and views of the editors, authors, contributors, or ASCE.

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CHAPTER 1

West Gate Bridge Collapse, 1970

The West Gate Bridge, in Melbourne, Victoria, Australia, spans the Yarra River to connect Melbourne and its western suburbs (Figure 1-1). The bridge and its approaches, with a total length of 2,583 m (8,473 ft), rise from ground level on the west bank of the Yarra River onto a concrete viaduct, cross the river in five steel spans 58.5 m (192 ft) above the water, and finally extend onto another concrete viaduct that reaches the east bank and descends back to ground level. The project was designed to carry four lanes of traffic in each direction at speeds of 112.7 km/h (70 mi/h).

Construction of the bridge began on April 22, 1968, in a period when this type of box girder bridge had become popular. The “skin” of the system supports local loads by resisting bending, shearing, torsion, and other load effects. In a box girder bridge, the low profile helps improve aerodynamic stability. The main drawback is that the plates that make up the “skin” are subject to distortion during fabrication, and buckling is difficult to predict.

The central bridge was designed as a five-span continuous steel box girder with stay cables radiating from two towers (Figure 1-1). The bridge consisted of a series of trapezoidal steel boxes. Each box was 4 m (13.1 ft) deep, 16 m (52.5 ft) long, and 25.5 m (83.5 ft) wide at the top flange, with a cantilever bracket (used to build extensions between the arms of the bridge) at each side extending out another 3.2 m (10.5 ft). The usual construction method was to fabricate the boxes on the ground and to raise and bolt them in the air to create a cantilever bridge, one that has two sections (or arms) extending from opposite banks and joining in the middle, above the water.

The collapse occurred on October 15, 1970, while the bridge was still under construction. A half-span of the bridge had been assembled on the ground. When it was raised into position, it was discovered that the camber of the new half-span was about 114 mm (4.5 in.) out of alignment compared to the camber of the previous half-span to which it was to be attached ([West Gate Bridge Royal Commission 1971](#)). To force the camber to match and allow the new span to be attached, seven 8-ton concrete blocks, known as kentledge, were used to load down the new half-span and remove the camber difference. This large load caused one of the inner upper panels of the bridge to buckle, and an 88.9 mm (3.5 in)



Figure 1-1. Photo of the West Gate Bridge, Melbourne, Australia, taken on February 15, 2006.

Source: Courtesy of Wikimedia Commons, Wikipedia.

bulge occurred in the steel. The bulge had to be flattened out to allow the intermediate diaphragm connections to be made, and it was decided to accomplish this by removing bolts from the transverse splice. After more than 30 bolts were removed, the panel flattened, but the buckle spread into the adjacent two outer upper panels. At this point, the new half-span was no longer capable of supporting its own weight; however, it did not fall immediately as it was able to partially bear on the previous half-span. Although members of the construction team were concerned by the additional buckling, they failed to appreciate the immediate danger it presented, and there was no immediate evacuation of the workers. At approximately 11:50 a.m., 50 minutes after the spread of buckling, the new half-span and the previous half-span, a length of 112 m (367 ft), collapsed. Approximately 2,000 tonnes (2,000 tons) of steel, concrete, machinery, and tools crashed down into the Yarra River, killing 35 workers and injuring many others.

The Royal Commission of Inquiry placed most of the blame for the collapse on the design engineer, but also found fault with the contractor. The design engineer was found to have failed to design the bridge to have a great enough margin of safety both during erection and when it was to be put into service. In addition, the design engineer did not properly check the safety of the erection proposals put forth by the contractor ([West Gate Bridge Royal Commission 1971](#)). The fact that the design engineer was located halfway around the world, in London, further complicated matters. The contractor was found to have not recognized the need to exercise special care required to implement its unusual erection plan for constructing the half-spans separately before attempting to connect them after raising them into position ([West Gate Bridge Royal Commission 1971](#)). The contractor at the time of the collapse came onto the project part

way through construction due to delays by the original contractor. The replacement contractor had limited experience with structural steel construction, and yet had to follow the erection procedure established by the original contractor.

Eventually, work continued on the broken bridge, and it was finally opened on November 15, 1978, with the total cost having escalated from 22 million to 200 million Australian dollars. The West Gate Bridge still stands today as one of Australia's iconic structures.

LESSONS LEARNED

For a structure of this type, size, complexity, and importance, a detailed erection sequence plan must be engineered and implemented. The plan should consider the strength and stability of the structure at each stage of construction. Participation by a team of qualified and experienced engineers and contractors is essential to ensure that all phases of construction and all construction loads are considered. Regular planning and coordination meetings are needed involving all parties. The contractor's qualifications and experience with the special type of construction should be carefully reviewed and considered.

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CHAPTER 2

University of Washington Stadium Collapse, 1987

At approximately 10 a.m. on February 25, 1987, the north addition to the University of Washington football stadium in Seattle, Washington, collapsed suddenly. The project was under construction at the time. The structure is a very large steel-framed assembly providing seating for 20,000 people. It is as tall as a 15-story building. Nine bents of steel framework, over 52 m (170 ft) high, are each supported by four 711-mm (28 in.) diameter steel pipe columns, filled with concrete, on cast-in-place concrete piles. The steel framing for each bent includes a cantilever truss with large wide-flange sections forming the top chord and steel pipe sections forming the bottom chord (Figure 2-1). At the time of the collapse, two of the nine steel bents had been erected. The pipe columns were not yet filled with concrete, and very little of the designed structure was in place to provide lateral stability. The completed design would provide resistance to lateral loads by braced frame action in one direction, and by cross-bracing and diaphragm action of the seat plates and metal roof deck in the other direction. None of these were fully in place at the time of the collapse.

Fortunately, there was sufficient warning that this collapse was imminent. About an hour before the collapse, a construction worker observed that a structural steel member was beginning to buckle. If not for this report and the actions of an alert construction superintendent in clearing the construction site, there would certainly have been serious injuries and fatalities. The collapse sequence was recorded by an accomplished Seattle architectural photographer, John Stamets, who just happened to be in the vicinity at the time (Figure 2-1).

Inadequate temporary support was clearly the cause of this failure. There was no indication of either wind or seismic disturbance. However, lateral loads were not necessary to cause this collapse. Without bracing, a cantilevered structure of this configuration is unstable under gravity loads alone. Forensic investigations concluded that there was no evidence of design defect. Investigators blamed an incomplete system of temporary guying cables. Some stabilizing cables had reportedly been removed on the morning of the collapse to facilitate the progress of the steel erection.

One interesting side note on this failure is that during a subsequent visit to Seattle, a labor union official stated, "This is what happens when imported steel is

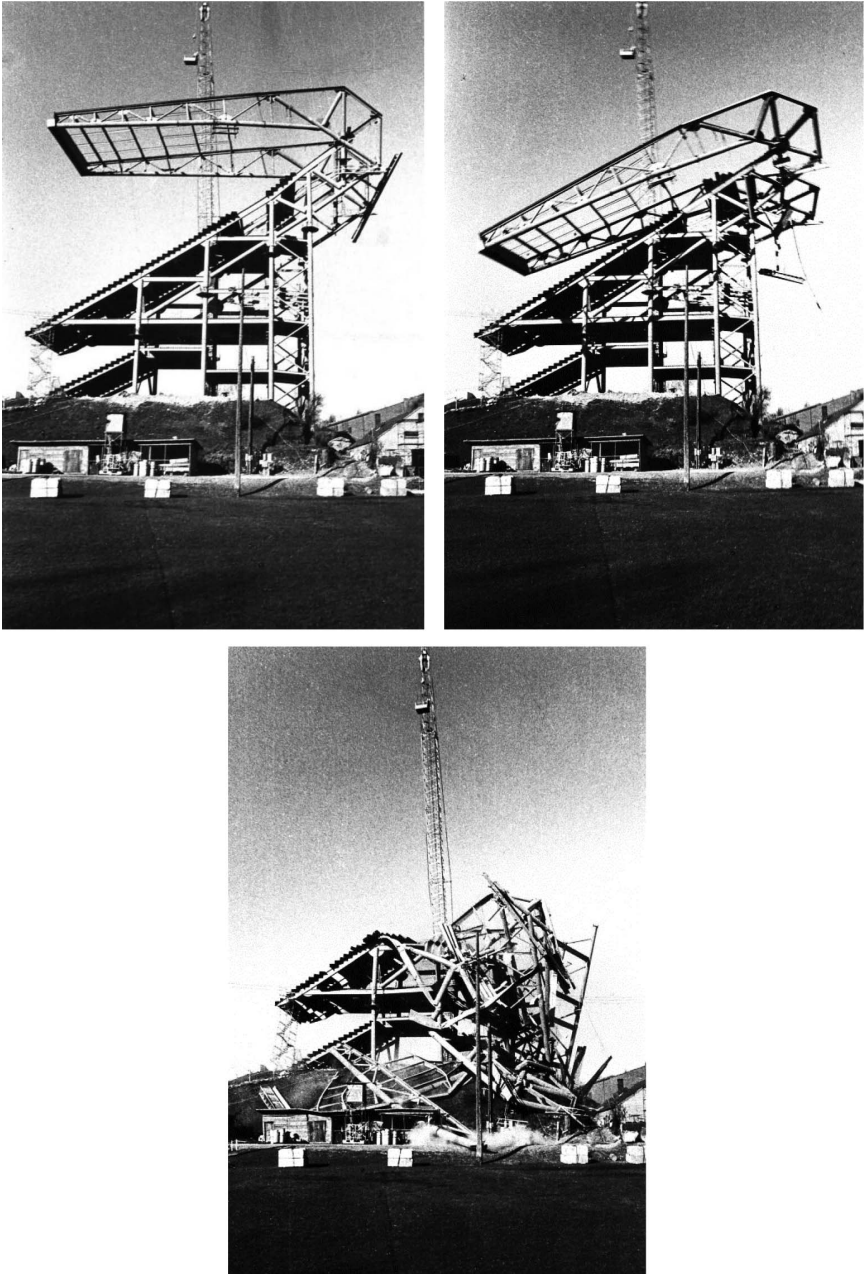


Figure 2-1. Collapse sequence recorded by John Stamets on February 25, 1987.
Source: University of Washington Libraries, Special Collections, John Stamets Photograph Collection.

used.” The steel was imported, but it was found to meet design specifications. Any knowledgeable observer should recognize that this stability failure had nothing to do with the quality of the structural material, but rather it was about erection procedures and the lack of temporary support during construction.

This project was being constructed under an extremely tight schedule. The general contractor recovered from the catastrophic setback and went on to complete the project in time for the start of the football season. The remaining seven bents were erected while the steel was being fabricated to replace the damaged section. Although the economic loss was substantial, the fact that no one was injured, and the project was successfully completed on schedule is an inspiring management story.

LESSONS LEARNED

During construction of structures of this type, temporary stability must be provided until the design features required for structural stability are in place. Stability-related failures during erection of steel, timber, and precast concrete assemblies are all too frequent. Provision of temporary support is typically the responsibility of the erection contractor. It is critical to have a detailed erection plan, along with on-site personnel qualified to implement the plan. The proper sequence of assembly and the provision of adequate temporary bracing for the incomplete structure are essential.

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CHAPTER 3

Damage to Steel Moment-Resisting Frames during the Northridge Earthquake, 1994

On January 17, 1994, the magnitude 6.7 Northridge Earthquake shook Southern California. The earthquake epicenter was near Northridge, California about 32.2 km (20 mi) northwest of downtown Los Angeles. Sadly, 60 people lost their lives, more than 7,000 suffered injuries, and more than 40,000 buildings of all construction types were reported damaged in Los Angeles, Ventura, Orange, and San Bernardino Counties ([USGS 2016](#)).

Among the damaged buildings in the Los Angeles area were a significant number of the more modern Steel Moment-Resisting Frame (SMRF) structures, which suffered major structural damage and in some cases were red-tagged as unsafe for occupancy. The observed damage and the resulting advances in the design of SMRFs is a perfect example of engineers advancing the art of structural design by “learning from our failures”, understanding, researching, and designing better structural systems.

The observed damage to SMRFs was typically in the form of cracks in the complete joint penetration welds between the beam and column flanges. In some instances, the cracks propagated into the columns, resulting in column fracture; and in some cases, cracks were observed in beam flanges. Figure 3-1 depicts the typical detail of the web-bolted, flange-welded moment connection commonly used prior to the Northridge Earthquake, often referred to as Pre-Northridge connection. Figure 3-2 schematically illustrates the typical connection weld fracture observed after the Northridge Earthquake.

Such damage observations came as a surprise to the structural and earthquake engineering community. Prior to the earthquake, the SMRF buildings were widely believed to have ductile seismic performance capable of resisting earthquakes of this size with little or no damage to their structural system and elements. The observed failure, however, demonstrated a brittle performance with little ductility. Many SMRF connections failed at relatively low stress levels and under only a few significant cycles of vibration, during which they were expected to remain essentially elastic. Figure 3-3 presents photos of some of the observed connection damage reported in the FEMA 355E report ([FEMA 2000a](#)).

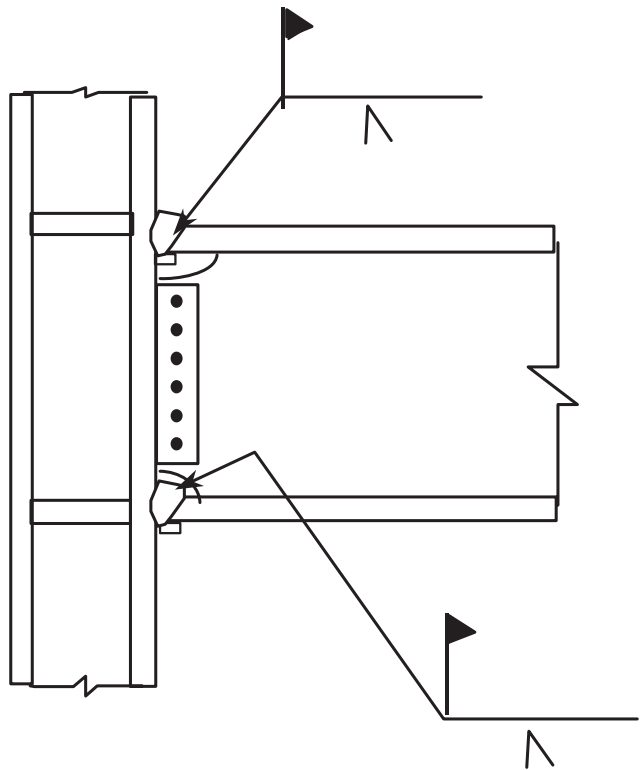


Figure 3-1. Typical pre-Northridge flange-welded, web-bolted moment-resisting connection.

Source: Figure 1-1 in FEMA 350 and FEMA 355E (2000).

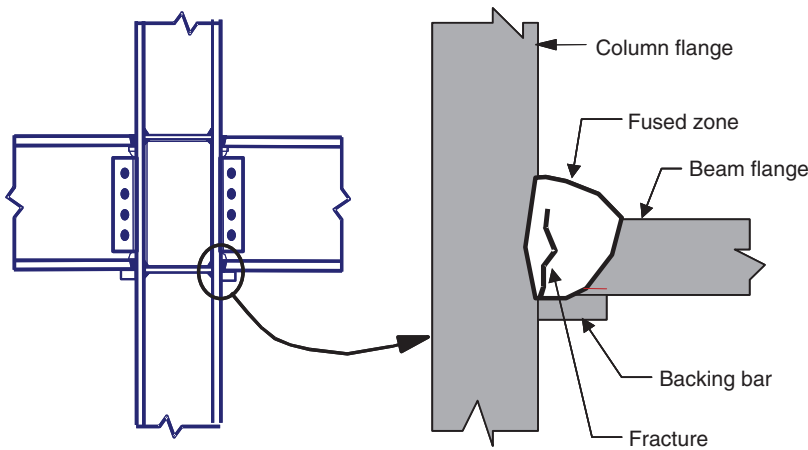


Figure 3-2. Observed weld fracture in beam-column connections.

Source: Figure 1-2 in FEMA 350 and FEMA 355E (2000).

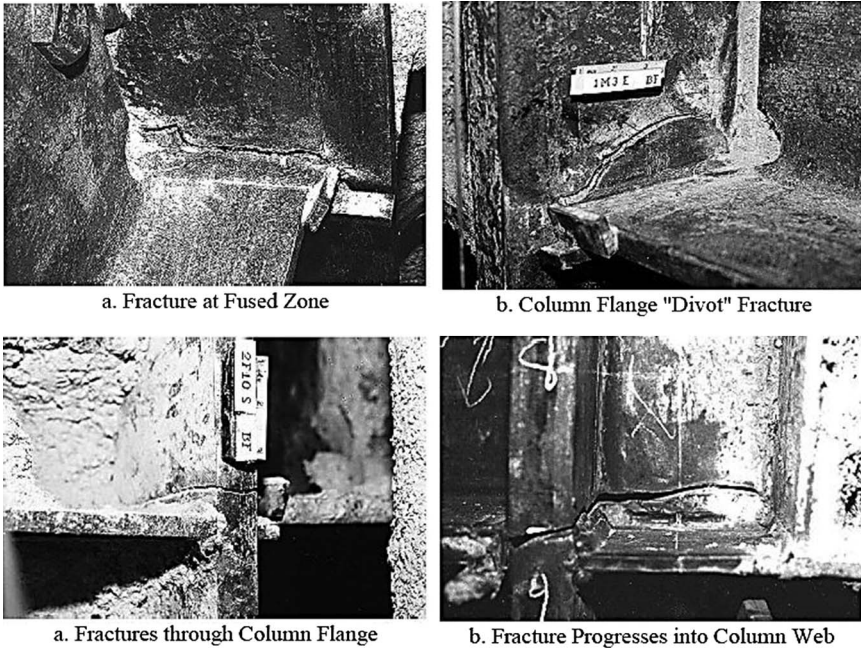


Figure 3-3. Damage to beam-to-column connections.

Source: Figures 1-3 and 1-4 in FEMA 350 and FEMA 355E (2000).

Figures 3-4 and 3-5 show examples of the documented instances of fracture in columns.

In response to the unexpected damage observations, the SAC Steel Project, including a number of university research projects, was funded by the Federal Emergency Management Agency (FEMA) as a joint venture between the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and the Consortium of Universities for Research in Earthquake Engineering (CUREe) to investigate the cause of the unexpected brittle behavior of the SMRF connections.

The findings of the SAC project were primarily published in the FEMA 350, 351, 352, 353, and 355 series of reports, which highlighted concerns over the SMRF design and construction practices prior to the Northridge Earthquake. The results of these studies initially concentrated on the local connection defects that potentially initiated the observed cracks. It was primarily concluded that weld imperfections and low quality, lack of proper weld quality control measures, and issues associated with the backing bars and access holes commonly used in construction of the Pre-Northridge connections were the main causes of the observed damage.

Other studies suggest that the observed lack of ductility may be attributed to low-cycle fatigue in SMRF connections, contribution of higher modes of vibration, near fault effects, relatively weak columns, geometry of the connection, constraints

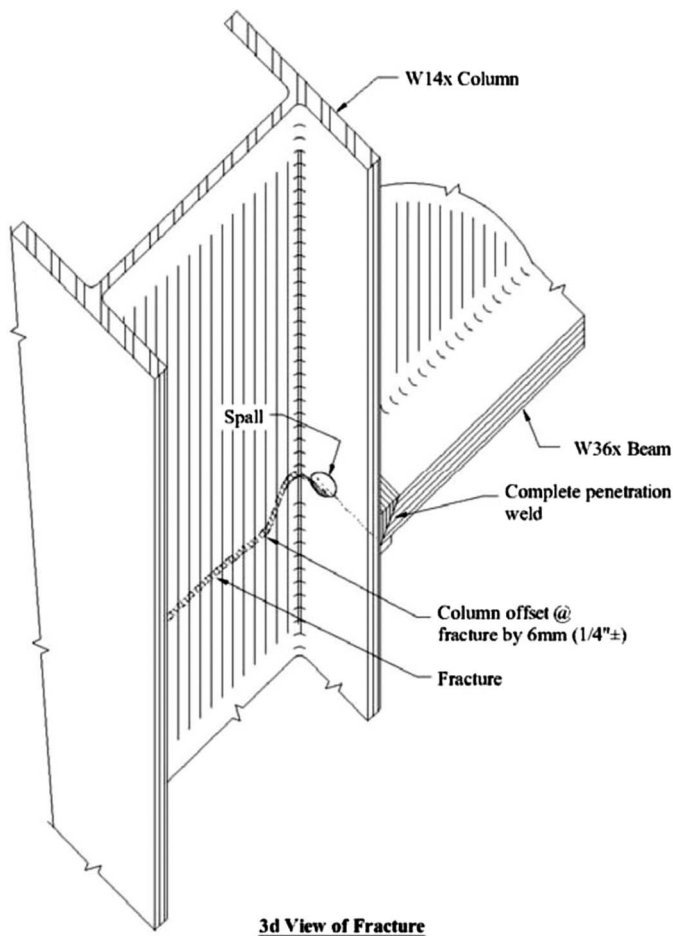


Figure 3-4. Damage to a moment frame column at the beam–column connection of an 11-story building.

Source: Figure 1.16b in [Maranian \(2010\)](#).

and stress concentrations built into the connection, secondary stresses, and effects of member sizes in the performance of the connections. The discussion on the significance of these factors continues today.

LESSONS LEARNED

Findings and recommendations from the SAC Steel Project were incorporated into revised connection design and construction practices and became the basis for modern codes. New Post-Northridge connection details were developed and



Figure 3-5. Brittle fracture observed after the Northridge Earthquake in a W14 steel column.

Source: Figure 2-6 in [Hamburger et al. \(2009\)](#).

tested, with a focus on weld quality and the geometry of the connection. The American Institute of Steel Construction (AISC) incorporated the recommendations in their seismic design provisions and prequalified connections, which were subsequently adopted by newer generations of building codes. Each earthquake demonstrates that our knowledge is limited, and new lessons about the material behavior and performance of structural systems help us advance the “state of the art” of our profession.

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CHAPTER 4

Colorado State Route 470 Overpass Collapse, 2004

At 10:04 a.m. on May 15, 2004, a fabricated steel girder for the Colorado State Route 470 overpass over Interstate 70 in Golden, Colorado, rotated and sagged from its temporary bracing, striking a sport utility vehicle which was eastbound on I-70 (Figure 4-1). The girder struck the hood of the vehicle and sheared off the top of the vehicle, while the lower portion of the vehicle traveled an additional 249 m (818 ft). All three occupants of the vehicle were killed. The girder was part of a bridge-widening project. It had been erected three days earlier and temporarily braced. The I-70/C-470 interchange construction project was intended to improve traffic capacity and safety at the interchange of these two routes, including the widening of C-470 by adding two lanes. The project was funded by the Federal Highway Administration (FHWA) and the State of Colorado. The Colorado Department of Transportation (CDOT) prequalified Asphalt Specialties as the prime contractor on the project. Asphalt Specialties subcontracted with steel erection firm Ridge Erection Company, Inc. ("Ridge") to erect the three new girders needed to widen the C-470 bridge. There was no requirement to prequalify subcontractors.

A planning meeting was held on March 24, 2004, which discussed the installation of the girders. However, there was little discussion on the temporary bracing of the girders. Neither the contractor nor the subcontractor was required to submit plans for the erection or temporary bracing of the girders, and no detailed temporary bracing (falsework) drawings were prepared. The temporary bracing was to consist of steel angles connected to girder number 1 (Figure 4-2) and the existing bridge deck, and cross bracing between girders 1 and 2. The subcontractor's safety officer, who came up with the bracing scheme, was not a Registered Professional Engineer and had no engineering education or training. No Registered Professional Engineer reviewed or was otherwise directly involved.

I-70 was scheduled to be closed from 9:00 p.m. on Tuesday, May 11, until 5:00 a.m. on Wednesday, May 12, so that the erection and temporary bracing of the two girders could take place. Ridge encountered several problems, which delayed completion of the work. They did not have the proper tools on hand to remove the shipping bolts, and initially had raised one of the girder sections backwards. The CDOT lead inspector caught the mistake. After rotating the



Figure 4-1. Fallen girder and top portion of the accident vehicle on I-70 (looking north).

Source: Figure 1 in [NTSB \(2006\)](#).



Figure 4-2. Post-accident photograph of one of the angle-shaped steel braces used to brace the collapsed girder.

Source: Figure 3 in [NTSB \(2006\)](#).

section and splicing the sections together, they realized that they would not have enough time to install both girders. At 4:00 a.m. they began installing the bracing on the single girder. They fabricated the steel angle bracing on site, cutting one leg of the angle to make the necessary bend, resulting in a net area of steel which was approximately 50% of the section's gross area.

The braces were to be connected to the girders with bolts, and to the concrete bridge deck with expansion bolts. Ridge's workers had difficulty attaching the expansion bolts to the deck, and after several unsuccessful attempts, they acquired different expansion bolts from their shop and used them to attach the bracing to the deck. After the installation of the bracing for girder 1 was completed (Figure 4-3), I-70 was reopened for traffic.

Due to inclement weather, completion of the girder installation (installation of girder 2 and the cross bracing) was postponed, until the accident occurred, more than three days later.

The report of the investigating committee identified the following causes of failure (NTSB 2006):

- The girder had not been installed plumb (Figure 4-4). It was installed 4.26 degrees out of plumb at the south abutment and 2.33 degrees out of plumb at the center bridge pier, leaning toward the existing bridge. The



Figure 4-3. Diagram overlaid on photograph to show the girder that failed in its erected position along with the five braces in relation to the C-470 bridge deck.

Source: Figure 4 in NTSB (2006).



Figure 4-4. Portion of the photograph taken by a driver on May 13, 2004, while the accident girder was in place over I-70.

Source: Figure 5 in NTSB (2006).

bracing had been installed while the structure was still partially supported by a crane, without allowing it to first reach its dead load deflection. The loads on the bracing were significantly higher than they would have been if the girder had been installed plumb.

- The five lateral braces were to be bolted flush with the existing bridge deck, but none of the braces were flush with the deck.
- The bolt hole diameters in the existing bridge deck measured 0.90 in., whereas the bolts were only 0.75 in. With these oversized holes, a horizontal load was required to maintain some pullout resistance.
- With the exception of one bolt, the bolts were not embedded in the concrete to the minimum required depth of 3.25 in. This improper installation of the expansion bolts prevented them from reaching their pull-out design capacity.
- Wind loads and lateral vibrations on the braces eventually led to the incorrectly installed expansion bolts pulling out of the concrete. Analysis showed that if brace number 2 failed, the collapse would occur.

The National Transportation Safety Board (NTSB) determined that the probable cause of the girder collapse was “the failure of the girder’s temporary bracing system due to insufficient planning by Ridge Erection Company, Inc., Asphalt Specialties, Inc., and the Colorado Department of Transportation, which were responsible for putting the girder and its bracing in place, and due to deficiencies in the installation of the girder and the bracing, so that the bracing ultimately failed to adequately secure the out-of-plumb girder to the existing

bridge deck.” (NTSB 2006). Additionally, lack of sufficient oversight by CDOT contributed to the problem.

The failure to install girder 2 and the cross-bracing was a major deviation from the intended temporary bracing (falsework) plan. However, the intended plan was never drawn or submitted. Even if the contractor or subcontractor had submitted the intended detailed plan for review by an engineer, it would have been reviewed as submitted (with the installation of girder 2 and the cross bracing)—not as actually constructed at the time of collapse.

LESSONS LEARNED

Proper design of temporary bracing and construction for bridge projects is essential. Following this incident, recommendations were made by NTSB to FHWA, Occupational Safety and Health Administration (OSHA), and CDOT regarding the design and construction of falsework and formwork (NTSB 2006). Most notably, the designs must be prepared or approved by a Registered Professional Engineer. Also, erection subcontractors performing safety-critical work on highways must be prequalified. Although state DOT personnel should not supervise the subcontractor(s), they should intervene when a subcontractor exhibits a lack of competence. This case study also shows the need for redundancy in temporary as well as permanent structures.

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CHAPTER 5

Pittsburgh Convention Center Expansion Joint Failure, 2007

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The David L. Lawrence Convention Center in downtown Pittsburgh was built between 2000 and 2003 for the Sports & Exhibition Authority of Pittsburgh and Allegheny County, or SEA. It is a four-level structure, roughly 265×174 m (870×570 ft) in plan. An expansion joint along column line X9, roughly 146 m (480 ft) from the west end, splits the center approximately in two. The connections of the expansion joint are exposed to ambient temperatures (WJE 2008).

At about 1:30 p.m. on Monday, February 5, 2007, a tractor-trailer was parked on the second-floor loading dock of the convention center. The trailer had just hitched its bumper to the loading dock. Under the weight, a 6.1×18 m (20×60 ft) section of concrete slab and the steel beam supporting it collapsed. There were, fortunately, no injuries. The ambient temperature at the time was about -19 to -14 °C (-3 to 7 °F). Problems with 18 misaligned portions of the column foundations had halted construction work in November 2001, and the collapse had occurred in the vicinity of the shifted columns. Work was resumed once repairs had been made to some precast concrete beams. The \$370 million building opened in 2003. The collapse led to the cancellation of the Pittsburgh International Auto Show (Ritchie and Houser 2007, WJE 2008).

Several independent investigations were carried out. The owner hired the Wiss, Janney, Elstner Associates Cleveland office and Leslie E. Robertson Associates, and the architect hired Thornton Tomasetti. A follow-up story was printed in the *Pittsburgh Tribune-Review* after the investigators briefed the public on their initial findings. It was disclosed that a beam had failed at a similar connection in 2005, causing the beam to drop 64 mm (2 1/2 in.) before it was stopped by a column, but the earlier collapse had not been disclosed to city and county officials. Another failure had occurred in February 2002 during construction. In the 2002 collapse, an ironworker was killed when incorrect nuts were used to connect some of the steel structural elements (Houser and Ritchie 2007). The failed expansion joint detail is shown in Figure 6-10 of Delatte 2009.

The expansion joint essentially divided the building into two large sections. Twenty-five slotted expansion joint connections were provided along the expansion joint.

The WJE investigation addressed issues with design, materials, fabrication, and construction. The main design issue was that the slotted hole expansion joint was almost guaranteed to fail because of significant friction and insufficient room for thermal contraction. Also, the design drawings did not prohibit bolt threads on the bearing surface, which increased friction further. Other design errors did not contribute significantly to the collapse. These errors included inadequate length of the slot and no limitation on bolt torque (WJE 2008).

Materials and fabrication issues included steel with too high a strength: ASTM A992, not A36. This high strength kept the angles from bending and caused them instead to tear away at the weld. Other problems were bolt threads bearing on the slot surface and slots fabricated with a bump in the middle. There was little evidence of the bolts actually sliding within the slot; instead, the threads seem to have worn away at the same spot. Washer plates were added, although they were not shown on the drawings (WJE 2008).

The construction issues included the use of the wrong type of steel angles, with high-strength steel. A drift pin was lodged tightly into one of the bolt holes, helping to lock the connection. Other construction errors that did not contribute significantly to the collapse were a missing bolt and the torque of the bolts (WJE 2008).

LESSONS LEARNED

Joints and connections are often weak points in structures. They have to fulfill the competing needs of allowing movement and preventing stress buildup and, on the other hand, of transferring loads.

WJE estimated the amount of free movement required at the expansion joint as 41 mm (1.6 in.), based on a thermal coefficient for steel of 0.00001 mm/mm/°C (0.000006 in./in./°F), a temperature change of 28 °C (50 °F), and a length equal to half the length of the building: 133 m (435 ft or 5,220 in.). The WJE finite element analysis estimated that the distortion of the ASTM A992 high-strength angles imposed a force of 630 kN (140,000 lb) of tension at the connection welds, with approximately 8 mm (0.3 in.) of displacement. With lower strength A36 steel, the force on the welds would have been reduced by 40%.

There are two obvious problems with a slotted hole expansion joint of this type. The first is that the slot must be long enough, and the bolt must be centered, so that the bolt can move freely back and forth in the slot without bearing against either edge. If the beam contracts and the bolt bears against the edge of the slot, the connection is locked and will behave as a fixed connection and pull apart when any additional contraction occurs. The second problem is that the bolts must be loose enough to keep from locking the joint. If the bolts are tightened, which can easily happen during construction, the joint won't work. Corrosion, paint, and debris can also lock the joint. For this particular type of joint to work, it must be built and maintained perfectly, and that may not happen in the field.

During cold-weather contraction, as shown in Figure 6-10 of [Delatte 2009](#), the beam pulls away from the connection. The two slotted angles were welded only at the outer edges, which made them weak in tension. When they pulled free, the connection failed.

A more reliable detail for this type of connection is a low-friction supporting bracket. To retrofit the connections at the convention center, the bolts were removed and Teflon-coated supporting seats were added. This detail was designed by Thornton Tomasetti and is shown in Figure 6-10 of [Delatte 2009](#).

The failed expansion joint did not conform to the American Institute of Steel Construction (AISC) *Manual of Steel Construction* (1998). Charles Carter, an AISC engineer, observed that the manual recommends two ways to build an expansion joint. One way is to provide a double line of structural columns, one on each side of the joint, and in essence create two buildings that can move independently. The other is a low-friction sliding connection, such as the shelf support connection developed for the retrofit. Carter noted that the original detail, with the sliding steel bolts, would create a lot of friction and would probably not be an effective joint. The recommended retrofit of the 25 expansion joint connections was estimated to cost \$350,000 ([Houser 2007](#)).

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CHAPTER 6

I-35W Bridge Collapse, 2007

At approximately 6:05 p.m. on August 1, 2007, the I-35W Bridge (National Bridge Inventory structure No. 9340) over the Mississippi River in Minneapolis, Minnesota, collapsed within a matter of seconds (Figure 6-1). The eight-lane, 581 m (1,907 ft) bridge was designed in 1965 with 14 spans and 13 reinforced concrete piers. The main portion of the bridge was the 324 m (1,064 ft) long deck truss, comprised of two Warren-type trusses supported by four piers (Figure 6-2). Cast-in-place concrete deck rested on steel stringers along the direction of traffic, which were supported by steel floor trusses arranged perpendicular to the direction of traffic. The steel floor trusses were carried by the two non-load-path-redundant Warren trusses. The bridge was opened to traffic in 1967 with a minimum 165 mm (6.5 in.) thick concrete deck slab. The minimum thickness was increased to 216 mm (8.5 in.) in 1977, and new concrete parapets and guard rails were added to the bridge in 1998.

At the time of collapse, the concrete deck was under another major reconstruction, with construction materials and equipment concentrated in the center span above the upper 10th node (U10, numbered from the south—see Figure 6-3). The catastrophic failure of the main deck truss started from a lateral shifting instability of the upper end of a diagonal member connected to the U10 node on the west side, which caused the fracture of the bowing 12.7 mm (0.5 in.) thick gusset plates at the U10 nodes (Figure 6-4). As a result a 139 m (456 ft) long section of the main span of the bridge fell into the river. The collapse killed 13 people and injured 145. Fire and rescue units were notified of the accident immediately, and the rescue was coordinated well among different departments.

The National Transportation Safety Board (NTSB), the Federal Highway Administration (FHWA), and the Minnesota Department of Transportation (MnDOT) performed thorough investigations of the collapse. The complete findings of the investigation can be found in [NTSB \(2008a\)](#) and [Wiss, Janney, Elstner Associates \(2008\)](#).

All possible collapse scenarios were investigated, including undetected fatigue cracks caused by cyclic load, strength reduction due to corrosion damage in gusset plates, settlement of the piers, capacity of steel trusses, pre-existing cracks in the bridge superstructure, temperature effects, and design of the gusset plates. The investigations concluded that the collapse was caused by inadequate capacity of gusset plate connections due to an error in the original design, manifested in the



Figure 6-1. Collapsed bridge center section.

Source: Figure 14 in [NTSB \(2008a\)](#).



Figure 6-2. Center span of the I-35W Bridge before collapse.

Source: [NSTB \(2008b\)](#).

bowing of the gusset plates. The substantial increases of dead load due to the 1977 and 1998 bridge modifications, along with the lateral shifting instability of the diagonal member connected to the U10 node due to concentrated loads and traffic load on the day of the accident, caused the fracture of the improperly designed gusset plates at the U10 node and the following collapse of the main deck truss.

The bridge design firm was ultimately responsible for the incorrect design of the gusset plates. The gusset plate connections would have been stronger if



Figure 6-3. Node U10 West before collapse.

Source: [NTSB \(2008b\)](#).

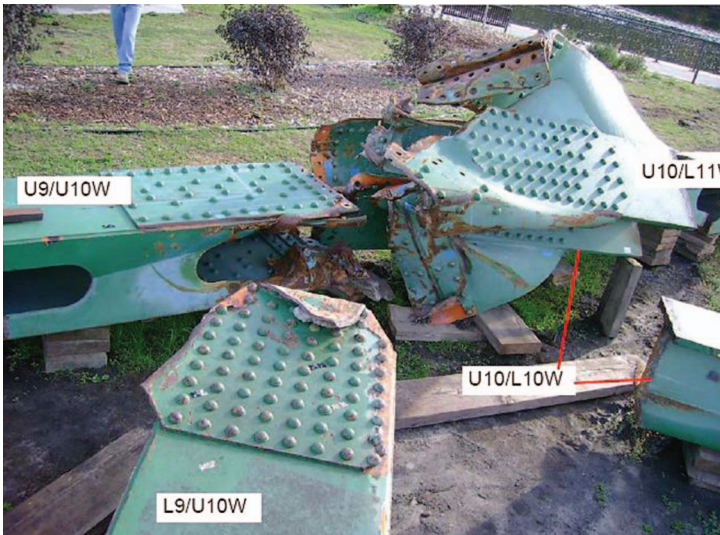


Figure 6-4. Node U10 West after collapse.

Source: [NTSB \(2008b\)](#).

properly designed. However, the detailed design of the gusset plates was not evaluated by federal or state authorities to detect the design errors because it was not standard practice for them to do so. Also, the American Association of State Highway and Transportation Officials (AASHTO) guidance did not include gusset

plates in load rating, and AASHTO did not provide specific guidance for gusset plate inspection. MnDOT ignored the importance of the gusset plates by incorrectly assuming that the connections were designed more conservatively than other truss elements. MnDOT also failed to perform an engineering review of the stockpiling of raw materials, due to lack of specifications and guidelines.

The I-35W Bridge was rated as “Structurally Deficient” in the National Bridge Inventory. (There are more than 72,000 Structurally Deficient bridges in the United States.) Nevertheless, the investigation concluded that the failure of the bridge was not caused by the conditions responsible for the Structurally Deficient rating.

LESSONS LEARNED

The collapse of the I-35W Bridge renewed public attention to our aging infrastructure and warned the engineering community about the importance of performance evaluation of gusset plate connections in steel trusses. [Higgins et al. \(2010\)](#) proposed a rapid ranking procedure for bridge engineers to identify potentially problematic gusset plate connections. A study commissioned by the Washington State Department of Transportation and FHWA developed a rapid procedure to evaluate the maximum stresses and probability of yielding ([Berman et al. 2011](#)). In addition, a forensic investigation model was developed by [Brando et al. \(2012\)](#) to store, sort, and analyze design, repair, and maintenance data. This model was then used to create a 3D simulation of the collapse mechanics of the I-35W Bridge.

Some of the recommendations made by NTSB to FHWA and AASHTO to address the major safety issues identified in the investigations included ([NTSB 2008a](#)):

- “Develop and implement . . . a bridge design quality assurance/quality control program, to be used by the States and other bridge owners, that includes procedures to detect and correct bridge design errors before the design plans are made final . . .”.
- “Require that bridge owners assess the truss bridges in their inventories to identify locations where visual inspections may not detect gusset plate corrosion and where, therefore, appropriate nondestructive evaluation technologies should be used to assess gusset plate condition . . .”.
- “Revise . . . Manual for Bridge Evaluation to include guidance for conducting load ratings on new bridges before they are placed in service.”
- “Develop specifications and guidelines for use by bridge owners to ensure that construction loads and stockpiled raw materials placed on a structure during construction or maintenance projects do not overload the structural members or their connections.”
- “Include gusset plates as a commonly recognized (CoRe) structural element . . .”.

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CHAPTER 7

Elliot Lake Algo Centre Mall Collapse, 2012

At 2:18 p.m. on June 23, 2012, a portion of the rooftop parking deck of the Algo Centre Mall in Elliot Lake, Ontario, Canada, collapsed without warning (Figure 7-1). The mall was a shopping center and social hub built in 1979–1980 with 15,000 m² (165,000 ft²) of retail space, 1,200 m² (12,500 ft²) of offices, and 334 parking spaces on the rooftop parking deck. The mall played an important role in the overall economic well-being of the retirement community in Elliot Lake. The parking deck was constructed using 203 mm (8 in.) deep and 1.2 m (4 ft) wide hollow core prestressed precast concrete slab panels, typically spanning 9.1 m (30 ft). The prestressed units were supported by steel beams, and had 75–100 mm (3–4 in.) of concrete topping, with insulation placed underneath the hollow core slabs (Figure 7-2). A V shape was formed along the long side of two adjacent hollow core slabs, and it was filled with grout. The concrete topping was cast directly on top of the panels in the parking area. It was designed to increase the capacity of the hollow core slabs and to function as a part of the waterproofing system. The 75–100 mm (3–4 in.) varied depth was intended to create a sloping surface to drain the water. Three expansion joints to accommodate thermal expansion and contraction were created in the building.

The sudden collapse occurred at a steel frame connection between a beam and a column, which was heavily corroded by salty slush due to rainwater leaking through the failed waterproofing of the parking deck for decades (Figures 7-3 and 7-4). Two people were killed and 19 were injured in this heartbreaking incident. The Elliot Lake Commission of Inquiry was established to investigate the cause of the collapse. It concluded that the collapse was “one of human, not material, failures. Many of those whose calling or occupation touched the Mall displayed failings—its designers and builders, its owners, some architects and engineers, as well as the municipal and provincial officials charged with the duty of protecting the public. Some of these failings were minor; some were not. They ranged from apathy, neglect, and indifference through mediocrity, ineptitude, and incompetence to outright greed, obfuscation, and duplicity . . .” (Elliot Lake Inquiry 2014).

The roof collapse was directly caused by the failure of the unique, unprecedented, and untested waterproofing system, which was created by pouring concrete over hollow core slabs without a full waterproofing membrane. The



Figure 7-1. Collapse of the rooftop.
Source: Figure 2.3.1 in [Elliot Lake Inquiry \(2014\)](#) © Queen’s Printer for Ontario, 2014.

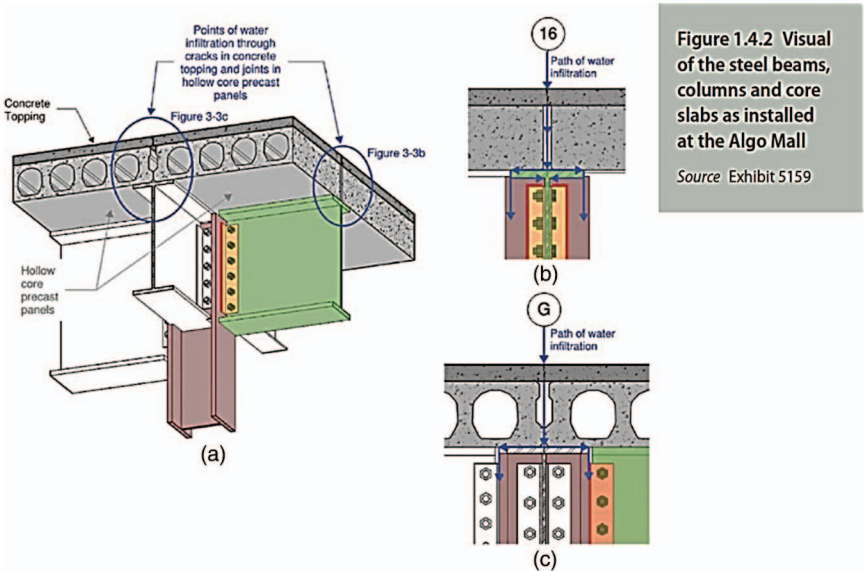


Figure 7-2. Connection of the steel beams, columns, and slabs.
Source: Figure 1.4.2 in [Elliot Lake Inquiry \(2014\)](#) © Queen’s Printer for Ontario, 2014.

rooftop was poorly sloped, which resulted in water ponding in numerous locations on the deck. Crack-control joints were applied in the concrete topping above the grouted space along the long side of every third hollow core slab. Additional cracking where the hollow core panels abutted was expected to be limited due to

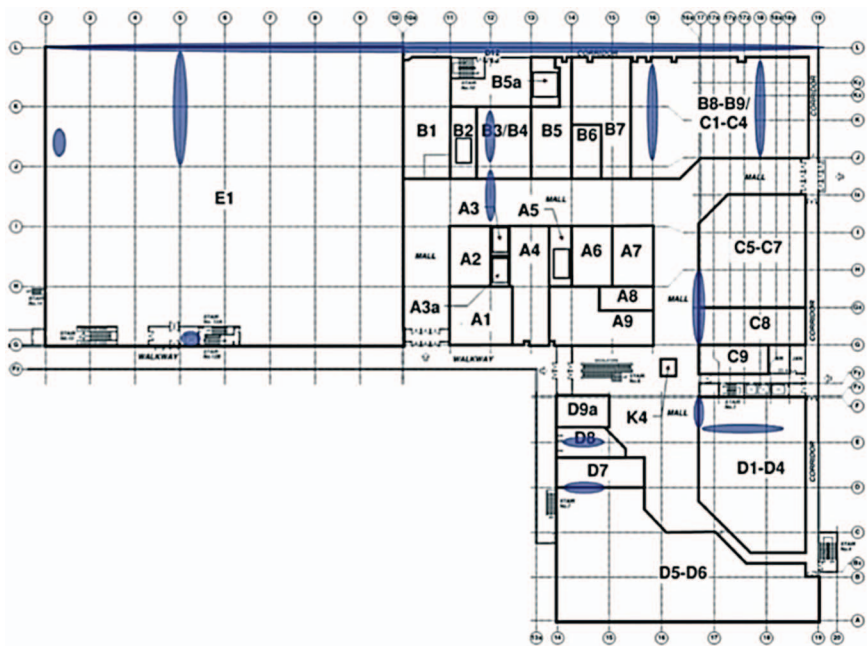


Figure 7-3. Mapping of the water leakage on the floor plan.
Source: Figure 1.4.9 in *Elliot Lake Inquiry* (2014) © Queen’s Printer for Ontario, 2014.



Figure 7-4. Severely corroded connection exposed after the roof collapse.
Source: Figure 1.3.5 in *Elliot Lake Inquiry* (2014) © Queen’s Printer for Ontario, 2014.

presence of these crack-control joints. However, almost the entire concrete topping cracked above these points. The expansion joints were sealed with a device to accommodate thermal movement of the parking deck, but they became a source of significant leaks. Also, the installation of the concrete topping occurred in wet, cold, and snowy conditions, which negatively impacted its performance. From day one, water penetrated through the joints, which were compromised over time, and found its way in between the concrete topping and the slab. The water flowed onto the supporting steel framing and caused significant corrosion over the lifetime of the rooftop deck, especially when it contained deicers in winter. In addition, the hollow core slab panels were simply supported by the steel beams and were not tied to each other. When the steel frame connection failed, there were no alternative load paths or redundancy to prevent the collapse of the roof.

During its lifetime, the property belonged to three owners. The commission criticized all three and found that they had missed numerous opportunities to fix the water-leak issue. The original owner did not hire a consultant to manage the entire design and construction, but hired a specialized firm to design and install the ineffective waterproofing system. After the roof leak was noticed, an engineering firm recommended that the owner install a full waterproofing membrane; otherwise the leaking would compromise the structural capacity of the roof system. However, the owner only patched and sealed the roof cracks and sold the problem to the second owner, who continued the repair that was “not working” ([Elliot Lake Inquiry 2014](#)). The third owner did not want to invest in the “black hole” ([Elliot Lake Inquiry 2014](#)) to fix the issue, but hired an engineer who performed only very cursory and incomplete inspections and incorrectly rated the structural condition of the mall as reasonable. Throughout the lifetime of the parking deck, none of the owners, architects, engineers, or public officials anticipated the severe corrosion of the steel framing below caused by decades of water leakage.

LESSONS LEARNED

The commission recommended that buildings be inspected periodically by a qualified structural engineer according to minimum structural maintenance standards for buildings. The record of inspection, including condition rating and recommendation of repairs, should be easily accessible and understandable to the owners and be filed in a publically accessible registry. The local jurisdiction should have the power to require buildings that do not meet the minimum standards to be repaired or demolished for the protection of the public. From this sad incident, design professionals should learn that unusual and untested design features may have unexpected consequences. The redundancy, continuity, and robustness of the structural systems should be considered in the design of all buildings. Although a successful design is often a function of what is technically possible and what is economically feasible, in considering building maintenance, repair, and

rehabilitation, the more critical scenarios should be considered, and necessary actions based on governing codes and standards should be taken to protect the public.

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CHAPTER 8

Skagit River Bridge Collapse, 2013

On May 23, 2013, a truck hauling an oversized steel container on a flatbed trailer was traveling south on Interstate 5 near Mount Vernon, Washington. The trucking company had obtained a Washington State Department of Transportation (WSDOT) permit for the travel route and was following a pilot vehicle escort. Despite these precautions, the oversized container impacted critical structural members of a through-truss bridge over the Skagit River, causing the collapse of a 49 m (160 ft) simple-span section of the 339 m (1,112 ft) bridge into the river.

The I-5 Skagit River Bridge is configured in twelve simple spans (Figure 8-1). Four approach girder spans on the north and south ends of the bridge lead to four central through-truss spans. The oversized container struck portions of the bridge in the first through truss encountered while traveling south (Span 8 in Figure 8-1). This impact caused the collapse of the span (Figure 8-2). Two passenger vehicles, one pulling a travel-trailer, fell into the river with the bridge (NTSB 2014). Fortunately, of the eight vehicle occupants involved in the collapse, only three received minor injuries, and there were no fatalities. However, the loss of the bridge caused major economic disruption for six months. The economic loss affected a region far beyond the local communities. The entire north-south transportation corridor from western Canada to the western United States was impacted. Repair costs for the bridge totaled \$15 million, but the overall economic consequences were far greater.

A comprehensive review of the I-5 Skagit River Bridge collapse is provided by Stark, Benekohal, Fahnestock, LaFave, He, and Wittenkeller (2016). This paper discusses a wide range of transportation-related issues pertaining to over-height load strike incidents and this collapse in particular, “such as permitting, route databases, signage, buffer between posted and actual clearance, insurance coverage, and pilot car requirements and operation.” (Stark et al. 2016). The structural analysis presented in this paper provides information on the truss design, damage sequence, and failure mode.

The major factor in this incident was the vertical clearance of the bridge, in terms of both magnitude and variability (Figure 8-3). There were variable vertical clearances across the portal frame, as well as along the bridge from span to span.

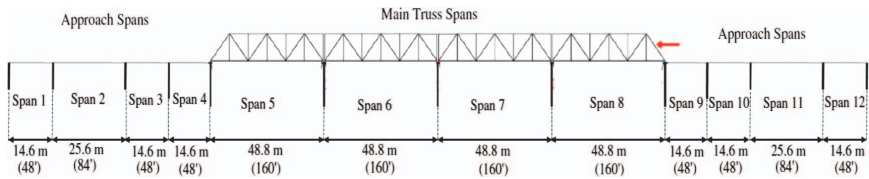


Figure 8-1. I-5 Skagit River Bridge elevation; arrow indicates where the bridge was struck.

Source: Figure 2 in [Stark et al. \(2016\)](#).



Figure 8-2. Through-truss Span 8 collapsed into the Skagit River.

Source: Figure 3 in [Stark et al. \(2016\)](#), reprinted from [NTSB \(2014\)](#).

Along a permitted route, clearances can vary from lane to lane. The permitting process is discussed by [Stark et al. \(2016\)](#), who note that “Permitting of oversized vehicle routes is not federally regulated . . .”. States establish their own permit requirements, which vary from state to state. Online permitting is often available to trucking companies. Signage informing oversize-vehicle and escort-vehicle drivers of the vertical clearance of an upcoming structure is an important factor in preventing this type of incident. In this case, Washington State placed the responsibility for determining route clearance on the trucking company and allowed the permit to be issued online, without review by WSDOT personnel. Washington State had recorded clearance heights for the bridge that were available to the trucking company, but only the minimum and maximum clearances were recorded. These heights did not indicate that the lane where the truck struck the truss had a lower clearance than the adjacent lane, where the truck would have cleared.

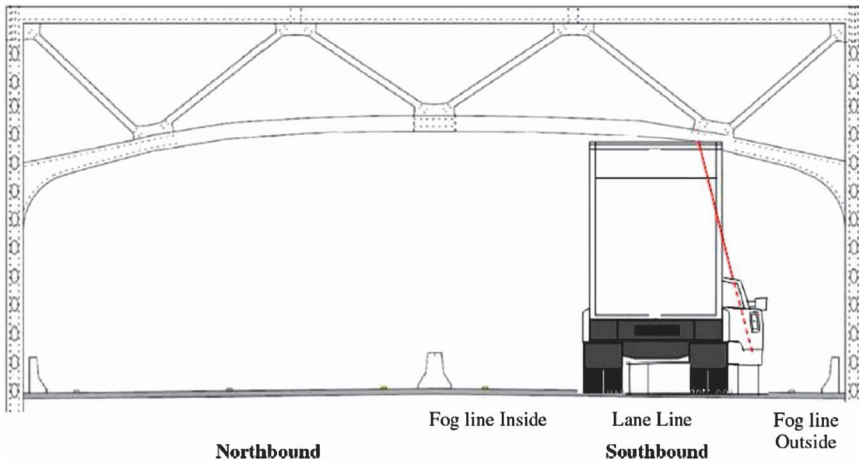


Figure 8-3. Section through bridge showing relationship of oversized vehicle height to portal frame clearance.

Source: Figure 7 in [Stark et al. \(2016\)](#).

Agencies usually record vertical clearances that are less than the actual measured heights to account for variability in clearance measurements and to reduce risk ([Stark et al. 2016](#)). Such clearance “buffers” attempt to account for conditions such as vehicles bouncing vertically while traveling on the roadway, snow, added pavement surfaces, and measurement inaccuracies ([Stark et al. 2016](#)). Pilot vehicles are intended to provide a further layer of protection. However, in the I-5 Skagit River Bridge collapse, the escort-vehicle driver was reported to have been using a mobile telephone at the time of the accident ([NTSB 2014](#)) and did not alert the oversize vehicle to the change in vertical clearance on the bridge ([Stark et al. 2016](#)).

LESSONS LEARNED

While the collapse of the I-5 Skagit River Bridge could have been prevented by increased precautions and extra diligence in the permitting of oversized loads and pilot-escort procedures, there are some lessons in this failure for design engineers. This bridge was classified as a “Fracture Critical” bridge, defined by [AASHTO \(2015\)](#) as a bridge containing non-redundant tensile members, whose failure would likely result in the collapse of the bridge. Increasing structural redundancy can improve bridge performance in the event of a horizontal impact and other unpredictable accidents ([Stark et al. 2016](#)). As noted by [Stark et al. \(2016\)](#), “adding primary load-carrying elements to provide load-path redundancy is complicated and difficult to achieve”, given the limitations of a through-truss structural configuration. Additional secondary elements might be used to enhance redundancy and robustness.

The potential for over-height load strikes is clearly an aspect of bridge performance that should be considered during the design phase, as such incidents occur more frequently than might be expected. One recent survey reported that four over-height load strikes occurred per day in the United States between 2005 and 2008 ([Agrawal et al. 2011](#)). The National Transportation Safety Board's 2014 report ([NTSB 2014](#)) on the I-5 Skagit River Bridge collapse estimates that 90% of the through-truss bridges in Washington State experienced over-height load strikes over a 10-year period ([Stark et al. 2016](#)).

The most important lessons in this case are operational considerations. Stark et al. made several operational recommendations ([Stark et al. 2016](#)), which included:

- “A comprehensive database of actual bridge clearances using minimum clearance should be developed for permitting purposes. The database should include variable heights across the bridge, prior bridge strike data from inspection reports, and changes in vertical clearance with time.”
- “Vertical clearance signage should convey variable clearances across and along the roadway. Variable clearance signage should appear well before the structure to allow lane changes.”
- “A vertical clearance buffer of greater than 76 mm (3 in.) should be used because changes in pavement thickness, weather conditions, and vehicle bounce will occur over time.”
- “The pilot vehicle driver should not use any electronic communication device that is not related to communication with the oversize vehicle driver.”
- “Pilot car height poles should be mounted vertically and be equipped with technology that immediately informs the escort and oversize vehicle drivers that a structural element has been contacted by the height pole, so the escort driver does not have to relay the information.”
- The need for both pilot vehicles and trucking companies to retain appropriate insurance coverage is highlighted by this incident. “In the case of the I-5 Skagit River Bridge incident, the pilot car company was only required to carry \$1 million worth of insurance coverage, while the bridge repair costs [alone] exceeded \$15 million...”. Insurance coverage should also consider the economic loss while the bridge is out of service.

In addition to these specific recommendations, it should be noted that redundancy in the planning of overall transportation corridors is desirable. Provision of two separate bridges over the Skagit River, for example one northbound and one southbound, would have preserved the integrity of the transportation system while repairs were being made to the damaged bridge. Disproportionate loss of an entire transportation corridor should not result from local failure of a single non-redundant structural member in a single bridge.

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