Anatomy of a Collapse

Portions of the original design of the indoor practice facility, which collapsed in May 2009, injuring a dozen people, did not meet accepted codes and standards.

ASSOCIATED PRESS

In May 2009, a practice facility used by the Dallas Cowboys collapsed during a storm that had wind speeds less severe than the nominal wind speed for which the structure was designed. An investigation of the collapse detailed several design errors that probably led to the failure of the large, fabric-covered steel structure.

By Christopher Trautner, P.E., S.M.ASCE, Rasko Ojdrovic, Ph.D., P.E., M.ASCE, Benjamin W. Schafer, Ph.D., P.E., M.ASCE, and Nicholas P. Jones, Ph.D., M.ASCE

IN MAY 2009 A LARGE, fabric-covered steel truss structure used as a practice facility by the National Football League's Dallas Cowboys collapsed, injuring a dozen people, two of them very seriously. Although news coverage of the event was quick to name high winds as the cause of the failure, several design errors precipitated the failure in winds far less severe than the nominal design wind speed. This article summarizes the results of a failure investigation we conducted, including a comparison of the wind loads used in the original design with the provisions set forth in ASCE's standard 7 (*Minimum Design Loads for Buildings and Other Structures*). The article also presents an analysis of the strength of the steel trusses and the results yielded by a three-dimensional (3-D) finite-element model of the structure, including the outer fabric covering. Portions of the original design that did not meet accepted codes and standards are highlighted and discussed.

On the basis of these results and photographs of the collapsed structure, a likely initial failure mechanism and progressive collapse sequence are proposed. The results of the analysis and proposed failure sequence are compared with the results of an investigation performed by the National Institute of Standards and Technology (NIST), which released its findings in January 2010. On the basis of the design errors found by both investigations, recommendations are made for the design of lightweight fabric-covered structures.

Located in Irving, Texas, the practice facility in question was approximately 204 by 406 ft in plan. Although described by some reports as a tent structure, the facility's main vertical and lateral structural system comprised a series of steel trusses. These trusses were made up of several prefabricated segments and had been assembled in the field. The trusses were spaced 15 ft on center along the structure's length and were supported on caissons connected by grade beams. Cold-formed hollow structural sections (HSSs) with a circular cross section 5 in. in diameter formed the trusses'

0885-7024/13-0004-0066/$30.00 PER ARTICLE

chord members, while hot-rolled, 3 by 3 in. single and double angles formed the web members. In the direction perpendicular to the ridge (the highest point of the facility's roof), the trusses provided lateral resistance; in the direction parallel to the ridge, a system of purlins and sway cables arranged in an X pattern provided lateral resistance.

The trusses were sandwiched between two layers of fabric that were connected to attachments screwed onto the truss chord members. The inner fabric was relatively lightweight (1,000-denier) and was made of polyvinyl chloride, while the outer fabric was a relatively heavy scrim made of polyolefin and high-density polyethylene coated with low-density polyethylene. Both layers of fabric were pretensioned in the direction of the short axis of the building by a series of straps connected to baseplates at the bottom of each of the trusses.

Designed and built in mid-2003, the facility was investigated in 2007 to address owner concerns about its reliability and durability. On the basis of that review, a retrofit was designed and implemented between 2007 and 2008. The structure collapsed during a storm on May 2, 2009.

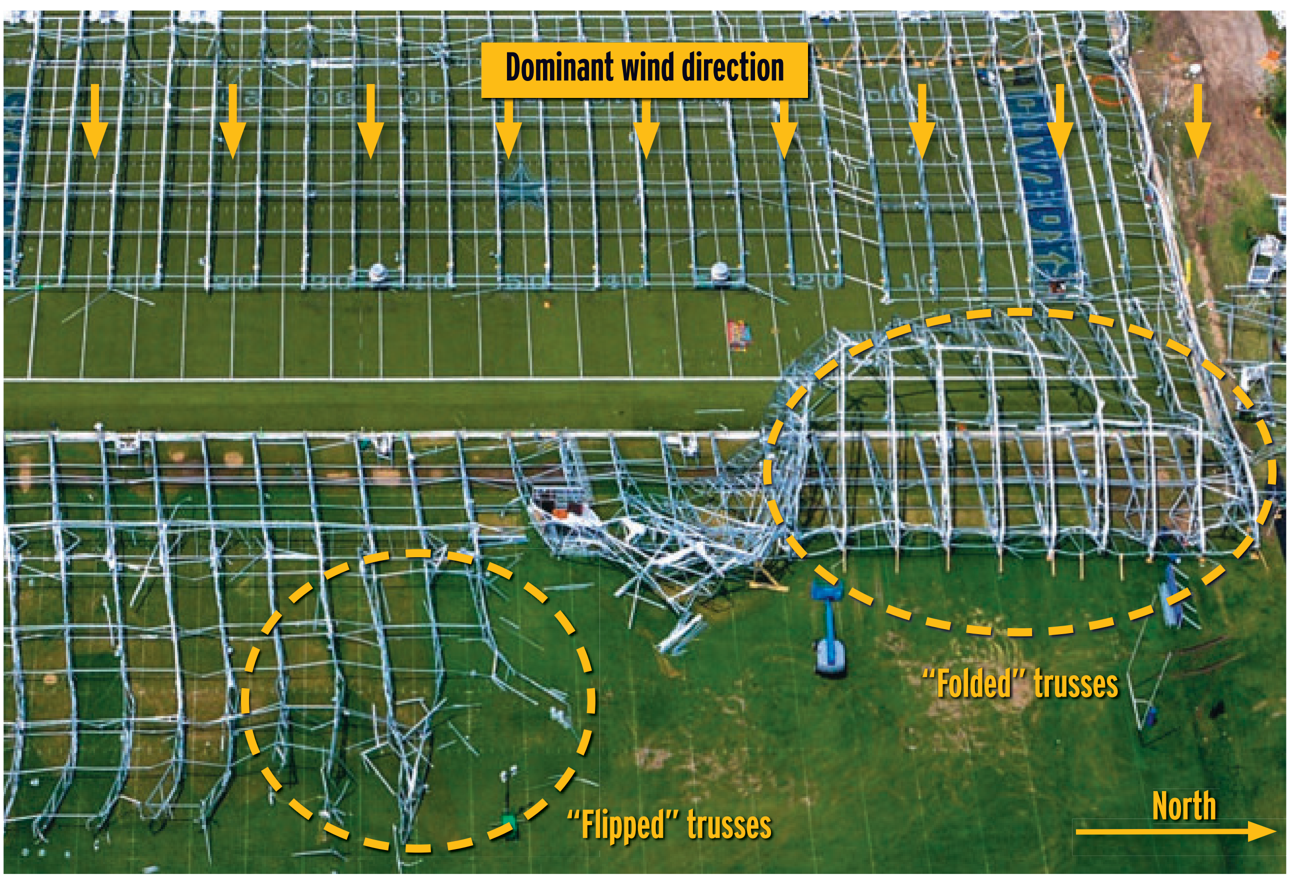
Our involvement in the project began in October 2009, when we were hired by a lawyer representing the parties injured in the collapse. Therefore, we did not observe the collapsed facility until after the debris had been moved from its original location. However, many photographs had been taken of the collapsed facility by firsthand observers, and a news crew filming a practice session captured some of the collapse sequence from inside the building. We reviewed these photographs and videos and the saved debris remnants to determine the likely collapse sequence and possible initiating failures.

The dominant wind direction on the day of the collapse was from the west, and the overall east–west collapse pattern strongly suggests that collapse was initiated by failure of the east–west lateral resisting system—that is, the main truss elements.

Figure 1 is an aerial photograph of the north end of the facility after the fabric coverings had been removed. The dominant wind direction on the day of the collapse was from the west, and the overall east–west collapse pattern strongly suggests that collapse was initiated by failure of the east–west lateral resisting system—that is, the main truss elements. Further examination of the photograph shows two distinct patterns of failure: trusses that have been broken at the ridge and "flipped" outside the plan of the structure and trusses that have been pushed over but have folded into the plan of the structure.

This pattern suggests one of two collapse sequences. The first is that collapse began with failure of a truss at the ridge, which made it possible for the wind to catch the inside of the

FIGURE 1
Aerial View of North End of Dallas Cowboys Practice Facility



NELSON ARCHITECTURAL ENGINEERS, INC

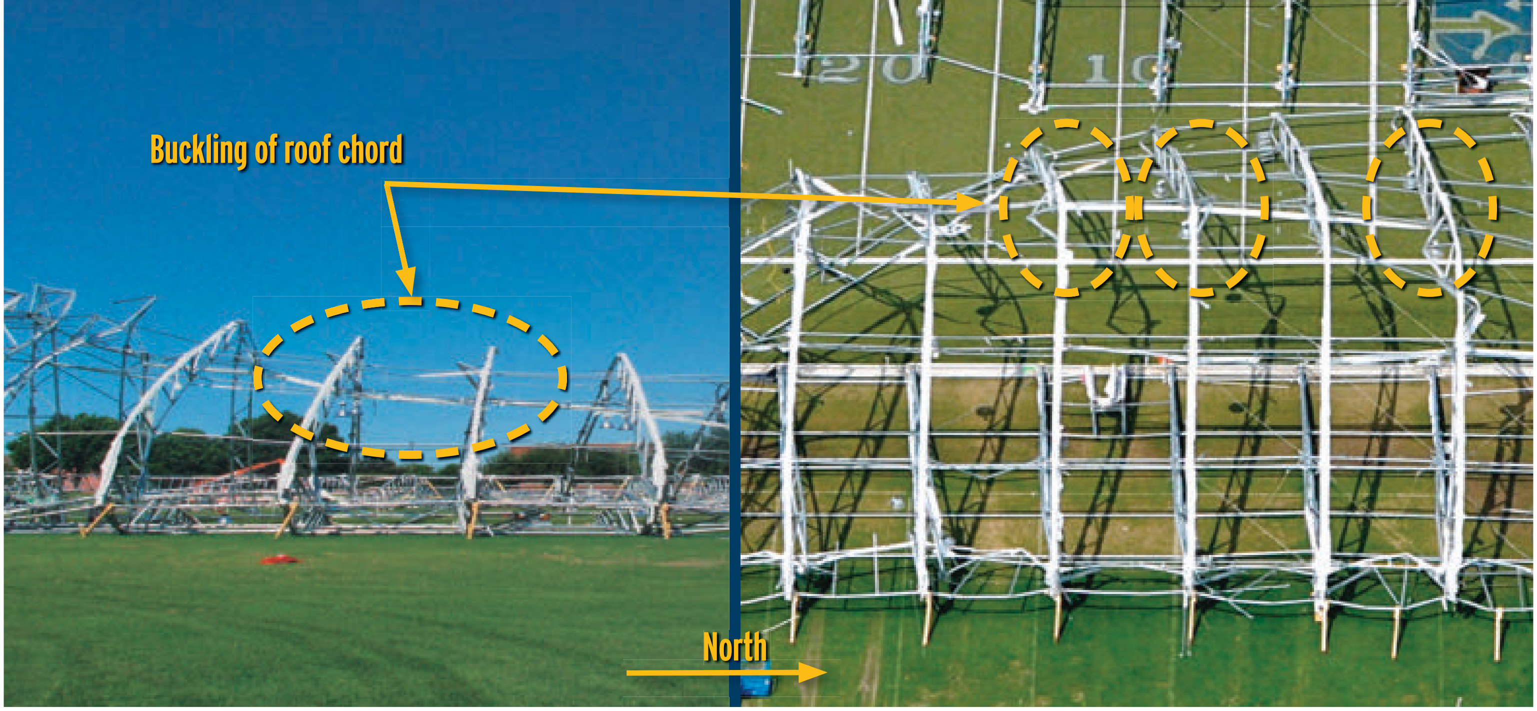


FIGURE 2
BUCKLED ROOF CHORDS AT NORTH END OF DALLAS COWBOYS PRACTICE FACILITY

adjacent trusses and sequentially flip them outside the structure. The second possible sequence is that wind caused the failure of one of the folded trusses near the north end of the structure and that this failure progressively overloaded adjacent trusses, causing them to fold inward. Eventually, a large enough hole was opened in the structure for the wind to catch the inside surface of the structure and flip the remaining trusses outside.

The second possible sequence was deemed more likely. This conclusion was based on the notion that, if failure had started at the ridge and the wind had flipped trusses outside the structure, it would have been unlikely for this sequence to stop abruptly near the north end. Meanwhile, no strong driving force would exist to fold the remaining trusses in on themselves.

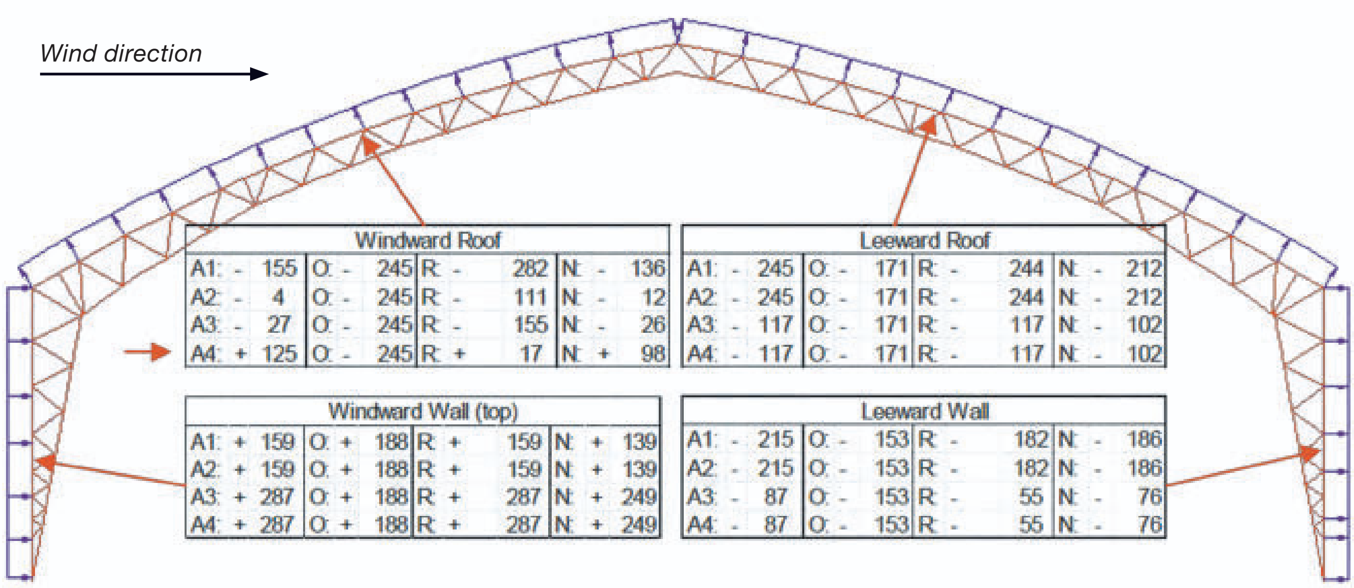
The postulated failure sequence is further strengthened by closer examination of the folded trusses. As shown in figure 2, a buckled inner roof chord is clearly visible in at least two of the trusses. This type of failure is consistent with initiation of failure at the folded trusses.

Buckling of this chord as the initiating event is consistent with the overall collapse sequence as well as with the suddenness of the failure, which was evidenced by the video. As discussed below, an evaluation of the structure's design supports the theory that this failure was a likely initiating cause.

The original design of the practice facility was performed using load and resistance factor design (LRFD) based on loads given in the 1998 edition of ASCE 7 and member design strengths calculated using the 1999 standard *Load and Resistance Factor Design Specification for Structural Steel Buildings* and the 2000 standard *Load and Resistance Factor Design Specification for Steel Hollow Structural Sections*, both published by the American Institute of Steel Construction, which has its

FIGURE 3
MAIN WIND-FORCE-RESISTING SYSTEM DESIGN PRESSURES

SIMPSON GUMPERTZ & HEGER, INC., *BOTTOM;*
NELSON ARCHITECTURAL ENGINEERS, INC, *TOP TWO*



Windward Roof											
A1:	-	155	O: -	245	R:	-	282	N:	-	136
A2:	-	4	O: -	245	R:	-	111	N:	-	12
A3:	-	27	O: -	245	R:	-	155	N:	-	26
A4:	+	125	O: -	245	R:	+	17	N:	+	98

Leeward Roof										
A1:	-	245	O: -	171	R:	-	244	N:	-	212
A2:	-	245	O: -	171	R:	-	244	N:	-	212
A3:	-	117	O: -	171	R:	-	117	N:	-	102
A4:	-	117	O: -	171	R:	-	117	N:	-	102

Windward Wall (top)										
A1:	+	159	O: +	188	R:	+	159	N:	+	139
A2:	+	159	O: +	188	R:	+	159	N:	+	139
A3:	+	287	O: +	188	R:	+	287	N:	+	249
A4:	+	287	O: +	188	R:	+	287	N:	+	249

Leeward Wall										
A1:	-	215	O: -	153	R:	-	182	N:	-	186
A2:	-	215	O: -	153	R:	-	182	N:	-	186
A3:	-	87	O: -	153	R:	-	55	N:	-	76
A4:	-	87	O: -	153	R:	-	55	N:	-	76

During the evaluation, two features of the design and subsequent retrofit were determined to be inconsistent with contemporary codes and standards: the application of inappropriate wind load provisions and the use of an incorrect effective length factor for the chord members of the truss.

headquarters in Chicago. However, the subsequent retrofit used the 2005 edition of ASCE 7 to calculate design loads. A review of the original design and retrofit was conducted primarily to determine whether the facility, as originally designed, met the applicable codes and standards. The analyses performed were used to uncover factors that may have led to the collapse and to evaluate the evidence for the postulated collapse sequence. Given the meteorological conditions at the time of the failure, wind loads and truss chord member capacities were the focus of primary concern. During the evaluation, two features of the design and subsequent retrofit were determined to be inconsistent with contemporary codes and standards: the application of inappropriate wind load provisions and the use of an incorrect effective length factor for the chord members of the truss.

The first major inconsistency found was the application of several of the wind load provisions of the 1998 edition of ASCE 7 to the main wind-force-resisting system (MWFRS) in the east–west direction, that is, the main roof trusses. The structure was designed using the "low-rise" provisions in subsection 6.5.12.2.2 of ASCE 7, even though the mean roof height of 66 ft exceeded the allowable height of 60 ft for classification as a low-rise building. Although the structure was described as a "fully enclosed" building in the design calculations, the internal pressures stipulated in table 6-7 of ASCE 7 for enclosed low-rise buildings were not applied to the net MWFRS design pressures. Furthermore, all trusses were designed for identical MWFRS design pressures. Consequently, zones of higher pressure near the ends of the structure were not considered in relation to the required load cases, which are shown graphically in figure 6-4 in ASCE 7.

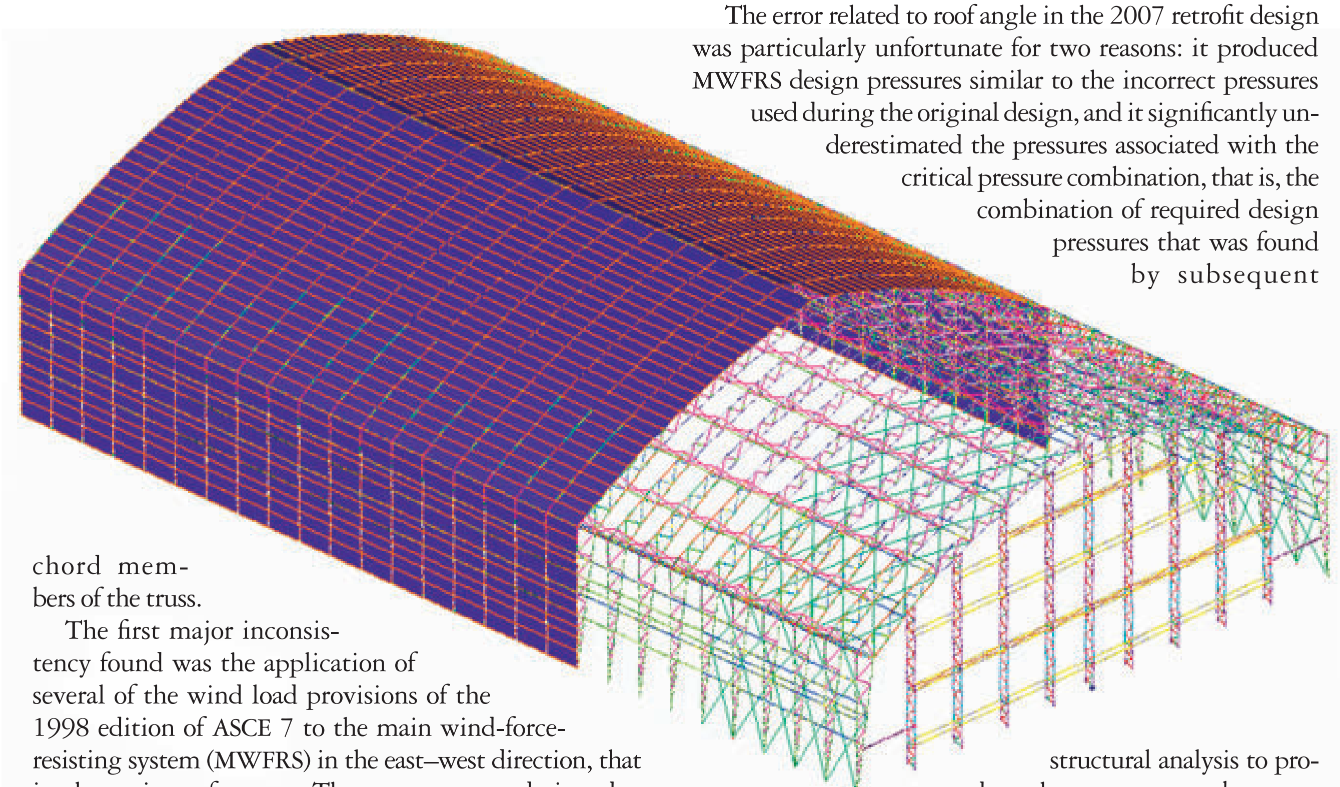


FIGURE 4
THREE-DIMENSIONAL MODEL OF DALLAS COWBOYS PRACTICE FACILITY

SIMPSON GUMPERTZ & HEGER, INC

The design of the structure's retrofit addressed many of the shortcomings of the original design with regard to proper application of the MWFRS design pressures given in ASCE 7. However, the retrofit designer's calculation of the average roof angle mistakenly used the full width of the building instead of half the width. As a result, the angle used to calculate the pressure coefficients (11 degrees) was approximately half the correct angle (21 degrees).

The table on page 70 summarizes the criteria that were used in the original design and the 2007 retrofit related to the design for wind in the direction perpendicular to the ridge, as well as the criteria that NIST and we separately determined should have been used. Criteria inconsistent with ASCE 7 are shaded in the table.

The error related to roof angle in the 2007 retrofit design was particularly unfortunate for two reasons: it produced MWFRS design pressures similar to the incorrect pressures used during the original design, and it significantly underestimated the pressures associated with the critical pressure combination, that is, the combination of required design pressures that was found by subsequent structural analysis to produce the greatest member overloads. The wind pressures used for evaluating the structure in the direction perpendicular to the ridge are shown in figure 3.

The combination highlighted by the short arrow to the left in figure 3 is particularly significant because of the relatively high positive pressure required by ASCE 7, in contrast to the high negative pressure of the original design and the low positive pressure used by the retrofit design. Application of positive pressure to the windward roof tends to develop high compressive axial forces in the leeward inner chord member near the midpoint between

Design Criteria for Main Wind-Force-Resisting System

Factors affecting wind load	Authors	National Institute of Standards and Technology	Original designer	Retrofit designer	Notes
Environmental parameters:					
Basic wind speed	90 mph	90 mph	90 mph	90 mph	Figure 6.1a in 1998 edition of ASCE 7
Exposure category	C	C	C	C	Subsection 6.5.6 in 1998 edition of ASCE 7
Directionality factor	0.85	0.85	0.85	0.85	Table 6-6 in 1998 edition of ASCE 7
Topographic factor	1.0	1.0	1.0	1.0	Subsection 6.5.7 in 1998 edition of ASCE 7
Structural parameters:					
Occupancy category	III	II	IV	III	Classifications III and IV have the same importance factor (1.15)*.
Flexibility category	Rigid	Rigid	Rigid	Rigid	Fundamental frequency is about 1.4 Hz (cutoff being 1 Hz); categorization as rigid is conservative.
Height classification (roof angle)	All-heights (21 degrees)	All-heights (21 degrees)	Low-rise (21 degrees)	All-heights (11 degrees)	Mean roof height is about 66 ft (greater than the 60 ft cutoff for low-rise structures).
Internal pressure consideration	Considered	Considered	Not considered	Considered	Positive and negative internal pressures must be considered for all building types (subsection 6.5.12.2.1 or 6.5.12.2.2 in 1998 edition of ASCE 7).
Building end zones**	Considered	Considered	Not considered	Considered	Consideration of end zones is required (figure 6-4 in 1998 edition of ASCE 7).

**According to table 1-1 in the 1998 edition of ASCE 7, category II (importance factor = 1.0) might also be appropriate.*
***Not applicable to all-heights procedure.*

the ridge and the knee. By contrast, this section is not highly stressed under negative pressure, as envisioned in the original design. High compressive forces in this particular segment of the truss are consistent with the postulated failure sequence, as discussed below.

The second major design feature inconsistent with contemporary codes and standards was the application of the effective length factor, *K,* for the chord members of the truss. Both the original design and the retrofit assumed an effective length factor of 0.5, consistent with fully fixed-end conditions. Section 4.1 of the 2000 standard *Load and Resistance Factor Design Specification for Steel Hollow Structural Sections* stipulated a minimum *K* value of 0.9 for HSS truss chords connected to non-HSS web members. However, a *K* value of 1.0 would be consistent with the typical pinned-end truss behavior assumed in basic structural analysis. No justification was provided for designing the chord members on the assumption that fixed-end conditions obtained. As a consequence, the chord design capacities used in both the original and the retrofit design were significantly higher than those allowed by the HSS specification.

Another issue related to the compression capacity of the chord members was the assumed role played by fabric in bracing the chord members against buckling out of the plane of the truss. Both the original design and the retrofit assumed that both the inner and the outer fabric completely prevented failure caused by buckling out of the plane of the truss. However, no analysis was performed to justify this assumption.

Both the original design and the retrofit assumed that both the inner and the outer fabric completely prevented failure caused by buckling out of the plane of the truss. However, no analysis was performed to justify this assumption.

Support of an axially loaded column by a fabric or other flexible membrane may increase buckling resistance in the plane of the membrane but will not prevent it entirely. To increase the resistance, the fabric must have sufficient strength to resist tearing before the onset of buckling. The fabric must also have sufficient stiffness to significantly decrease the tendency of the column to buckle in the plane of the membrane. However, even if the fabric possessed sufficient stiffness and strength, the following factors might preclude the fabric from increasing the column's buckling capacity:

• Inadequate fabric pretension could cause a lateral gap that would make it possible for the column to buckle before the engagement of the fabric.

• Structural deformation could result in slack fabric, again leading to a gap in which buckling resistance would not be provided by the fabric.

The good correlation between the wind speed required for the onset of structural failure and estimated wind speeds at the site further supports the proposed failure sequence.

• The fabric may be damaged by wind-borne debris.

Given the uncertainties associated with relying on the fabric, our design evaluation considered the possibility that the truss would buckle out of plane without increased resistance from the fabric. The unbraced length of the chord members is significantly larger out of the plane of the truss than between the points at which the purlins and cables are attached. Therefore, calculated member capacities were significantly below those calculated in both the original and the retrofit design.

Member demands were calculated using the wind pressures described previously by means of a two-dimensional model of a single truss and a 3-D finite-element model of the full structure, the latter shown in figure 4. The 3-D model included a representation of the outer fabric, which was idealized as a linear elastic membrane.

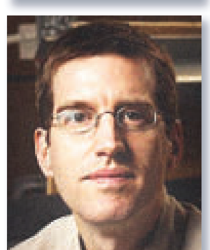
Under the assumption that the fabric provides no lateral bracing, the highest unfactored demand/capacity ratios at the nominal design wind velocity of 90 mph were found to be 2.99 at the inner chord at the truss knee, 2.77 at the outer chord of the windward roof, and 2.74 at the inner chord of the leeward roof. Given the similarity of these ratios, any of these locations could be regarded as possible initial failure sites. However, a geometrically nonlinear analysis of the 3-D structure indicates that the onset of buckling began at the inner chord of the leeward roof.

Both the high calculated demand/capacity ratios and the calculated buckling behavior of the inner chord strongly support the postulated collapse sequence discussed previously. Assuming that demands scale by the square of the velocity, which is consistent with linear structural analysis and the design pressure equations of ASCE 7, these demand/capacity ratios are consistent with onset of failure at about 55 mph. In its review of meteorological data, NIST estimated that the maximum speed of the wind gusts was in the range of 55 to 65 mph on the day of the collapse. The good correlation between the wind speed required for the onset of structural failure and estimated wind speeds at the site further supports the proposed failure sequence.

The design review presented here strongly suggests that the failure of the Dallas Cowboys practice facility was caused by several design errors, including the improper application of the wind load provisions of ASCE 7, the improper use of effective length factors, and assumptions that were hardly conservative regarding the role of fabric as bracing for compression members. Failure of the structure was probably initiated by buckling of the inner chord on the leeward roof of the structure, which was followed by overload and similar failure of adjacent bays until failure of the outer and inner fabric made it possible for the wind to catch and flip the remaining structure outward. Based on unfactored demand/capacity ratios calculated, failure would be expected at a wind speed of approximately 55 mph. The findings of this investigation were generally consistent with NIST's findings.

Trautner

Ojdrovic



Schafer

Jones

On the basis of the evaluation of the design of the practice facility and its subsequent collapse, we make three general recommendations regarding the design of similar structures. First, standard design practice should include careful checking of all design calculations, regardless of how basic the calculations may seem. In the design of the retrofit of the practice facility, a very simple arithmetic mistake related to the geometry of the structure resulted in wind pressures that significantly departed from the requirements set forth in ASCE 7, resulting in very serious consequences.

Second, the use of fabric should be carefully evaluated if the material is regarded as a participating element in the structural system. As with any bracing element, consideration should be given to the stiffness and strength of the fabric. Fabric presents additional challenges, including the potential for relatively rapid degradation and the possibility of loss of prestress, both of which could impair its effectiveness in a structural system. Evaluation of the stiffness, strength, durability, and suitability of the fabric as a bracing element under design load considerations should be performed up front.

Finally, the rather large difference between the low-rise and all-heights wind loads calculated for the design of the practice facility in accordance with the 1998 and 2005 editions of ASCE 7 indicates that buildings slightly more than or slightly less than the 60 ft height limit may be subject to significantly different member design forces. Current code provisions should be reviewed to examine this apparent discontinuity in design provisions. **CE**

Christopher Trautner, P.E., S.M.ASCE, was a member of the staff at Simpson Gumpertz & Heger, Inc., which has its headquarters in Waltham, Massachusetts; he is currently a doctoral student at the University of California at San Diego. Rasko Ojdrovic, Ph.D., P.E., M.ASCE, is a senior principal of Simpson Gumpertz & Heger. Benjamin W. Schafer, Ph.D., P.E., M.ASCE, is a professor at Johns Hopkins University and the chair of the school's civil engineering department. Nicholas P. Jones, Ph.D., M.ASCE, is the Benjamin T. Rome Dean of Johns Hopkins's Whiting School of Engineering. This article is based on a paper the authors presented at ASCE's 6th Congress on Forensic Engineering, which was held last fall in San Francisco.