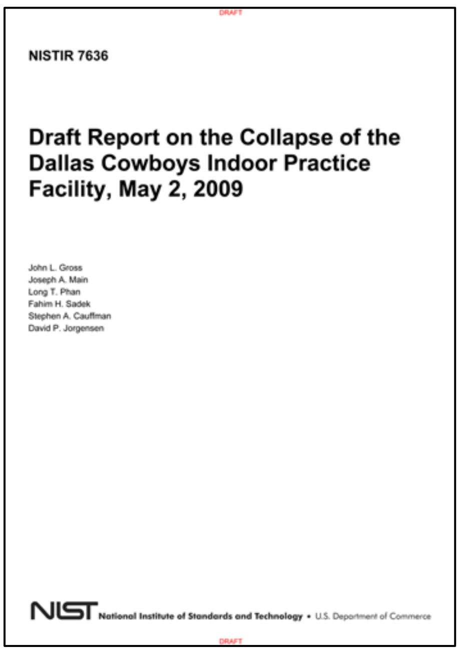
Public Comments Received by NIST on Draft Report:

Report on the Collapse of the Dallas Cowboys Indoor Practice Facility, May 2, 2009, (NISTIR 7636)
DRAFTS FOR PUBLIC COMMENTS

October 2009


DRAFT

NISTIR 7636

Draft Report on the Collapse of the Dallas Cowboys Indoor Practice Facility, May 2, 2009

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From: Mikush David [david.mikush@xlgroup.com]

Sent: Tuesday, October 06, 2009 2:57 PM

To: structuralsafety@nist.gov

Subject: Was ground roughness "C" used in the design?

I don't know the topography in the area, but 95% of the time, the presence of open fields, parking lots, lakes, etc. warrant the use of ground roughness "C".

Dave Mikush

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From: David Campbell [dmc@geigerengineers.com]

Sent: Monday, October 19, 2009 10:19 AM

To: structuralsafety@nist.gov

Subject: Comments on Draft NIST Report on Cowboys Practice Facility Collapse

Report Committiee:

I read your draft report with great interest. I have the following comments regarding the tensile membrane:

1. Irrespective of the reliability of the tensile membrane, the end frames' top chords are not braced by the intact roof membrane and are subject to lateral loads from the membrane. This does not appear to be considered in the draft report.

2. In general, as noted in the draft report, it is inappropriate to rely on tensile membrane to brace the chord members of frames in structures such as this . This is a consequence of the relatively (in relation to tensile strength) low tear strength of most composite textiles employed as tensile membranes as well as their vulnerability to tears initiated by wind-borne debris.

3. In the event of a tear in the tensile membrane, it is also necessary to consider the changed nature of the demand on the structure local to the tear. In the subject structure it would be likely for a tear to propagate sufficiently to result in frame outside (top) chord to be subject to lateral loads from the tensile membrane of the panel opposite the tear. (Note in the subject structure, under significant surface loading the membrane will act primarily one way, spanning between the frames. In the case of outward surface pressures the local curvature of the tensile membrane will reverse.) It is good design practice to consider the demand from load conditions where a membrane tear has occurred. Such consideration for wind conditions would necessarily address the implications of the likely extent of a tear on the enclosure's internal pressure. This is not industry practice nor is it expressly addressed in the current ASCE draft standard.

4. The subject structure likely used a pvc-coated polyester or nylon fabric membrane, possibly laminated. (If the membrane type and properties were discussed the draft report, I did not find it.)

Dependent upon the type of material, the top finish, and formulation of the coating, the anticipated service life of the membrane should be expected to be in a nominal range of 5 to 15 years. Generally, for these materials strength properties degrade over the service life, primarily due to UV exposure. For the types of fabric membranes utilized in such structures, tear strength degrades more quickly than tensile strength. This brings to light the importance of the points 2 and 3 above, as even if the tensile membrane would be seemingly have enough initial tear resistance to reliably brace steel members, it most likely will not as it approaches the end of its service life. As the intended service life of the steel structure is likely intended to be much greater than that of the tensile membrane, it is clearly poor practice to rely on the tensile membrane as an essential component of the primary structure.

Please do not hesitate to contact me should you have any questions.

Best Regards,

David M. Campbell P.E.

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From: David Nickerson [dnickerson@rubbusa.com]

Sent: Friday, November 06, 2009 10:37 AM

To: structuralsafety@nist.gov

Subject: Comments on NIST Draft Report on Dallas Cowboys Collapse

Dear Sir/Madam:

Please find enclosed an electronic copy of my comments on the NIST draft report on the Dallas Cowboys practice facility collapse. Please direct these to the attention of Stephen Cauffman.

I have also enclosed some pages of trial testimony related to the collapse of a Summit building in Philadelphia in 2003.

If there are any questions on the enclosed, please direct them to me at 207-324-2877 or via e-mail.

Thanks for your consideration of the enclosed.

Sincerely,

Dave Nickerson

We Cover The World®
RUBB
BUILDING SYSTEMS

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November 5, 2009

Mr. Stephen Cauffman
National Institute of Standards and Technology
U.S. Department of Commerce
100 Bureau Drive, Stop 8611
Gaithersburg, MD 20899-8611

Via e-mail: structuralsafety@nist.gov

Reference: Draft Report on Cowboys Practice Facility Collapse

Dear Mr. Cauffman:

I am writing to you with regard to the draft report issued by NIST on the May 2, 2009 collapse of a fabric-covered, steel frame facility produced by Summit Structures, LLC, a division of Cover-All Building Systems of Saskatchewan, Canada. In full disclosure, Summit is a competitor to our company and we were an unsuccessful bidder on the Dallas Cowboys practice facility project.

Your draft report welcomed comments and the enclosed is provided in the spirit of trying to be both constructive and blunt regarding the matters at hand. These comments are based on my experience of over twenty five years in the fabric structure industry and represent my own opinion based upon this experience. We were all saddened by the irreparable personal injuries that occurred as a result of the Summit collapse. If there is any good to come out of this, perhaps it will be to effect positive changes in the fabric structure industry and its customers such that future collapses can be prevented.

On the whole, your report is very thorough as it relates to the wind load events which occurred on May 2, 2009 and to the relative structural capability of the Summit design to resist wind loads. However, it did not address other critical factors of structure design such as code live load requirements. In addition, it did not get into significant commentary regarding connection detailing, foundation design and other aspects of engineering a building system to safely withstand extremes of weather. Note that I have highlighted key comments/concerns in boldface.

Perhaps my most significant concern is that the NIST report limits its recommendations to fabric-covered steel frame structures. To the extent that these recommendations give the impression that only this specific type of structure is at risk, the report ignores serious deficiencies which can exist with any structure when poor design practices are followed.

In addition, for those of us in the industry that do take compliance with the building codes very seriously, I ask you not to paint with too broad a brush. **Properly engineered and constructed fabric-covered structures offer a number of environmental, life/safety, operational and other advantages over alternative types of construction.** Our employees take great pride in building reliable and safe products and they and their families depend on the well-being of this industry for their livelihoods. **This collapse did not occur because the structure involved was a fabric-covered structure. As the NIST report clearly shows, it collapsed because a single building supplier made fundamental and substantial errors during the engineering design process which resulted in a structure that was significantly under-designed.**

Regarding NIST's principal recommendations, I offer the following comments on each:

1) Rubb designs do not rely on the fabric to laterally brace to main truss chord members in our buildings. We believe it is a fundamental error to rely on fabric cladding for lateral support of main span trusses. Trusses are braced with both purlins and diagonal cables.

   Some companies in the fabric-covered structures industry, including companies utilizing aluminum versus steel frame systems, do rely on the fabric as an integral structural component and assume that their primary structural elements are continuously braced by the fabric. In my opinion, this non-conservative approach should not be used for structures which are required to comply with building code standards.

   - **The NIST recommendation should extend to both steel and aluminum frame fabric covered structures and neither should be allowed to utilize the fabric to laterally brace the structural frame system**

2) The NIST report recommendation regarding designing for "partially enclosed" rather than "fully enclosed" seems to be based, at least in part, on NIST's observation on page 41 of the report that, "the structure is flexible and under design wind loading conditions deforms significantly (see table 5-3), resulting in substantial distortions to the door framings.....".

   While I agree that section 6.2 of ASCE 7-05 should be followed with regard to determining the classification of a structure as fully or partially enclosed, **I disagree with the thesis that all fabric-covered structures deform significantly under wind loadings**. The calculations in the report were clear that structural members in the collapsed structure were significantly over-stressed with some internal truss chord members reaching a demand to capacity ratio of 500 to 600 percent. Clearly this implied the need for significantly more structural truss spans, larger frame elements, a substantially deeper structural truss or some combination thereof. There is no doubt in my mind that a properly engineered fabric-covered structure with a truss type frame will, for equal design load requirements, have substantially less frame deflection under load than will an aluminum beam structure or, for that matter, a pre-engineered metal building portal frame.

   In my experience, properly designed fabric-covered structural frames withstand severe hurricane forces and it is either the cladding system that is damaged and/or rolling steel door curtains that fail and not the structure itself.

3) The NIST report notes that the failure of one or a few frame members may propagate leading to a partial or total collapse of the structure. While this statement is true it also applies to the vast majority of pre-engineered and conventionally built structures in existence today. One need look no further than the progressive collapse of the World Trade Center to understand that not all risks can be guarded against. That said, clear and concise code requirements regarding when and how redundancy measures should be incorporated into structural design methodology would reduce the risk of catastrophic failure due to localized failure of a small number of structural elements.

Other Observations

1) **Major structure failures are more likely to involve pre-engineered metal or other type buildings than they are to involve fabric-covered structures.**

As I reviewed the NIST report I also went back and reviewed the trial transcripts and trial court conclusions in the case of the collapse of a 100,000 square foot Summit warehouse at the Port of Philadelphia during a snowstorm on President's Day in 2003. The structural engineer expert for the Port of Philadelphia in that case was a Charles N. Timbie, P.E. who has also now been working on behalf of the Dallas Cowboys. In his testimony on day 4 of the Summit trial in Philadelphia on June 26, 2006, Mr. Timbie testified (see page 176) that,

"We've seen a lot of pre-engineered buildings collapse in snow storms"

and when asked if those buildings were similar to the Summit building he replied:

"Well, they are, except the skin is metal. This particular building has a PVC membrane skin on it. The skeleton is steel, but the skin on all the other buildings I looked at was metal."

Page 177 of the transcript reveals that Mr. Timbie investigated three other structural failures which occurred during the 2003 President's Day storm, none of which were fabric structures.

2) **The root causes of failure in the Dallas Cowboys collapse were very similar to those in the Port of Philadelphia collapse and both involved the same building manufacturer.**

In the collapse of the Summit building in Philadelphia Mr. Timbie testified that,

"I think that there were three deficiencies in the building. Firstly, interpretation of the code. In order to design a building like this, you have to pick out the design criteria from the code, and I think there were mistakes and misinterpretations in applying the code to this building in both factors that determine what the load would be and, also, the extent of the roof where snow would lay.

Secondly, the building has what's called eccentric moment connections.... the bolted connections are moment plates....and it turns out these plates (splices/flanges) are eccentric. They are not concentric and it would cause a certain moment and a tendency for that plate to buckle.

The third defect we found was there are missing members. There were missing web members. Web members shown on the calculation and partially shown on the Summit drawings, but in the actual building, there were 124 of these members missing."

And in that case, two of many findings of the Court were as follows:

"75. In addition to designing a building which was inadequate to perform under the conditions and requirements contracted for, Summit further failed to construct the building in accordance with its own design requirements."

"76. These two fundamental failures produced a building which simply collapsed under the weight of the first significant snowfall of the new year which were conditions that would have been easily tolerated by the building had it been properly designed and constructed.'"

In the Philadelphia case the structural expert found that design and other errors essentially resulted in demand to capacity ratio of 300 percent for certain structural elements. In the case of the NIST report, the under design was apparently nearly twice as egregious with demand to capacity ratios of 500 to 600 percent for some members.

Again, these faults do not relate to the structure being a fabric-covered building, they relate to faulty design by a company that happens to make fabric-covered buildings. Had these structures been designed to existing codes, they would not have collapsed.

3) **The NIST report does not address the issue of design assumptions regarding roof live load. This leads to a fundamental and serious omission in the NIST conclusions.**

As a point of background, the clear interpretation of the 2006 International Building Code section 1607 is that the Code does not allow for a reduction of roof live load below a minimum of 12 psf. Yet it is standard practice for certain manufacturers in the fabric structure industry to ignore this code standard and to continue to claim compliance with the building code. This applies to manufacturers of both steel frame <u>and</u> aluminum frame fabric-covered structures.

A review of the web sites of at least two major manufacturers of aluminum frame fabric-covered structures will reveal language to the effect that the "Structure is engineered to <u>shed</u> snow". Independent engineering review data from a 1993 analysis of these manufacturers indicated that overstresses in the range of 400 to 500 percent of allowable were present for structures purported to comply with a 25 psf code specified live loading. These analyses revealed deflections for the aluminum structures that far exceeded the NIST conclusions for the steel frame structure which collapsed in Dallas. **It would be an omission to neglect to include a review of such aluminum frame structures.**

There is insufficient data in the NIST report to conclude what live load factor was used in the design of the collapsed Summit structure. However, data was provided in the NIST report on the dead load of the Summit structure. These dead loads are approximately one-third the weight that a Rubb structure would be if designed for the same building size and location.

	Summit	Rubb	Estimated Difference
Weight of Main Spans:	2.35 psf	+/- 5.8 psf	2.5x
Weight of Axial Steel:	0.44 psf	+/- 2.4 psf	5.5x
Total Frame Weight:	2.79 psf	+/- 8.2 psf	2.9x

Clearly there is little incentive for a company to over-design to the point that its product weighs approximately three times that of its competitor's for the same application and using basically the same materials. Conversely, there is a major competitive advantage gained by claiming to comply with the building code but actually designing to far below code standards.

I would also submit that had the Dallas Cowboys practice facility been properly designed using the code minimum 12 psf live roof load plus dead and collateral loads, it would have likely survived the 60 mph winds present during the microburst event on May 2, 2009.

The above leads to a recommendation that sets a minimum standard for live load capacity:

- **Roof live loads should be determined in accordance with the provisions of Chapter 7 of ASCE 7, but the design roof load should never be less than that determined by Section 1607 of the International Building Code (12 psf minimum).**

4) Major Errors in Design including lack of application of code prescribed loads, inadequate lateral bracing, poor design detailing and eccentric connection details can all lead to significant design overstresses.

In light of an AP report today that a huge facility which Summit completed last year at Texas A&M wasn't built to code, and is undergoing repairs, it should be clear that there is a pattern of questionable engineering design practice that has not been resolved even after two significant structural failures. From the AP report it appears that Summit has now contracted for wind tunnel testing apparently in order to find a way to counter the opinion of structural experts hired by Texas A&M that incorrect wind loading assumptions were used. In my experience code prescribed wind pressure coefficients are well-researched and appropriate for use in most circumstances. In our experience we have also found that certain manufacturer's wind tunnel test claims cannot be replicated by independent test laboratories.

One means to protect the end user from potential problems is to conduct a fully independent structural review of design assumptions, member sizing, lateral bracing, connection details, etc. as a condition of contract with a building supplier. The experience of both the Dallas Cowboys and the Port of Philadelphia and now Texas A&M has been that professional engineering reviews and seals provided by engineers subservient to the building supplier have proved ineffective in obtaining a structure which meets code standards.

- **NIST should consider recommendations regarding truly independent engineering review and/or consider recommendations to increase civil penalties for professional engineers who are grossly negligent in their review function**

These are my initial thoughts on the NIST draft report. I remain hopeful that NIST will not take any action that reflects negatively on an entire industry based upon the performance of a single company in that industry. If I can be of any further assistance in your work, please do not hesitate to contact me.

Sincerely,
Rubb, Inc.

David C. Nickerson
President

balanced load test?
A. Well, the truss did not fail and we knew this was not the -- this was not the critical loading, but in the other bay, the trusses failed at the ridge where there's a splice, and I thought that that would be the most logical place for the truss to fail under this loading. So in that regard, it didn't tell us the capacity of the truss.
We also hung from the truss three tape measures and had a transit to read the deflection on the sag in the roof as we loaded it up. What the test did tell us is that the deflections that we were actually measured during the test coincided very closely to the computer model that was developed by O'Donnell & Naccarato, and so it's sort of a verification of that model that's performing properly.
Q. And you said earlier that you didn't perform an unbalanced load test. Why is that?
A. Could you repeat that?
Q. You said earlier that you did not perform an unbalanced load test.
A. We did not perform an unbalanced load test because we could not find two suitable samples to load.

Q. And what would have the unbalanced load test tested for?
A. Well, the unbalanced load test indicates, according to the computer models, very, very much higher stresses on that particular type of load than you get in the load that we -- we actually performed here. This is a uniform test. Uniform, meaning it's an equal amount spread across the entire truss.
Q. And were the results of this uniform or balanced load test helpful in your investigation of the warehouse's collapse?
A. Not particularly, other than verifying the computer model.
Q. Okay. Taking into account all of the photographs and investigations and tests that you discussed yesterday and this morning, based on those, have you formed an opinion as to why the building collapsed on February 17th, 2003?
A. Yes.
Q. And in your opinion, why did the building collapse that day?
A. I think there were two deficiencies in the building. I think there were three deficiencies in the building. Firstly, interpretation of the code. In order to design a building like this, you have to

pick out the design criteria from the code, and I think that there were mistakes and misinterpretations in applying the code to this building in both factors that determine what the load would be and, also, the extent of roof where the snow would lay.
Secondly, the building has what's called eccentric moment connections. The building is fabricated at a steel shop. It's delivered in bundles to the site in segments. Each segment has a number on it indicating where it should go in the truss. Those segments are laid down on the ground and then bolted together, and the bolted connections are moment plates, similar to the one that was just put on the lawyer's table, and it turns out those plates are eccentric. They are not concentric and it would cause a certain moment and a tendency for that plate to buckle.
The third defect we found was there are missing members. There were missing web members. Web members shown on the calculation and partially shown on the Summit drawings, but in the actual building, there were 124 of these members missing.
Q. Okay. Mr. Timbie, did you reach those opinions to a reasonable degree of engineering certainty?
A. Yes, I did.

Q. And did you put those conclusions into a report?
A. Yes, I did.
Q. And those reports were made on April 22nd, 2005?
A. Yes.
Q. And March 30th, 2006?
A. Yes.
Q. And they have been marked, Your Honor, as Exhibits P-147 and 148.
Now, Mr. Timbie, if we could look at each of your conclusions a little more in-depth now.
Your first conclusion dealt with the snow load calculations?
A. That's correct. It's the Summit calculations.
Q. What is one of the first things that an engineer does when designing a building?
A. Well, the very first thing you do is find the -- if you're designing it structurally is to find the design criteria. You define the code that's applicable and then from the code you take the various factors in loading that the code prescribes for your particular building, and then you design from that point.
Now, obviously, if you start at the

residual strength a building would have. If you design a building for a certain load, you would incorporate in that load a safety factor of maybe one and a half, and that would account for any kind of damage to the material in transit, a missing bolt, erection cable too tight, that sort of thing. And this particular building with their loading was beyond that safety factor. It was an unsafe building the way it was designed here using these factors in uniform, and it was even more in the unbalanced. The more critical, as I say, is the unbalanced and, you know, that was like over 200 percent under-designed.

Q. Okay. You mentioned the unbalanced snow load. Could you explain what that is?

A. Yes. The code responds to the fact that during a heavy snow storm there's frequently wind accompanying that, and so the code requires a certain amount of wind to be scouring snow from -- scouring snow from one side of the building and then depositing a drift on the leeward side.

Q. Could you, please, show Exhibit P-542?

Can you zoom in on those calculations on the top, please? The length of the whole --

Do you recognize this drawing, Mr. Timbie?

A. This is a drawing that I prepared and it's in my report.

Q. And this drawing discusses the unbalanced snow load?

A. That's correct.

Q. And you were explaining earlier how to code -- how the code requires you to take into account the unbalanced snow load?

A. That's correct. The code, the applicable code, is actually -- I'm really not good at this -- that line. What this is a graph as to what load is going to be imposed on the building. The actual snow, of course, would be down in here, but this other graph at the top is the loading for ASCE 7-93 and the code requires the load to start at the ridge continue to increase until you get to the valley where there's a maximum load, in this case, it's 63 pounds a square foot and then it diminishes as you approach the second ridge.

In this case, it's 9.5 pounds a square foot and then the code says there will be a drift continuing until you reach 30-degree mark. At 30 degrees, it would start to diminish, and at 70 degrees it's assumed that's the eave and it's going to drop off the building.

Again, as we're looking at this, you'll note that the code -- the code recognized that there's no snow sliding off of the building when you're in the valley region.

Q. Is that why that is the highest point on the graph on the top?

A. Yes. That's the highest point. That's the maximum amount of snow that you would get on this type of roof in a windy snow storm.

Q. Mr. Timbie, what amount did Summit Cover-All use as the maximum unbalanced snow load?

A. Well, they arbitrarily took a value of 35 pounds a square foot. There's no formula. I believe there was a discussion among the engineers at Summit Cover-All and they just said, well, let's use 35 pounds a square foot. 35 pounds a square foot, of course, is in here someplace, but the more critical item here is the extent of snow. The code requires almost the full length of the building whereas Summit Cover-All confined their snow to an area in here. I think you have a graphic for that.

Q. Could you, please, show P-542A, and could you, please, zoom in?

Do you recognize this drawing, Mr. Timbie?

A. Yes, this is my earlier drawing where I superimposed the Summit Cover-All unbalanced snow load.

Q. Could you point out where that area was?

A. Well, I'll certainly try.

Well, it's that cross-hatch area there. It's this crossed hatch. I think you better than -- how's that?

Q. Good. And could you, please, explain why the use of -- what the effect of the use of 35 pounds per square foot was important?

A. Well, 35 is less than the required by code, which it was 62, I believe, behind that circle, but, of course, more importantly, they're assuming a little snow drift right down here. That's, obviously, not what the code intended. This code intended to have the unbalanced snow where the valley has snow and there's a snow drift on the opposite side.

So what this indicates is that the loading they've used was considerably less when the code requires and so their building was considerably less under structure and had less support for unbalanced snow loading.

Q. Could you clear that drawing out a second?

And what was the amount that was supposed to be used?
A. 63 pounds per square foot, according to the '93 code.
Q. And they used?
A. They used 35 on a much smaller extent.
Q. And in your investigation, did you uncover any calculations to reach this number 35 done by Summit Cover-All?
A. No, there's no formula or calculation.
Q. And were the use of these improper -- were the use of these variabilities, the use of these values a substantial factor in the collapse of the warehouse?
A. Absolutely. What this represented was an under-design, amazing proportions it was under-designed, over 200 percent.
What that means, at every existing truss, in order to support the code-required load, you need two other ones, so it's considerably under-designed.
Q. And this under-design contributed to the collapse of the warehouse?
A. Yes.
Q. Mr. Timbie, are these snow load calculations normally rechecked and verified for code compliance

by the engineer of record prior to ceiling drawings?
A. Yes, they would always be checked by the engineer of record unless, of course, the engineer of record prepared them, but they would be checked and the ensuing calculations would be checked as well and then you'd see them sign the calculations.
Q. Turning to your second conclusion as to why the building failed, you referred to it as a weak splice connection?
A. Eccentric, actually.
Q. Eccentric?
A. That doesn't mean it's goofy, it's a structural term for two loads that don't actually meet each other.
Q. Could you spell the word "eccentric" that you use there?
A. Spell the word eccentric?
Q. Yes.
A. E-C-C-E-N-T-R-I-C.
MR. TROY: Thank you.
THE COURT: What was the purpose of that?
MR. FREY: Your Honor, there's some confusion as to the spelling, if there was an X involved instead of two Cs.

THE COURT: Is there such a word E-X-E-N-T-R-I-C?
MR. TROY: Not that I'm aware of, Your Honor.
BY MR. TROY:
Q. In what respect was structural truss steel system effective to eccentric loading?
A. I could better describe probably by showing the Court the sample we have down here.
Q. Sure. And you're referring to what's been marked as Exhibit P-602?
THE COURT: Before we take -- before we -- I have to take care of some other business.
Take a ten-minute break, please.
(Whereupon, a short recess was taken.)
THE COURT: Resume.
BY MR. TROY:
Q. Before the break, we were discussing the eccentric loading of the structural steel truss system?
A. Yes, we were.
Q. And you were about to discuss Exhibit P-602, which was a portion of the roof truss?
A. Yes.
THE COURT: I don't think we were

discussing eccentric loading, we were discussing an eccentric splice, as I understand the testimony; is that right?
THE WITNESS: Yes, actually an eccentric-loaded splice.
THE COURT: That's because of the fact that the flange only covers 270 degrees of the diameter of the pipe?
THE WITNESS: That's part of it. There's a stiffener in there that plays as well.
THE COURT: All right. Go ahead.
Is this flange section marked, by the way?
MR. TROY: Yes, Your Honor. It's marked as 602.
THE COURT: Okay. P-602. Is it labeled?
MR. FREY: I'll label it now to make sure.
BY MR. TROY:
Q. Could you please show Exhibit P-181?
Do you recognize this drawing, Mr. Timbie?
A. Yes, this is a drawing that I traced off of

Michener Art Museum in Doylestown. We just added a lot of units to The Folkways Retirement Community in Lower Gwynned. Actually had a -- we designed the EXXON engineer exhibit at EPCOT in Orlando, Florida. We also renovate buildings. We renovated a lot of loft buildings in Philadelphia, like the Wireworks, Banks Street Court, Latisha Court, Riverworks and, actually, renovated the City Hall in Harrisburg into luxury apartments. All those buildings were renovated into luxury apartments.

We do a lot of forensic work as well concerning natural disasters, man-made disasters. For example, snow losses. We looked at the Winnebago Factory in Floyd City, Iowa, which collapsed. Put them out of business for about a year. I'm working currently on a fire loss at a Mercedes dealership burned in Reading, and we're there to try to see how to put the building back together.

We do wind damage. You may recall the Academy of Music was closed before the season was over several years ago because of a wind damage claim, and we looked at that for a Factory Mutual Insurance Company. We've looked at buildings that have been hit by almost anything you can think of, cars, trolley cars, moving vans, airplanes, ships,

trains. Even it was a conductor backing a train into a factory and he forgot how many cars he had and he backed it through the back wall of a factory. So we do a lot, quite a range of work in new buildings, renovations and forensic.

Q. And on average, how is your work divided between structural design of new buildings, refurbishing older buildings and investigating building failures?

A. It's been about a third of each, lately, one third for each.

Q. Okay. And have you taught any courses in engineering?

A. Yes, I've taught at Drexel University. I taught there for twenty years as an adjunct assistant professor in the evenings, taught Structural Systems I, II, III, Materials and Structural Design I, II and III, Engineering Economy and Statistics, and my favorite has been teaching architectural studios.

I'll actually go to the architectural studios and sit with the students and consult with them as though I were a consulting engineer, and as they work on their building, design their building architecturally, we individually try to develop a structural system for building as well. I did that

at Drexel for 20 years. I did that at Temple for eight years.

Q. And do you belong to any professional societies?

A. National Society of Professional Engineers.

Q. And you explain that you were an instructor for the American Institute of Architects?

A. Yes. I taught the exam preparation seminars for about eight years for the American Institute of Architects. Those are the exams where the architects are trying to be registered and their least favorite part of the exam turns out to be structures for some reason, and I conducted seminars to try to bring them up to speed in those fields.

Q. And how many times have you been retained as an expert in building failure cases?

A. I would say 50, 60, something like that.

Q. And have you investigated the cause of building failures in cases where metal buildings collapsed in snow storms?

A. Yes. Yes, I have. Probably the largest one was the Mannington Carpet Mills in Calhoun, Georgia which collapsed. They lost -- it was actually a set of buildings and they lost 400,000 square feet of buildings in a snow storm. We investigated,

recently, the Toys R Us collapse in just outside Baltimore County. A large building called Del Homes catalogue group in New Castle, Delaware or in Dover, Delaware, a bedding manufacturer collapsed in New Castle. We've investigated a shopping center in Sicklerville, New Jersey which collapsed and an adjacent drug store which collapsed, and a Sun Sweet Fruit building warehouse that collapsed in Temple, Pennsylvania. One called Amatax. We've seen a lot of pre-engineered buildings collapse in snow storms.

Q. And they are similar to the design of the building at issue here?

A. Well, they are, except the skin is metal. This particular building has a PVC membrane skin on it. The skeleton is steel, but the skin on all the other buildings I looked at was metal.

Q. And they were also pre-engineered metal frames?

A. Yes, they are pre-engineered as opposed to conventionally-framed building, a conventionally-framed building is one where the architect would or the owner would hire an architect and he would hire his team of engineers and they would draw up drawings of the building, take those drawings and give them to a steel fabricator or steel contractor. They would fabricate the steel members and deliver them then and an erector would come and put the building up.

With these pre-engineered buildings, it's one package. The contract is one package where the building is designed, fabricated, delivered and, in this case, erected by one company.
Q. Okay. And have you investigated other building failures in connection with the Presidents' Day 2003 snow storm?
A. Yes.
Q. And can you give some examples to the Court of those investigations?
A. Well, the Toys R Us building collapsed during that snow storm. The catalogue resources collapsed during that snow storm in Dover, Delaware. The one in New Castle County, the bedding manufacturer, fiber products, collapsed in that snow storm.
Q. Okay. And when you do these investigations, who hires you to do them?
A. Mostly insurance company. I've done work for public adjusters. I've done work for owners of the building, but most of the work I got is from insurance companies. And it's, basically, to investigate the building, to determine if it can be repaired, if it can be repaired, how would you do

that and, also, frequently to determine the cause of damage to the building.
Q. And you've testified for insurance companies, owners and builders in the past?
A. Yes, I have, mostly insurance companies.
Q. Could you please show Exhibit 149?

Mr. Timbie, is this a copy of your CV?
A. Yes, it is.
Q. Can you show the next page, please?

There's a complete copy that was attached to your report?
A. Yes.

MR. TROY: Your Honor, I offer Mr. Timbie as an expert in structural engineering with respect to the cause of the partial collapse on the Tioga Marine Terminal warehouse on February 17th, 2003.

THE COURT: Does anyone wish to inquire?

MR. PHILLIPS: No questions, Your Honor.

MS. HORNEFF: Just a few, Your Honor.

\- - -

VOIR DIRE EXAMINATION - CROSS

BY MS. HORNEFF:

Q. Good afternoon, Mr. Timbie.
A. Good afternoon.
Q. My name is Elizabeth Horneff. I think we've met once or twice.
A. I believe in the building.
Q. A couple of questions. Looking at the CV you've presented to the Court, you've been involved in the design of over 1200 buildings, more or less?
A. More or less, yes.
Q. How many of those were frame-supported membrane structures?
A. I've never designed a frame-supported membrane structure. Most of my work is in pre-engineered or in conventional-framed buildings. Most of the work that I design, it turns out that most of the forensic work is in pre-engineered buildings.
Q. Of the renovation work that you do, the one-third of your work that involves building renovation, how much of that has involved frame-supported membrane structures?
A. I don't recall one.
Q. And, again, on the third of your work that you do is forensic, how many framed-supported membrane structures have you been asked to examine?
A. This is the first one with a membrane on it.

The structure, itself, is similar to the other buildings. It's a pre-engineered metal building.

In this case, steel trusses made out of tubes, and instead of skinning that with a metal skin, it's skinned with a membrane, but the analysis of the frames are pretty much the same.
Q. Now, you're a structural engineer?
A. That's correct.
Q. Not a mechanical engineer?
A. That's correct.
Q. Do you have any expertise as a fire suppression expert?
A. No.
Q. How about as a fire code expert?
A. No, I'm not. I'm a structural engineer.
Q. Sir, do you have any expertise as a meteorologist?
A. I really have no training in that. Sometimes in a small project, a small collapse, I will get involved in collecting surface data from the weather services in order to determine how much snow was on that building. On a larger project like this, I would always obtain the services of a meteorologist.
Q. And you're not a forensic meteorologist yourself?

From: Mark Waggoner [MWaggoner@walterpmoore.com]

Sent: Monday, October 19, 2009 12:40 PM

To: structuralsafety@nist.gov

Subject: Comments on Draft NIST Report on Cowboys Practice Facility

Hello,

I have the following comments on the Draft NIST Report on the Dallas Cowboys Practice Facility:

1. The report emphasizes a reliance on the fabric to provide lateral bracing for column stability of the top chord. I agree with this conclusion as there are no documented tests or literature that suggest that the type of fabric used has either adequate stiffness or reliable strength to provide bracing. Detailed analytical evaluations of frame and membrane structures confirm this. However, the emphasis on the bracing aspect misses a larger design issue, which is the lack of design of the top chord for any horizontal bending moments. Such moments would arise from the unbalanced horizontal component of the fabric membrane in-plane force (note this is much higher than the applied vertical component) due to pattern loading on adjacent fabric panels, or could result from a tear in a fabric panel on one side of an internal truss frame. It is good engineering practice to design for unbalanced horizontal loading on a top chord resulting from the tear case under some form of “extreme event” load combination. This is unfortunately not widely followed in practice, but is consistent with the spirit of Section 1.4 “General Structural Integrity” of ASCE 7-05. Likely if the truss chords had been sized for horizontal bending they would have been of adequate proportion to function as a compression member between cable brace points.

2. The report comments that the building should have been designed as partially enclosed rather than fully enclosed based on the extent of openings. While the statement made on page 41 regarding door openings is debatable, consideration should also be given to the potential for internal pressure changes resulting from tears in the fabric membrane skin. Often such membranes are treated as non-structural covers and not explicitly designed, but reliance on the integrity of the enclosure has a large influence on the total wind design pressures on the building frame. In my opinion the consequences of

breaches in the enclosure likely have a larger influence on the sequence of collapse than is addressed in the draft report.

3. As discussed above the potential for tears in the fabric can lead to significant design issues. Fabric tearing is not a well understood phenomenon, and is currently not an explicit design consideration when evaluating fabric design (reference draft ASCE tensioned fabric structures standard). As a result, many common structural fabrics in use today have relatively low tear strengths as compared to their tensile strength. This area needs more research to better develop appropriate design methods.

4. Figure 5-3 indicates a pattern of reactions from fabric membrane preloading applied to the frame. These appear to have been derived from hand calculations based on the fabric curvature. It is unclear whether a proper large displacement nonlinear fabric analysis was done by either NIST or the original designer for either prestress or applied loads. Even for what seem like relatively simple structures like this, fabric behavior can be quite complex and should be explicitly evaluated. The NIST analysis appears to include only the self-equilibrating horizontal loading under prestress, and does not account for the potentially significantly higher horizontal loads that would be imparted on the steel frame with the membrane under full wind or live load. It is also unclear whether the fabric membrane stress limits under load were evaluated by the original designer.

5. The report does not appear to address whether the cable bracing system was adequate for local chord compression member bracing or global frame bracing. Stability of roof truss systems is a complex subject that is not well understood. A proper evaluation would generally include an eigenvalue buckling analysis of the full system, and more detailed studies of modes identified as critical in the buckling analysis. This particular system is somewhat more complex due to the potential for non-linear behavior (slack/not slack) in the cable braces. It is unclear whether the cable braces were prestressed in any way.

The October 6, 2009 press release mentions that “NIST will brief and provide technical support on the recommendation to the American Society of Civil Engineers (ASCE) committee currently developing a building standard specifically for tensioned fabric structures.” I would recommend that NIST also engage the ASCE SEI Progressive Collapse Standards and Guidance Committee of which I am a member as well as several of the NIST staff, including H.S. Lew and Fahim Sadek. Also, Dr. Todd Helwig at the University of Texas at Austin is currently conducting the only significant research on truss bracing of which we are aware, and his recent findings may help lead to a better understanding of the truss stability issues seen in the Cowboys Practice Facility collapse.

Please do not hesitate to contact me if you have any questions on the above comments.

Best Regards,

Mark Waggoner

Principal

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From: Steve Fatzinger [sfatzinger@lightweightgroup.com]
Sent: Friday, October 23, 2009 1:18 PM
To: structuralsafety@nist.gov
Subject: NISTIR 7636 Comments

Stephen Cauffman,

I took the opportunity to read the 'Draft Report on the Collapse of the Dallas Cowboys Indoor Practice Facility, May 2, 2009' (NISTIR 7636). It is a very impressive report and I enjoyed perusing through it.

After reading the report, I had some thoughts to offer about the contents of the report. Since comments are being solicited, I am sending them to you in an attachment to this email. Some of my thoughts deal with the issue of classification of the structure as 'enclosed' versus 'partially enclosed'. The attached worksheet is included to hopefully support my thoughts.

Please let me know if you have problems opening any of the attachments, or if you have any questions regarding the attachments. I certainly look forward to hearing more about this incident as we are all learning quite a bit because of it.

Best regards,
Steven Fatzinger, PE
Senior Engineer
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COMMENTS ON NISTIR 7636

'DRAFT REPORT ON THE COLLAPSE OF THE DALLAS COWBOYS INDOOR PRACTICE FACILITY, MAY 2, 2009'

NATIONAL INSTITUTE OF STANDARDS AND TECHNOLOGY.

SECTION 4.1.1 DESIGN WIND LOADS
PAGE 41, LAST PARAGRAPH:

"In addition, this structure is flexible and under design wind loading conditions, the structure deforms significantly (see Table 5-3), resulting in substantial distortions to the door framings that can cause the doors to fail to remain closed or latched during a wind storm (see Figure 3-3)."

If it is accepted that the door frames distort enough to fail and/or open, then it would be expected that these distortions would occur to the doors that are located in the walls perpendicular to the direction of the applied wind. Therefore the opening of the doors increases the A_{oi} portion of the formula and thus decreases the A_o / A_{oi} ratio. Hence, the structure classification leans more toward "enclosed" if the side doors are considered open due to distortion, and not toward "partially enclosed" as implied by the report.

PAGE 41, LAST PARAGRAPH:

"Based on the consideration of vent openings and the possibility of additional openings due to the doors, this structure should be considered partially enclosed for the purposes of internal pressure evaluation."

If the analysis is accepted, then this statement is only true for wind applied to the north or south faces (gables). The structure can be considered enclosed for purposes of internal pressure evaluation when the wind load is applied to the east or west faces (sides) of the structure. This is based on an evaluation that considered all logical combinations of vents and doors being open or closed.

PAGE 41-42:

In order to be complete, the report should account for the second part of the ASCE 7 condition for determination of partially enclosed structures; where A_o > the smaller of 4 ft^2 or 0.01 A_g AND A_{oi} / A_{gi} <= 0.2. It cannot be determined from the report whether this condition was considered or not.

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FIGURE 4-1
PAGE 42, CHART LEGEND

In order for the rollup doors to provide 48 ft^2 of area as indicated in the Figure's legend, they would have to have dimensions equal to 12' x 4', 10' x 4.8', or possibly 8' x 6'. The picture on page 30 (Figure 3-3) shows that the rollup door is over 1.5 times the height, and about 4 times the width, of the personnel door. A common size for rollup doors in a facility of this type would be more in line with 12' x 14', which would be close to the scale of what is shown in the Figure 4-1. If this is the case, then the opening area for the rollup doors should be shown in the legend as 168 ft^2, and not 48 ft^2.

SECTION 4.2.2
PAGE 51, CASE 1

"… the un-braced length is taken to be the larger of the truss panel length or the length between points of cable bracing."

This is true, but only due to the fact that the radius of gyration for a round shape is the same in both the major and minor axis. In an effort to be complete, I believe that this statement should include this information and not leave it to be assumed.

SECTION 5.2
PAGE 68, FIGURE 5-5

From the Figure, it can be seen that the moment at the eave reduces to 0 in less than a panel width. The analysis in the report is conservative as it uses the maximum value at the node point, and does not consider the moment some distance away at the face of the gusset/web member. Given the rapid decrease in the moment, this could have a significant impact on the value used for the moment. In this case, the reduction of the moment will only affect how much over-stressed the member is, and not the fact that it is over-stressed.

SECTION 5.5 ANALYSIS USING ESTIMATED WIND LOADS ON MAY 2, 2009
PAGE 76, SECOND PARAGRAPH

"The directionality factor K_d was set to 1.0, rather than 0.85, because wind loads for a specific wind direction are sought …"

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Section 6.5.4.4 of ASCE7-05 states "The wind directionality factor, K_d, shall be determined from Table 6-4. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4. The wording in this section seems to imply that there is no allowance for a different value for K_d due to specific wind direction analysis. Further, none of the values for K_d in Table 6-4 are shown as 1.0.

Section C6.5.4.4 of ASCE7-05 states the factor K_d "… accounts for two effects: (1) The reduced probability of maximum winds coming from any given direction and (2) the reduced probability of the maximum pressure coefficient occurring for any given wind direction." By using a value of 1.0 for K_d, NIST implies that the maximum wind is applied in the specified direction and that the maximum pressure coefficients are also occurring. This may be conservative in an analysis that is determining the actual conditions that were occurring at the time of the structure's failure.

PAGE 76, SECOND PARAGRAPH

"Internal pressures were calculated assuming partially enclosed conditions …"

As noted in the comments for Section 4.1.1, I believe that the structure should be classified as enclosed when considering a wind load that is normal to structure's ridge line.

SECTION 7.2 NIST RECOMMENDATION
PAGE 86, FIRST RECOMMENDATION

"A review of the state of practice indicates that there is some disparity on this practice among designers and fabricators of this class of structures, as some rely on fabric to provide lateral support to the frames, while others do not."

The statement, as worded, could be taken to imply that the industry is split somewhat evenly on this practice. It is my experience that a significant majority of engineers in this industry do <u>not</u> use fabric to provide lateral support to the frames.

GENERAL

Not mentioned anywhere within the report, but probably as important as fabric failure, is the fact that the modulus of elasticity of the roof fabric may be too low to provide sufficient support for the steel. Before the fabric can develop sufficient resistance against sideways movement of the steel member, the elongation of the fabric and thus the deflection of the steel member could be such that a P-Delta effect is already occurring.

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Structure Dimensions (NISTIT 7636)

Height =	66.4 ft	(mean roof height)
Width =	204 ft	
Length =	406 ft	
Roof Slope =	21 degrees	

Areas of Structure and Openings By Location (ft^2)

		Wall-N	Wall-E	Wall-S	Wall-W	Roof	
	Unit	13,546	26,958	13,546	26,958	88,717	
Roof Vent	16.0	-	-	-	-	64	(4 @ 16 ft^2)
Gable Vent	25.0	250	-	150	-	-	(10, 6 @ 25 ft^2, respectively)
Pers. Doors	24.5	49	123	98	49	-	(2,5,4,2 @ 24.5 ft^2, resp.)
Roll-up Doors	168.0	-	-	336	-	-	(2 @ 168 ft^2)

NOTES:
Wall-N and Wall-S are gable ends.
Wall-E and Wall-W are sides.
Roof Vents are considered open at all times.
Gable Vents and Doors are either open or closed.

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Location Desc. ***Wall-N***

Roof Vent Open?	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Gable Vent Open	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE
Pers. Door Open	TRUE	TRUE	FALSE	FALSE	TRUE	TRUE	FALSE	FALSE
Roll Door Open?	TRUE	TRUE	TRUE	TRUE	FALSE	FALSE	FALSE	FALSE
A_o	299.0	49.0	250.0	-	299.0	49.0	250.0	-
A_{oi}	819.5	669.5	550.0	400.0	483.5	333.5	214.0	64.0
A_o / A_{oi}	0.36	0.07	0.45	-	0.62	0.15	1.17	-
Part 1 Satisfied?	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	**TRUE**	FALSE
A_g	13,546	13,546	13,546	13,546	13,546	13,546	13,546	13,546
A_{gi}	156,179	156,179	156,179	156,179	156,179	156,179	156,179	156,179
A_{oi} / A_{gi}	0.0052	0.0043	0.0035	0.0026	0.0031	0.0021	0.0014	0.0004
Min($0.01A_g$, 4)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
A_o > Min ?	TRUE	TRUE	TRUE	FALSE	TRUE	TRUE	TRUE	FALSE
A_{oi} / A_{gi} <= 0.2	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Part 2 Satisfied?	**TRUE**	**TRUE**	**TRUE**	FALSE	**TRUE**	**TRUE**	**TRUE**	FALSE
Part 1&2 Satisfied ?	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	**TRUE**	FALSE

(TRUE = Partially Enclosed, FALSE = Enclosed)

Location Desc. ***Wall-E***

Roof Vent Open?	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Gable Vent Open	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE
Pers. Door Open	TRUE	TRUE	FALSE	FALSE	TRUE	TRUE	FALSE	FALSE
Roll Door Open?	TRUE	TRUE	TRUE	TRUE	FALSE	FALSE	FALSE	FALSE
A_o	122.5	122.5	-	-	122.5	122.5	-	-
A_{oi}	996.0	596.0	800.0	400.0	660.0	260.0	464.0	64.0
A_o / A_{oi}	0.12	0.21	-	-	0.19	0.47	-	-
Part 1 Satisfied?	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
A_g	26,958	26,958	26,958	26,958	26,958	26,958	26,958	26,958
A_{gi}	142,766	142,766	142,766	142,766	142,766	142,766	142,766	142,766
A_{oi} / A_{gi}	0.0070	0.0042	0.0056	0.0028	0.0046	0.0018	0.0033	0.0004
Min($0.01A_g$, 4)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
A_o > Min ?	TRUE	TRUE	FALSE	FALSE	TRUE	TRUE	FALSE	FALSE
A_{oi} / A_{gi} <= 0.2	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Part 2 Satisfied?	**TRUE**	**TRUE**	FALSE	FALSE	**TRUE**	**TRUE**	FALSE	FALSE
Part 1&2 Satisfied ?	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE

(TRUE = Partially Enclosed, FALSE = Enclosed)

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Location Desc.	Wall-S							
Roof Vent Open?	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Gable Vent Open	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE
Pers. Door Open?	TRUE	TRUE	FALSE	FALSE	TRUE	TRUE	FALSE	FALSE
Roll Door Open?	TRUE	TRUE	TRUE	TRUE	FALSE	FALSE	FALSE	FALSE
A_o	584.0	434.0	486.0	336.0	248.0	98.0	150.0	-
A_{oi}	534.5	284.5	314.0	64.0	534.5	284.5	314.0	64.0
A_o / A_{oi}	1.09	1.53	1.55	5.25	0.46	0.34	0.48	-
Part 1 Satisfied?	FALSE	**TRUE**	**TRUE**	**TRUE**	FALSE	FALSE	FALSE	FALSE
A_g	13,546	13,546	13,546	13,546	13,546	13,546	13,546	13,546
A_{gi}	156,179	156,179	156,179	156,179	156,179	156,179	156,179	156,179
A_{oi} / A_{gi}	0.0034	0.0018	0.0020	0.0004	0.0034	0.0018	0.0020	0.0004
Min($0.01A_g$, 4)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
A_o > Min ?	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	FALSE
$A_{oi} / A_{gi} <= 0.2$	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Part 2 Satisfied?	**TRUE**	**TRUE**	**TRUE**	**TRUE**	**TRUE**	**TRUE**	**TRUE**	FALSE
Part 1&2 Satisfied ?	FALSE	**TRUE**	**TRUE**	**TRUE**	FALSE	FALSE	FALSE	FALSE

(TRUE = Partially Enclosed, FALSE = Enclosed)

Location Desc.	Wall-W							
Roof Vent Open?	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Gable Vent Open	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE	TRUE	FALSE
Pers. Door Open?	TRUE	TRUE	FALSE	FALSE	TRUE	TRUE	FALSE	FALSE
Roll Door Open?	TRUE	TRUE	TRUE	TRUE	FALSE	FALSE	FALSE	FALSE
A_o	49.0	49.0	-	-	49.0	49.0	-	-
A_{oi}	1,069.5	669.5	800.0	400.0	733.5	333.5	464.0	64.0
A_o / A_{oi}	0.05	0.07	-	-	0.07	0.15	-	-
Part 1 Satisfied?	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
A_g	26,958	26,958	26,958	26,958	26,958	26,958	26,958	26,958
A_{gi}	142,766	142,766	142,766	142,766	142,766	142,766	142,766	142,766
A_{oi} / A_{gi}	0.0075	0.0047	0.0056	0.0028	0.0051	0.0023	0.0033	0.0004
Min($0.01A_g$, 4)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
A_o > Min ?	TRUE	TRUE	FALSE	FALSE	TRUE	TRUE	FALSE	FALSE
$A_{oi} / A_{gi} <= 0.2$	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE	TRUE
Part 2 Satisfied?	**TRUE**	**TRUE**	FALSE	FALSE	**TRUE**	**TRUE**	FALSE	FALSE
Part 1&2 Satisfied ?	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE

(TRUE = Partially Enclosed, FALSE = Enclosed)

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