

5-2005

Failure of Cold-Formed Steel Beams during Concrete Placement

Norbert Delatte

Cleveland State University, n.delatte@csuohio.edu

Follow this and additional works at: https://engagedscholarship.csuohio.edu/encee_facpub

 Part of the [Civil Engineering Commons](#), and the [Structural Engineering Commons](#)

[How does access to this work benefit you? Let us know!](#)

Original Citation

Delatte, N. (2005). "Failure of Cold-Formed Steel Beams during Concrete Placement."
J.Perform.Constr.Facil., 19(2), 178-181.

This Article is brought to you for free and open access by the Civil and Environmental Engineering at EngagedScholarship@CSU. It has been accepted for inclusion in Civil and Environmental Engineering Faculty Publications by an authorized administrator of EngagedScholarship@CSU. For more information, please contact library.es@csuohio.edu.

Failure of Cold-Formed Steel Beams during Concrete Placement

Norbert Delatte, M.ASCE¹

Abstract: During a concrete placement on the second story of a building under construction, the supporting cold-formed steel beams collapsed. Four workers were injured. The collapse occurred while concrete was being placed onto steel decking on the second floor of the structure. Cold-formed steel beams, without shoring, supported the steel decking. Analysis of the steel beams under the weight of concrete and workers using the applicable American Concrete Institute and American Iron and Steel Institute documents indicated that the beams were overstressed for construction loads. After the collapse, part of the structure was rebuilt using thicker beams. For the reconstruction, the slab was shored. Designing with cold-formed steel requires knowledge of failure modes that can often be safely ignored with hot-rolled steel, such as local buckling. Engineers designing with this material should take care to obtain the proper codes and design documents.

CE Database subject headings: Cold-formed steel; Failures; Steel beams; Collapse; Construction site accidents.

Introduction

During a concrete placement on the second story of a building under construction, the supporting cold-formed steel beams collapsed. Four workers were injured, with one fracturing his hip. Approximately two-thirds of the deck had been placed. The project structural engineer had been at the site earlier, but had left prior to the collapse.

The collapse occurred while concrete was being placed onto steel decking on the second floor of the structure. The steel decking was supported by 203 mm (8 in.) deep cold-formed 1.21 mm (0.0478 in., 18 gauge) steel beams, without shoring. Some of the workers raised concerns about the safety of the structure with the project structural engineer. He assured the contractor and workers that shoring was unnecessary, and that the beams were rated with more than enough capacity to support the concrete.

The testimony of the workers, and the photographs available, indicate that good construction practices were followed with respect to placing and finishing the concrete. The project structural engineer contended that the failure had been due to workers allowing the concrete to form a pile on the formwork, thus increasing the loading.

This technical note reviews the structure, collapse, available records, structural loads imposed and analysis of the steel beams,

possible failure modes, and the reconstruction. Analysis indicated that the deck beams were not strong enough to carry the imposed loads.

Description of the Structure

The structure had three parallel masonry walls, each nominally 305 mm (1 ft) wide, with continuous cold-formed steel beams crossing the walls. The second level was approximately 7.62 by 28.3 m (25 by 93 ft) in plan. The spans across the masonry walls were 3.35 and 3.96 m (11 and 13 ft) center to center, and the beams were continuous across the two spans. Cold-formed steel beams 610 mm (2 ft) on center supported metal decking, on which the concrete was to be placed. Welded wire mesh reinforcing fabric was also placed on the decking before the concrete.

The steel grade and section properties of the 203 mm (8 in. deep) cold-formed 1.21 mm (0.0478 in., 18 gauge) steel beams were not provided in the documents reviewed by the writer. Therefore, it was necessary to assume section properties based on the available information from the design drawings as well as tables from the American Iron and Steel Institute (AISI) manual (AISI 1997).

In the *Cold-Formed Steel Design Manual* (AISI 1997), the CS designation refers to C-sections with lips. Therefore, the section was assumed to be 8CS1.625×045, which is the only 1.21 mm thick (0.0478 in., 18 gauge) 203 mm (8 in.) deep section listed. The grade of steel supplied was not provided in the documents reviewed. A yield stress of 228 MPa (33 ksi) was used in the computer calculations for the roof truss, which also used cold-formed steel. Therefore, it was most likely that the beams were 228 MPa (33 ksi). However, since this section may be made of steel with a yield point of either 228 or 379 MPa (33 or 55 ksi), both grades of steel were investigated.

One of the workers noted that some of the beams had been

Table 1. Maximum Moments by Loading Case

Case	w_1		w_2		M_N		Distance to M_{\max}		M_{\max}	
	kN m	plf	kN m	plf	kN m	ft kips	m	ft	kN m	ft kips
I	2.92	200	2.92	200	−3.31	−2.45	1.69	5.56	4.20	3.09
II	1.46	100	2.92	200	−2.68	−1.99	1.75	5.73	4.44	3.29
III	0	0	2.92	200	−2.06	−1.53	1.80	5.91	4.72	3.50

Note: plf=pounds per linear foot.

bent earlier as a heavy point load of decking was placed on them. The top flanges of the beams were damaged. They were repaired, straightened, and reused, rather than being replaced. A short length of intact beam was screwed to each straightened beam. These beams may have had local buckling or residual stress from the bending, resulting in reduced load carrying capacity. Damage to the beam compression flange, occurring in the midspan at the point of maximum positive moment, may significantly reduce the bending strength of a beam. The damaged beams may have triggered the progressive collapse.

Collapse

Immediately prior to the collapse, concrete was being placed from a pump onto the decking. An experienced worker was using the pump nozzle to spread the concrete. The workers started at one end, moving toward the other end of the second floor. One worker claimed that the deck was vibrating during the concrete placement.

When approximately two thirds of the concrete had been placed, the decking on the longer 3.96 m (13 ft) span gave way suddenly, and five of the workers fell. Two workers were able to grab wire mesh and avoid falling the entire distance. The others fell onto the first floor. One fell onto a plumbing fixture pipe and broke his pelvis. Photographs taken directly after the collapse showed that the beams were bent at the interior wall support, and at about the midpoint of the longer 3.96 m (13 ft) span. The damaged beams hung downward from the interior wall.

Review of Documents and Depositions

A number of documents and records were obtained and reviewed. Most of the project records and reports were available. The attorneys for the plaintiff and for the defendants obtained depositions from a number of individuals.

The writer was retained by the attorney for the injured workers. Because the building owner was listed as a defendant, it was not possible to arrange a site visit during the preliminary analysis. Instead, the writer was asked to prepare an analysis based on the available documents and records. Those included the depositions, photos taken before and after the collapse, and the building plans. Following the initial analysis, a site visit was to be arranged. However, the case was settled before trial.

Loads during Concrete Placement

The loads to be considered for concrete placement may be found in a number of sources. These include the Standard Building Code (Standard Building Code 1997), as well as the American

Concrete Institute (ACI) publication *Formwork for Concrete* (Hurd 1995).

Using these sources and the 100 mm (4 in.) thick deck slab shown on the plans, 31.4 kN per linear meter (200 pounds per linear foot) of combined dead and live load, concrete and workers, was used for the calculations. The Standard Building Code Chapter 16 does not specifically address live load for workers and equipment on a deck, except that all live loads should be distributed so as to cause “maximum effect,” in Section 1604.4, page 215 (Standard Building Code 1997). To cause maximum effect, they may be placed on one span or both.

Analysis of Beam Moments and Structural Capacity

In order to determine the bending moments in the two-span continuous beam, the slope-deflections equations were used. Three load cases were considered:

- Case I: Concrete and live load spread uniformly across both spans. This uniform load distribution would be impossible to achieve at all times during construction—concrete cannot be simultaneously placed across the entire 7.32 m (24 ft length) of a beam from a single pump nozzle.
- Case II: Concrete spread uniformly across both spans, live load on longer (3.96 m, 13 ft) span only. This load combination is commonly used for design, with dead load assumed along the full length of the beam, and live load only where it causes the greatest effect, in accordance with the Standard Building Code. However, as noted earlier, it is not possible to place concrete in this way.
- Case III: Concrete and live load spread uniformly across the longer 3.96 m (13 ft) span only. This would be a prudent load combination for the designer to consider, because the concrete must be placed on one area of the beam, and the workers need to be where the concrete is to work with it. This is also the only load combination that considers the “unbalanced loads” referred to in Chapter 5 of the American Concrete Institute publication *Formwork for Concrete* (Hurd 1995).

Bending moments for these three load cases are shown in Table 1, and moment diagrams are shown in Fig. 1. The greatest absolute value of the moment is always the positive moment near the midspan of the longer beam, for the load cases considered.

Therefore, the bending failure in the beams would occur near the center of the 3.96 m (13 ft) span, and at the support between the 3.96 and 3.35 m (13 and 11 ft) spans. This agrees with the damage shown in the photographs that accompanied the various depositions.

Possible Failure Modes

According to Chapter 4 of *Cold-Formed Steel Design*, (Yu 1991) C-sections used as beams can fail through bending, shear, com-

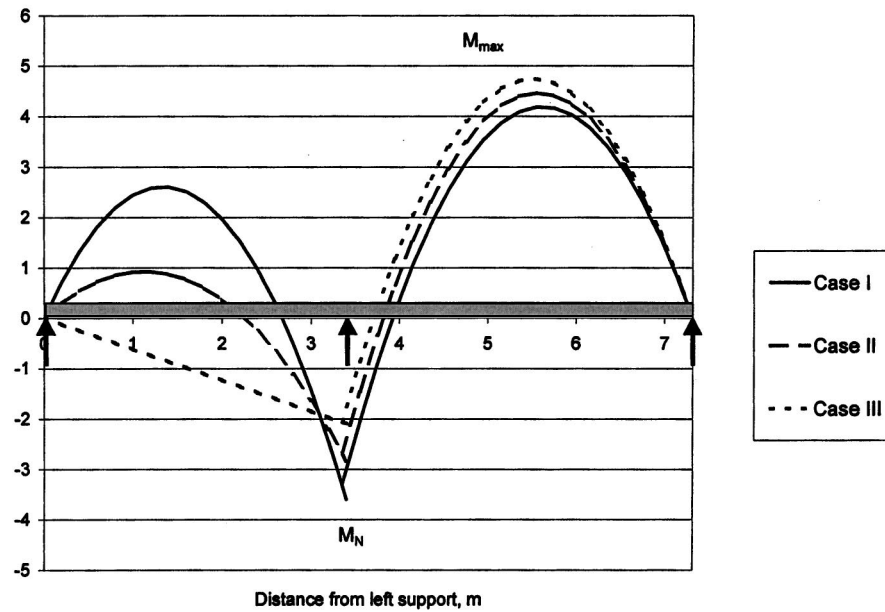


Fig. 1. Moment diagrams for cold-formed steel support beams

bined bending and shear, lateral-torsional buckling, and web crippling. The centroid and the shear center of such a singly symmetric section do not coincide, leading to torsion stresses.

The notation for the engineer's computer calculations for the roof truss indicated the use of the American Institute of Steel Construction (AISC) 2nd Edition Load and Resistance Factor Design Code (AISC 1998). This specification is only applicable to hot-rolled steel sections, not to cold-formed steel.

For bending, the nominal moment capacity in the 8CS1.625 \times 045 section is 4.27 kN m (3.16 ft kips) for 228 MPa (33 ksi) steel, and 5.49 kN m (4.07 ft kips) for 379 MPa (55 ksi) steel [Table II-1, page II-3, *Cold-Formed Steel Design Manual* (AISI 1997)]. These capacities are shown in Table 2. These are based on the effective section modulus, adjusted for local buckling of the beam compression flange.

For allowable stress design, these nominal capacities must be reduced by the appropriate factor of safety. For bending, the required factor of safety (Ω_b) is 1.67 [C3.1.1, p. V-45, *Cold-Formed Steel Design Manual* (AISI 1997)]. Therefore, the allowable moment is 2.56 kN m (1.89 ft kips) for 228 MPa (33 ksi) steel and 3.29 kN m (2.44 ft kips) for 379 MPa (55 ksi) steel.

Under all loading conditions considered above, the moment in positive bending exceeds the allowable moment regardless of the grade of steel used. In fact, for 228 MPa (33 ksi) steel, the moment exceeds the nominal moment for beams 1.21 mm thick (0.0478 in. thick, 18 gauge). Actual factors of safety are shown in the last column of Table 2. Factors of safety less than 1 indicate that the design loads exceed design capacity, with a risk of failure under service load conditions.

Because cold-formed steel sections are not universally standardized (p. 24, Yu 1991), it is possible that the section used had a larger flange than the 41.3 mm (1.625 in.) assumed. Therefore, section properties for a channel with a 63.5 mm (2.5 in. flange) were calculated and are shown in Table 2. Thus, even with a larger flange, the 203 mm deep 1.21 mm thick (8 in. deep, 0.0478 in. thick, 18 gauge) beam would be overstressed with 228 MPa (33 ksi) steel for all load cases, and for 379 MPa (55 ksi) steel and unbalanced concrete and live load.

For other construction materials, such as hot-rolled steel and reinforced concrete, multiple span beams have reserve capacity due to the formation of plastic hinges. However, cold-formed steel sections such as those investigated in this paper are not

Table 2. Beam Cross Section Moment Capacities

Metal [thickness, mm(in., gauge)]	Flange width [mm (in.)]	Steel yield strength [MPa (ksi)]	Nominal moment capacity [kN m(ft kips)]	Factor of safety (moment capacity/ M_{max})
1.21 mm (0.0478 in., 18 gauge)	41.3 (1.625)	228 (33)	4.27(3.16) ^a	0.90
		379 (55)	5.49(4.07) ^a	1.16
	63.5 (2.5)	228 (33)	4.19(3.10)	0.89
		379 (55)	6.98(5.17)	1.48
1.52 mm (0.0598 in., 16 gauge)	41.3 (1.625)	228 (33)	5.32(3.94) ^a	1.13
		379 (55)	8.48(6.28) ^a	1.80
	63.5 (2.5)	228 (33)	5.82(4.31)	1.23
		379 (55)	9.71(7.19)	2.06

^aFrom Table II-I (AISI 1997).

compact and may have local buckling, and cannot be relied on to form plastic hinges (Yu 1991). Failure of the system occurs when any part of the beam is overstressed.

All of these stresses are due to bending only. Torsional stresses, which were not calculated, would add to the bending stresses. The other potential failure modes, e.g., shear and web crippling, were not analyzed because the photographs of the collapsed structure strongly suggested bending failure.

Reconstruction

Following cleanup, the slab decking was rebuilt using 1.52 mm (0.0598 in., 16 gauge) steel beams to replace the damaged thinner beams. Beams that had not been damaged were not replaced. This time, the beams were shored. The concrete placement occurred without mishap. The building was completed and put into service.

Discussion

The structural integrity of the beams and decking was questioned, but the structural engineer provided assurances that they were adequate. No supporting documents were available.

The structural engineer contended that the collapse was due to poor construction practices leading to concrete piling up and causing unbalanced loading. However, the testimony of the workers indicated that the concrete placement was carried out in accordance with good practice. There was no testimony from the workers or observers present that the concrete was allowed to pile up at any point on the decking. In fact, this would have made screeding and finishing the concrete much more difficult.

It is well known that the structural integrity of formwork for concrete is important. Hanna writes on page 6 of *Concrete Formwork Systems*, "partial or total failure of concrete formwork is a major contributor to deaths, injuries, and property damages within the construction industry" (Hanna 1999).

The writer's investigation was hampered because it was not possible to access the site, and because the failed structure had been removed before the investigation started. Unfortunately, material samples had not been retained. Therefore, it was necessary to analyze the failure solely from the available documents and records.

Summary and Conclusions

There was only one engineer qualified by training, experience, and professional licensure on this project. The structural engineer should consult the proper references and perform the necessary structural calculations to ensure that the structure will be safe against collapse, under the load combinations prescribed by building codes. He should have analyzed the beam under an unbalanced load of concrete and live load, and compared the calculated moments to the section capacities provided in the AISI Manual (AISI 1997).

There are important differences between design procedures for hot-rolled structural steel, which are taught in most civil engineering undergraduate programs, and those for cold-formed steel. Designing with cold-formed steel requires a knowledge of failure modes that can often be safely ignored with hot-rolled steel, such as local buckling. Engineers designing with this material should take care to obtain the proper codes and design documents.

Acknowledgment

R. S. Prabhu provided important information about the properties of cold-formed steel sections to the writer, and his assistance is gratefully acknowledged.

References

- American Iron and Steel Institute (AISI). (1997). *Cold-formed steel design manual*, 1996 Ed., American Iron and Steel Institute, Chicago, with errata.
- American Institute of Steel Construction (AISC). (1998). *Manual of steel construction, load and resistance factor design*, 2nd Ed., Chicago.
- Hanna, A. (1999). *Concrete formwork systems*, Marcel Dekker, New York.
- Hurd, M. K. (1995). "Formwork for concrete." *Special Publication No. 4*, American Concrete Institute Committee 347, 6th Ed., ACI, Farmington Hills, Mich.
- Southern Building Code Congress (SBC). (1997). *Standard building code*, Birmingham, Ala.
- Yu, W.-W. (1991). *Cold-formed steel design*, 2nd Ed., Wiley, New York.