

November 6, 2017

Mr. John Erickson
DSGW Architects
2 West First Street, Suite 201
Duluth, MN 55802

Sent by Email to: jerickson@dsgw.com

**Re: Grand Rapids IRA Civic Center – West Venue Roof Capacity Review
NCE Job No. 17-665**

Dear John,

NCE was retained to field measure and perform an analysis of the existing West Venue Roof Structure for snow loading. It is our understanding that this information would be utilized to assist planning for the future of the overall facility. Although our scope was limited to this analysis, the results are concerning, and we have therefore provided this limited written summary report and also plan to attend a meeting to present this information on 11-8-2017 (and be available for questions).

The original building was apparently constructed in about 1962 by a non-profit community organization called the Itasca Recreation Association (IRA). We do not have access to original construction documents. Based on our discussions and observations, the venue was later expanded (apparently in the early 1980's) to the north approximately 40 feet by adding two new trusses and roof purlins. A large addition including an additional East Venue and entry area was added in 1995. The truss members of the 1980's addition are significantly larger than those of the original 1962 building having the same span and spacing.

The trusses spaced at 20 feet on center spanning 132' are made from doug-fir glue laminated engineered timber. The web members are solid doug-fir and connected with steel plates to the top and bottom chord members of the truss.

Review of Documentation:

1. We reviewed the three documents from Stuart Anderson P.E. These documents include:

- a) Documentation and repair of a roof truss diagonal web member (3/25/2001). This document describes the failure of a web member due to unbalanced but moderate snow loading and the repair procedures done in order to re-open the venue. No structural analysis was completed for the trusses but Mr. Anderson did recommend that one be performed. This analysis was apparently not completed until now.
- b) An Engineering Study and opinion regarding the addition of speakers supported from the roof structure (9/28/2007). No structural analysis was completed for this study.

- c) Structural Engineering Memorandum (2/13/2017) regarding the installation of a new air duct supported from the roof structure. A limited structural Analysis of specific members was apparently completed for this study.
2. We reviewed the construction documents from the 1995 Addition to the facility.

Code Requirements:

1. The State of Minnesota did not adopt a statewide building code until 1971 by adoption of the 1970 Uniform Building Code. Therefore the design loads used for the building in 1962 were likely determined by the engineer / Architect.
2. Generally, the Uniform building code (UBC) remained in effect from 1971, updated every three to 6 years until 2003 when the 2000 international building code (IBC) was adopted in Minnesota. Under the UBC, the snow load requirement for the Grand Rapids area was generally close to 30-35 psf and did not consider unbalanced roof snow loads. (*unbalanced snow load is when snow accumulates more on one side of a ridge assisted by the wind*) When the IBC was adopted in 2003, the required snow loads were significantly changed / increased.
3. The required uniform snow load for this building is now 46 psf. (*an increase of 53% compared to 30 psf as would be appropriate in the 1980s.*)
4. The current required unbalanced design snow load for this roof structure is approximately 92 psf maximum near the eave to 23 psf at the ridge.
[Engineers Note: Measured, snow density measurements generally vary from 15 pcf to nearly 28 pcf which means that current 46 psf design load is approximately 2 to 3 feet of snow]
5. Under the current 2015 Minnesota Conservation code: Generally the existing building is not required to be improved to resist current design loads unless:
 - a) The use changes
 - b) Additional weight is added
 - c) Additional thermal insulation is added
 - d) The building experiences a partial or total collapse
 - e) If there is a significant risk of collapse

Structural Analysis Description:

We have field measured the existing roof trusses and roof purlins (*roof members spanning between trusses*). We have used both a three-dimensional computer model and traditional hand calculations to generate and verify our results of analysis (see attached). We have also revisited the site to verify our measurements. Structural analysis is defined as “the determination of the effects of loads on physical structures”. In our analysis we used code prescribed loading as discussed above as well as several reduced loading conditions to determine the appropriate “design load capacity of the structure”. This design load capacity does not mean that this is the load in which failure will occur. Instead, the design load capacity is the calculated capacity of the structure based on accepted and published design material strengths of the materials in place and sound engineering principals.

Results of Analysis:

1. The Roof purlins (3 ½ x 13 D.F.) have a design snow load capacity of approximately 25 psf.

2. The 1962 Roof Trusses have a uniform snow load design capacity of approximately 20 psf.
3. The 1980's era roof trusses have a uniform snow load design capacity of approximately 30 psf.
4. Under unbalanced snow loading, several slender truss web members change from tension members to compression members, when this occurs, the truss can support only a design load of 15 psf over one half of the truss.
5. Connections: The connection capacity generally follows the member capacity, in other words the 1962 truss appears to have been professionally designed for 20 psf and the 1982 truss appears to have been professionally designed for 30 psf (both for uniform loading only)

Site Observations and discussions:

1. The existing structure appears to be performing fairly well. During our visit however, we did observe a few web members of the 1962 trusses that appear from the ground to be slightly bowed or out of alignment.
2. The cast-in-place Concrete piers supporting the trusses and the masonry walls appear to be performing adequately.
3. We discussed previous damage to adjacent roof top units from sliding snow.
4. We noted that insulation and a new membrane roof were added in the 1995 construction documents.

Professional Opinions:

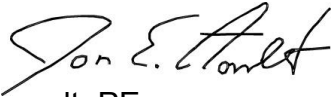
1. In our professional opinion, the severe under-capacity of the 1962 roof trusses is a dangerous condition and should be corrected. We recommend reinforcing or replacing the existing roof structure as soon as possible.

Note:

- a) This is a situation where the facility is most highly used during the winter season when several hundred people could be seated under a roof with only minimal unbalanced snow loading capacity.
 - b) A partial failure (web truss member failure) due to unbalanced loading has occurred in the past. This failure could potentially have resulted in a progressive collapse of the roof structure.
 - c) Since the time of the partial failure, the condition has apparently been slightly worsened by added insulation, ductwork and mechanical systems.
2. We understand that this cannot be corrected immediately and that the building has stood basically undamaged for nearly 55 years.
 3. Based on the above, we recommend the following until the situation is corrected.
 - a) Signage should be considered to warn the public and employees of the significant under-capacity of the roof structure for snow loading. This could potentially remind them to remove snow etc.
 - b) If a significant snow event occurs that results in more than 1 foot of snow being supported on the roof, the roof should be hand-shoveled before the facility is occupied.

c) If an event is scheduled in this facility during a severe snow event, that event should be moved to the east venue or cancelled.

Sincerely,



Jon E. Aamodt, PE
Principal Partner

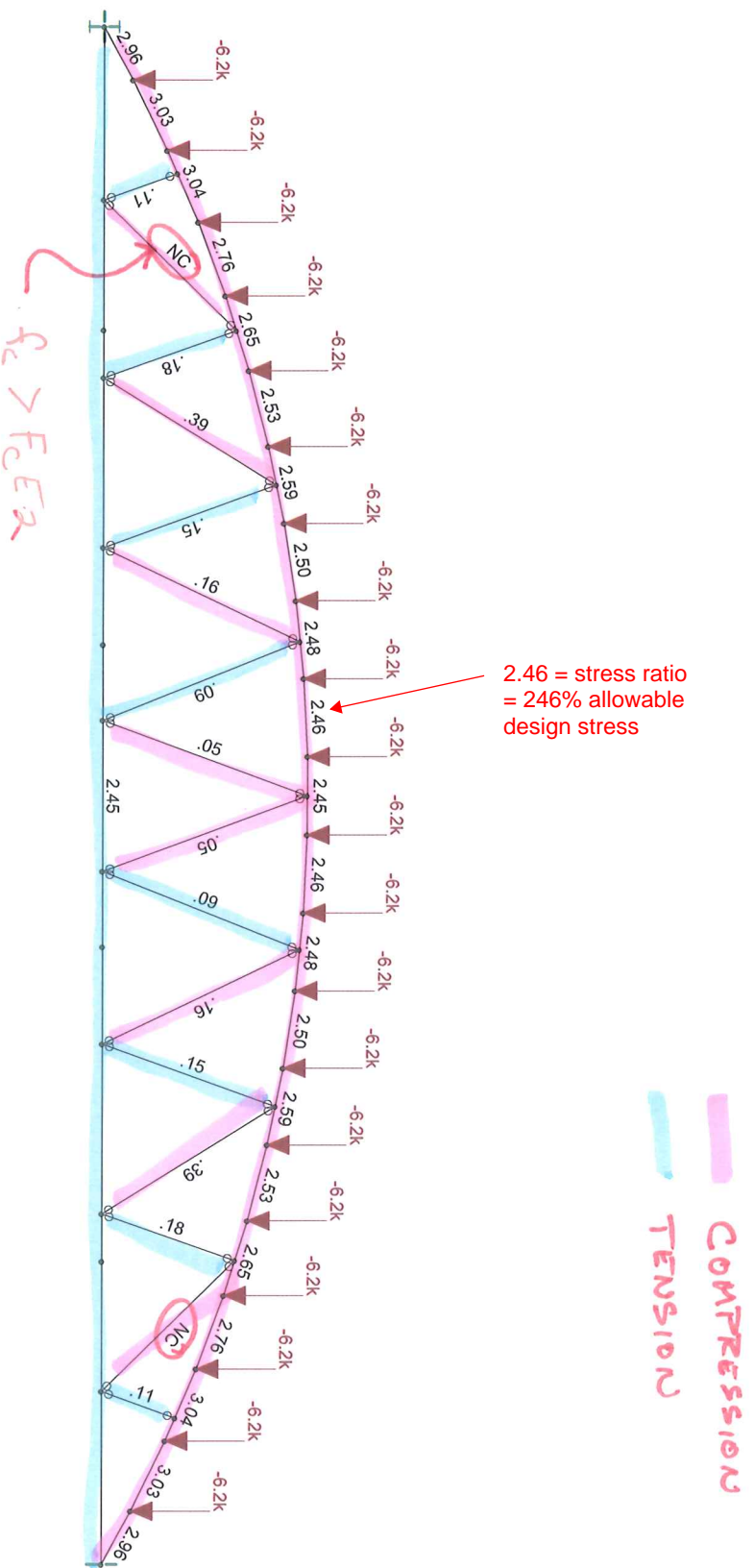
Professional Certification:

I hereby certify that this plan, specification or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.



Jon E. Aamodt, P.E.
MN Reg. No. 24838

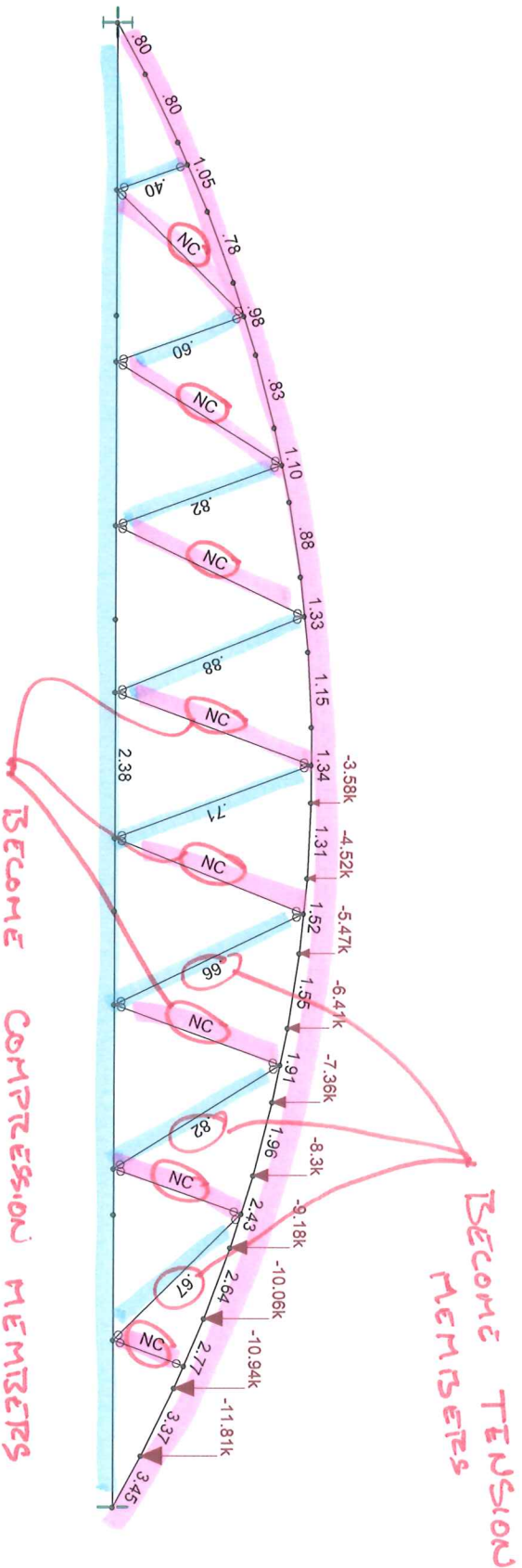
Date 11-06-17



COMPRESSION
TENSION

Member Code Checks Displayed
Loads: BLC 2, Uniform Snow
Results for LC 2, Dead + Snow

NCE		SK - 2
RTT	G.R. IRA Truss	Oct 20, 2017 at 11:02 AM
17-665	Current Snow Load = 46 PSF	17-665_Truss.r3d



Member Code Checks Displayed
Loads: BLC 3, Unbalanced Snow
Results for LC 3, Dead + Unbalanced

NCE

RTT

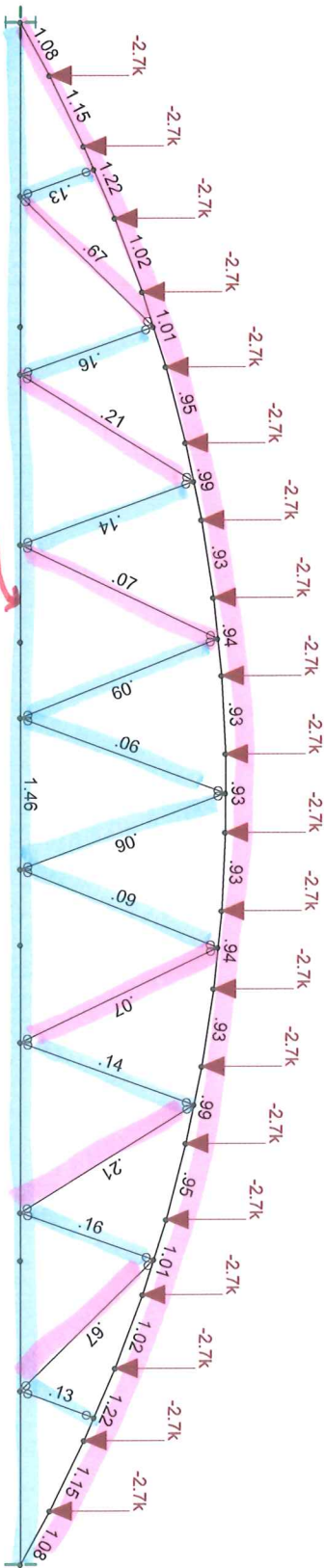
17-665

G.R. IRA Truss
Unbalanced Snow Load w/ Uniform Snow = 46 PSF

SK - 4

Oct 20, 2017 at 11:06 AM

17-665_Truss.r3d



Member Code Checks Displayed
Loads: BLC 4, Snow Capacity
Results for LC 4, Dead + Snow Load Capacity

NCE

RTT

17-665

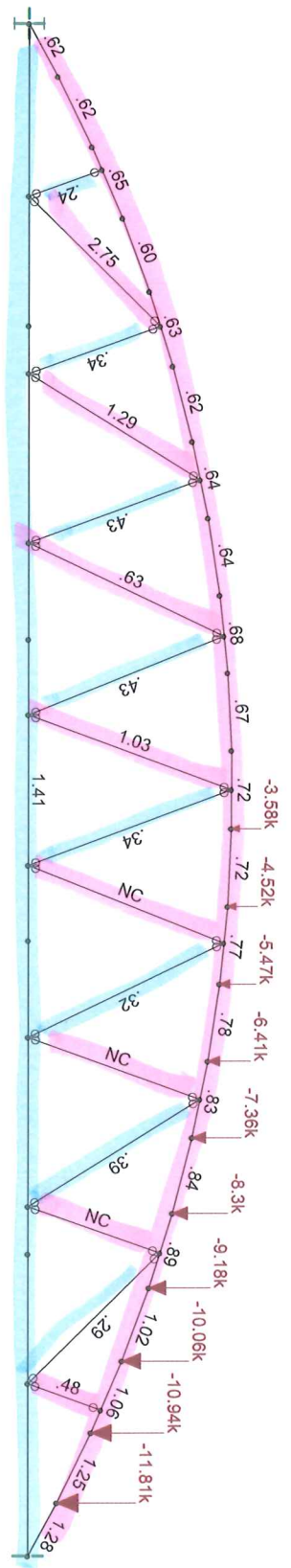
G.R. IRA Truss

Uniform Snow Load @ 20 PSF

SK - 3

Oct 20, 2017 at 11:04 AM

17-665_Truss.r3d



LOADS MULTIPLIED BY 20/46

Member Code Checks Displayed
 Loads: BLC 3, Unbalanced Snow
 Results for LC 5, Dead + Unbalanced Snow @ Capacity

NCE

RTT

17-665

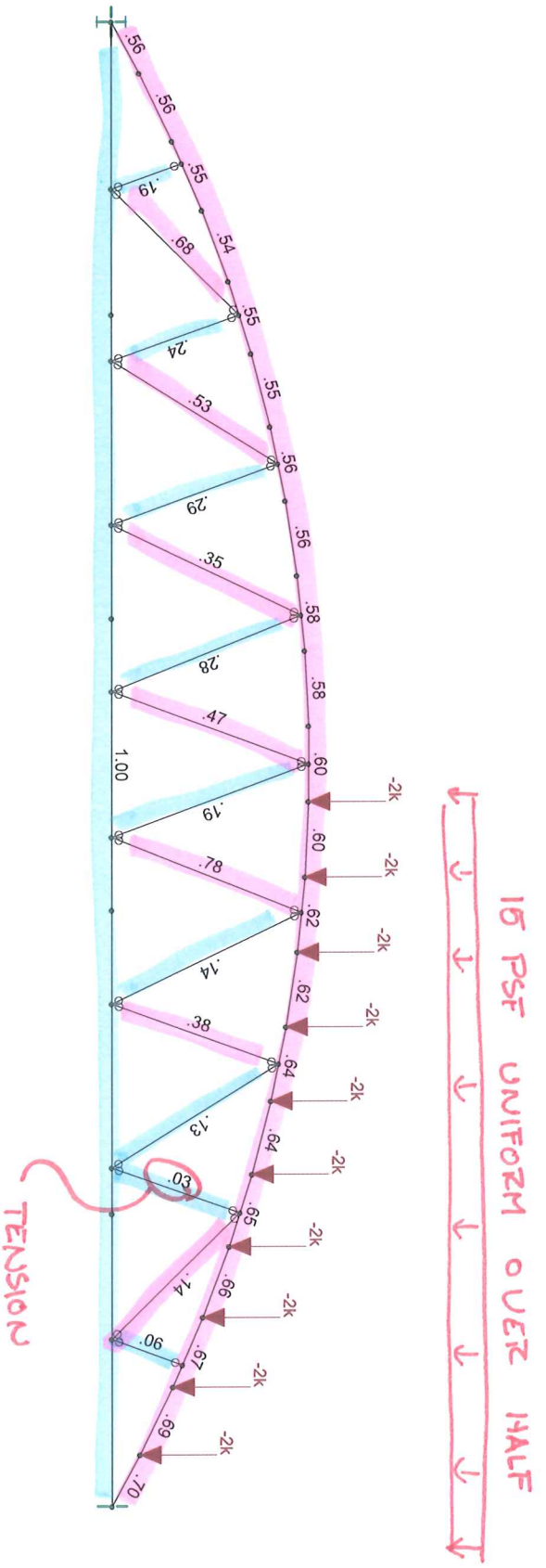
SK - 5

G.R. IRA Truss

Unbalanced Snow w/ Uniform Snow Load = 20 PSF

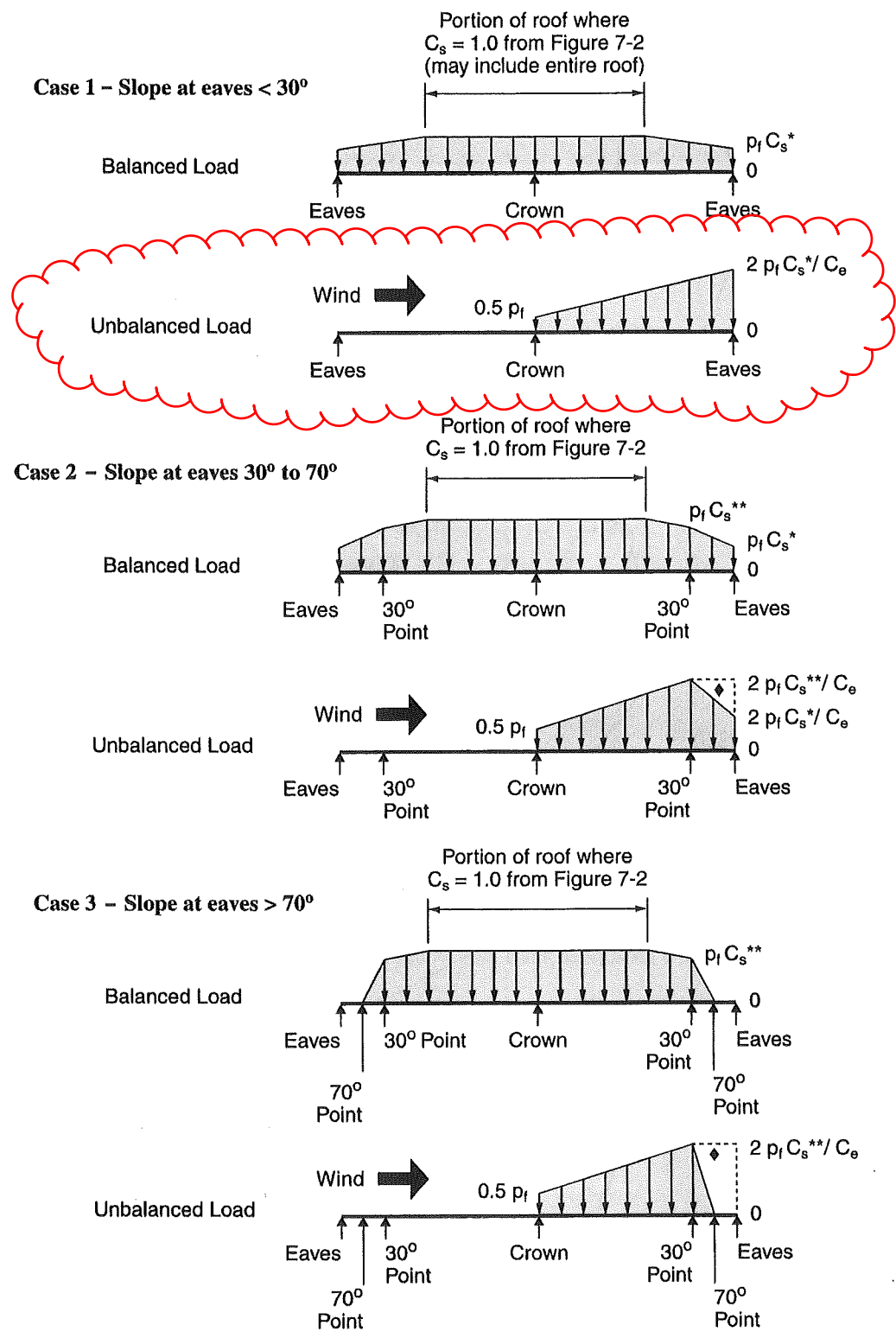
Oct 20, 2017 at 11:08 AM

17-665_Truss.r3d



Member Code Checks Displayed
Loads: BLC 5, Reduced Unbalanced
Results for LC 6, Dead + Unbalanced Snow Reduced

NCE		SK - 6
RTT	G.R. IRA Truss	Oct 20, 2017 at 11:10 AM
17-665	Uniform "Unbalanced" Snow @ 15 PSF Over Half of Truss	17-665_Truss.r3d



* Use the slope at the eaves to determine C_s here.
 ** Use 30° slope to determine C_s here.
 ♦ Alternate distribution if another roof abuts.

FIGURE 7-3 BALANCED AND UNBALANCED LOADS FOR CURVED ROOFS

STUART ANDERSON PROFESSIONAL ENGINEERING SERVICES INC.
35840 County Road 238, Deer River, MN 56636
tel. (218) 246-2396 fax (218) 246-2396 (auto receive)

DATA TRANSMITTAL:

TO: DALE ANDERSON DATE: 10-4-17
COMPANY: I.R.A. CIVIL CTR CONTRACT NO: —
ADDRESS: GRAND RAPIDS

FROM: Stuart Anderson, P.E.

SUBJECT: FORMER STRUCTURAL ENG. PROJECT
RECORDS

We are sending you herewith: THE ATTACHED 3 copies of the following:

Dwgs. — Prints — Reproducibles — Reports 3 Other —

Description or List:

- 1) ROOF TRUSS DIAGONAL MEMBER FAILURE (3/24/2001)
- 2) ENG STUDY RE: SPEAKE LOAD ON TRUSSES (8/25/2007)
- 3) NEW DUCT HANGER LOADING ON DECK MEMBERS (2/15/2017)

NOTE: ITEM 1) GIVES SOME HISTORY ON THE
STRUCTURES DESIGN (OR LACK OF DESIGN)

By: Stuart C. Anderson, P.E.
Stuart C. Anderson, P.E.

cc: file:

A STRUCTURAL REPORT on the I.R.A. CIVIC CENTER WEST VENUE ROOF'S BOW STRING TRUSS MEMBER FRACTURE

Located at Hwy. 38 & 14TH. St., Grand Rapids, MN

for

Mr. Dale A. Anderson, Mgr. IRA Civic Center
and the City of Grand Rapids

by

STUART ANDERSON PROFESSIONAL ENGINEERING SERVICE INC.

35840 Co. Rd. 238, Deer River, MN. tel. 218/ 246-2396

Ref. project C0114

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I hereby certify that this report and related calculations were prepared by me and that I am a duly Licensed Engineer under the laws of the State of Minnesota-



Stuart C. Anderson, PE.
Reg. No. 6721 Date: 3/24/01

STRUCTURAL ENGINEERING REPORT – GRAND RAPIDS I.R.A.CIVIC CENTER WEST VENUE ROOF TRUSS MEMBER FRACTURE

SUMMARY AND CONCLUSIONS:

At the request of Mr. Dale Anderson, Manager of the Itasca Recreation Association Civic Center, we made a site visit to the Civic Center within an hour of the request to evaluate the seriousness of an apparent structural fracture of a member of one of the west venue roof trusses. The purpose of our site review was to determine if the building was safe to allow the performance of the ice show, scheduled for that evening. We were to note conditions of the adjacent and related truss members, the effect of the fractured member on the stability of the truss in question and to determine the probable cause of the member failure, and possible remedial action to allow the continued safe and functional use of the building.

We met Mr. Rob McKay, acting City Engineer and arena Manager Mr. Dale Anderson at the site. Several men were on the roof removing the large drift of snow accumulated on the top and east side of the arch. The actual drift depth did not appear large enough to cause an overload problem (hip deep at the maximum), but the snow was apparently very dense with a high moisture content plus there was considerable ice at the bottom.

From the interior, we first observed the arch trusses all appeared sound without visible damage or noted excessive deflection. However, several of the diagonal members near the east quarter of the span from the center of the building toward the north quarter were bowed significantly (see Appendix A, page A1). We then noted the fifth arch-truss from the north end had its bowed diagonal fractured about three feet from its bottom chord connection.

With the effort of Hawk Construction in collaboration with the Grand Rapids Public Utilities Department, on very short notice the broken member was replaced with a temporary 4" x 4" post and three of the other most bowed diagonals were reinforced with screwed on 4 x 4 posts to stiffen each of them. The snow removal crew did a great job at relieving the unbalanced loading. Because of these efforts, we were able to allow the Ice Show to go on without danger of the roof being a hazard to the occupants. All of this work was completed within three hours of our receiving word of the problem. The Civic Center Maintenance staff, Hawk Construction and the Public Utilities department deserve commendation for their professional and expedient work.

The conclusions of this structural engineering review are:

- 1.) The actual roof snow load did not appear to be excessive in depth compared to normal roof overload that would create a threat to the primary wood arch structural system. The MN State Building Code for this area is 30 lb. per square foot of snow. It is probable that portions of the roof were subjected to a greater amount than this, but the over all average load on the roof appeared to be far less than its design load.
- 2.) The eccentric loading of the very dense snow drift apparently did create enough downward deflection to the arch on the east side so as to cause a line of web members, about forty feet from the east wall, to become bowed and distorted out of their axial plane. The apparent bow at the web member of the fifth-truss from the north end caused it to fracture about three feet up from its' bottom connection.
- 3.) The bowed web members were apparently intended to be tension members, since they were much thinner than their adjacent compression members. The reversal of stresses in them (from tension to compression) caused the unintended bow in them, greatly reducing their strength while in this distorted mode.
- 4.) The member that did fracture had a knots at two points of the fracture on a large piece found on the ice surface (see Appendix A, page A2). These knots further reduced the member's strength, hence the cause of the failure.

STRUCTURAL ENGINEERING REPORT – GRAND RAPIDS I.R.A. CIVIC CENTER WEST VENUE ROOF TRUSS MEMBER FRACTURE

HISTORY:

There is very little documentation on file regarding the "West Venue" building. The building was constructed by a non-profit community organization called the Itasca Recreation Association (IRA) in 1962. We contacted Mr. Robert Y. Sandberg, LA, who was active in the field of Architecture at that time to see what he knew of the project. We discovered he was the Architect for the IRA group. However, the IRA group acted as project managers, and Sandberg only prepared a minimal design layout that did not include the engineering or plan review of the arch truss roof. He has no documentation regarding it, as the owners did the bidding and purchasing of the roof members.

The trusses are stamped RILCO, and consulting with a former member of Structural Wood Co. of Minneapolis, we found out the RILCO firm is still in business in Albert Lee, MN. now under the name of ALAMCO. We contacted a Mr. John Foreman of ALAMCO (Tel 507-373-1401) and found out there is a 50-50 chance that they may still have drawings of that truss on file, but it will take some effort to find them.

We also discussed the truss work with Mr. Jack Goehl of Structural Wood Corp. (tel. 800-652-9058), who was familiar with this type of construction, as he had worked for RILCO shortly after the period when this structure was built. He also gave us the name of two engineering firms that are very familiar with this type of construction. These will be referred to later in the recommendations section of this report.

From what we have observed, the structure apparently served its functional purpose very well for the past 39 years. We heard some "here say" that there was a similar problem shortly after the building was opened, but found no verification of this.

OBSERVATIONS:

DOCUMENT OBSERVATIONS: No drawings and design details were available for our review. Mr. McKay stated none could be found on file with the city. It is highly unusual to have no record drawings for a building of this magnitude. However, due to the nature of the way the construction was performed under the control of a private group, the records were probably meager and never transferred to the city.

SITE OBSERVATIONS:

On our arrival at the site, we observed a crew on the roof removing snow with snow scoops. This appeared quite difficult due to the heavy density of the snow, the increasing slope of the curved surface and the iced roof surface.

From the interior, we observed a very visible bowing or lateral deflections of several of the web members in the vertical plane of the arch trusses (see attached photos in Appendix A). **The fifth truss from the north end had a broken member located just above the third bottom chord panel point from the east end.** The most severely bowed members were located at the same relative position, about 40 feet west of the east bearing, on several of the adjacent trusses.

It was apparent that the bowed members and the broken member were intended to be tension members in the arch truss for possible unbalanced loading on the arch. They were much thinner than the alternate "V" spaced web members, which were probably compression diagonals. Due to the observed bowing in them, it was also apparent they had become subject to compression forces, rather than the intended tensile forces. This was probably due to deflection of the arch caused by the partial roof loading at the limited area of heavy snow drift observed on the exterior.

Since no design or fabrication drawings were available, we made the following measurements while at the site that day and the following day. The building has a circular curved arch roof consisting of wood trusses spaced twenty feet on center that span 132 feet between the

STRUCTURAL ENGINEERING REPORT – GRAND RAPIDS I.R.A.CIVIC CENTER WEST VENUE ROOF TRUSS MEMBER FRACTURE

supporting masonry walls. The roof consists of a rubber membrane roofing over unknown insulation that is supported on probable two or three inch thick wood tongue and groove decking. The decking spans diagonally between 3" x 13" wood purlins that are spaced seven feet on center, These transmit the roof load to the upper chord of the wood arch.

These arch trusses are termed "bowstring arches" because their bottom tie beam (lower chord) combined with the upper circular arch (top chord) resemble a strung bow. Web members arranged in a series of "V" configurations attach the top arch chord to the bottom tie beam. The bottom of the arch, or tie beam chord is a wood beam consisting of several plies of laminated 2" x 6" boards for a total assembly 5 1/4" wide by 11 1/4" deep. We sonically measured the bottom chord clear height from the floor ice to be 23'-5". The upper curved or arch chord member is an identical 5 1/4" x 11 1/4" laminated beam. We sonically measured the height from the floor ice to the middle or high point of the roof deck is 42'-2".

We observed some dark discoloration on the underside of the roof deck boards near the middle of the second arch from the north. There was also a large white patch visible in the same area. The white appeared to be frost from moisture condensation. The dark stains might indicate staining and possible rot from previous leakage or condensation moisture saturation.

CALCULATIONS and REMEDIAL ACTIONS:

No calculations were performed at this stage of the investigation, as there was inadequate information on the existing structural configuration to accurately perform an analysis.

We discussed the problem with Mr. McKay and Mr. Anderson. Our conclusions were 1.) continue removing the large snow drift from the roof, 2.) replace the fractured web member, 3.) stiffen the worst of the bowed members to prevent their potential future fracture, and 4.) if the above could be completed before evening, then allow the performance to go on.

We contacted Mr. Doug Hanson, Gen. Mgr. of Hawk Construction Co., and advised him of the problem. He came to the site within a few minutes and alerted a crew, who was on the site within a half hour. Fortunately Mr. Tony Ward, General Mgr. of the Grand Rapids Public Utility Department was on the site as a volunteer helping to set up the ice show. The arena mobile lift platform used to service lights etc. would only raise to about 22 feet, so within a short time, Mr. Ward had a PUC truck with an operator and a lift bucket that would reach 38 feet on the site.

In less than two hours, all repairs were made and the majority of the problem snow drift was off the roof. The ice show went on as scheduled.

REVIEW AND RECOMMENDATIONS:

We reviewed the structure on site and found no available documents. We observed material conditions and made limited measurements of the structural components and estimated snow depth in the areas of concern. We video taped our observations. Our conclusions are listed at the beginning of this report. Our observations lead us to the following recommendations:

1. The basic structure is now very close to the strength and load carrying condition it had prior to the fracture because of the repair by Hawk Construction personnel with the aid of the lift truck and operator furnished by the Grand Rapids Public Utility Dept. The 4" x 4" replacement post for the fractured member is somewhat less strength than the original 3 x 6 member. **We recommend that this temporary 4" x 4" member be replaced some time this summer with a 3" x 6" member identical to those at the same location on other trusses.** Replacement member shall be Douglas Fir-Larch, select structural grade.
2. We recommend that close inspection of the roof deck where we noted frost and dark stains, to determine the soundness of the wood deck in that area. A man in a lift unit

**STRUCTURAL ENGINEERING REPORT – GRAND RAPIDS I.R.A.CIVIC CENTER WEST
VENUE ROOF TRUSS MEMBER FRACTURE**

could probe the wood with an ice pick or similar tool to determine if the wood is sound. If not, the upper exterior roofing should be investigated for leaks and/or insulation gaps.

3. The roof structure appears sound and serviceable, but our evaluation is limited from our ground level observations. The building has performed a functional service for 39 years. This flaw occurred with less than a recognized overload on it. Because of the building's age and condition of the event, we recommend that a more thorough, close up inspection be made by a qualified Structural Engineer of all of the wood bowstring arch trusses to determine serviceability for the next 30 years.

Item 1.), above, can be performed by any local contractor with access equipment, such as Hawk Construction Co. or even "by in house" staff maintenance carpenters, if access is available.

Item 2.) above could be performed by any in house person that can gain access to the area in question. By first probing a sound area and comparing it to a probe or series of probes at the area in question, it is easy to feel if rot or decay has occurred. If so, further action is required.

Item 3.) above could be done by the writer, but we would like to recommend the City contact one of the following instead, due to their apparent long time familiarity with this type of construction. They were recommended to us by Mr. Jack Goehl of Structural Wood Corp.

- 1.) Mr. Richard Eckroad PE., 10130 36th PLACE n., Plymouth, MN. 55441, (tel. 763-544-8599) or,
- 2.) Enterprise Engineering Corporation, 710 French St. P. O. Box 163, Peshtigo, WI. 54157-4501 (tel. 715-582-4501)

We are advised that both of the above are specialists in wood construction and arch trusses of this type. Such further review of the building upper structure should be a wise investment, as they may or may not find places for remedial work or replacement. Such investment would be minor, considering the cost of replacement of a structure of this type.

The snow removal crew, Hawk Construction and Mr. Tony Ward and his PUC crewman deserve credit for an expeditious job cooperatively well done.

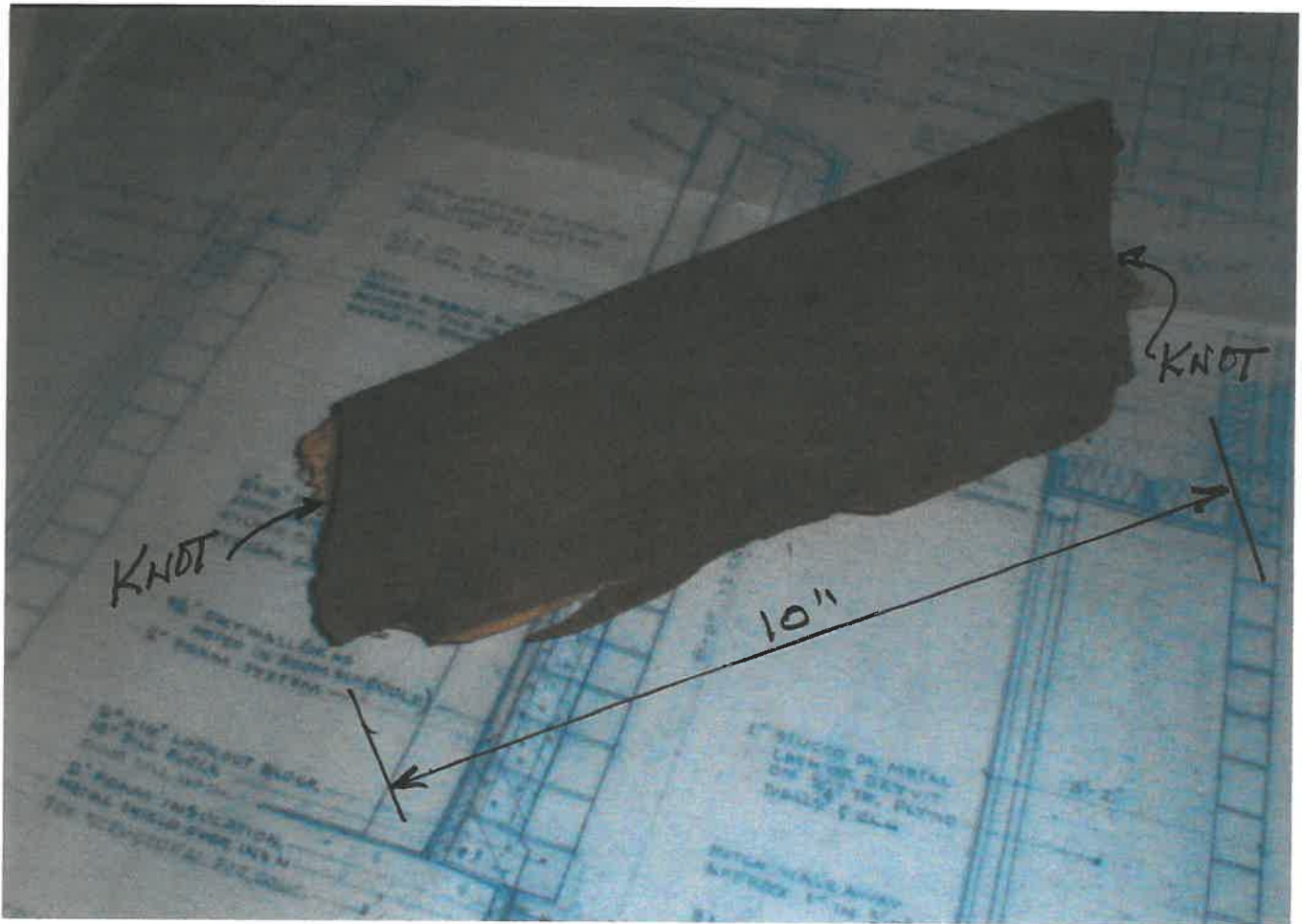
The conclusions of this report represent our professional opinions. These opinions are based on the limitations of observable items and our assumptions regarding the materials and procedures used in the construction. These conclusions are also based on our research, limited analysis of components, experience and judgment regarding comparable material and conditions for the construction.

The structural engineering services performed for this project have been conducted in a manner consistent with that level of skill and care ordinarily exercised by other members of the profession currently practicing in this area under similar budgetary and time constraints. No other warrantee, express or implied, is made.

This report represents our completion of this project, based on our understanding of the scope of services requested. It is presented for the exclusive use of the City of Grand Rapids, MN.

END OF REPORT





STUART ANDERSON PROFESSIONAL ENGINEERING SERVICE INC.

35840 Co. Rd. 238, Deer River, MN. 56636 Tel. 218/ 246-2396 Fax same /auto switch

August 28, 2007

Mr. Dale Anderson, Mgr.
IRA Civic Center Arena.
Hwy 38 & 14th. St.

Grand Rapids, MN.

Subject: **Structural Engineering Review** regarding the Fall Fest installation of 5,100 lb. of speakers and accessories onto one of the main roof trusses.

Dear Mr. Anderson:

We meet today at the IRA Civic Center (West Venue) to evaluate the existing roof truss for the subject loading. At that time, considering the small amount of added load(5,100 lb.) in comparison to the possible snow loading (78,000 lb.) my first impression, which was relayed to you, was that those audio equipment loads would probably be of no concern for overloading the truss. Upon further analysis though, I find the assembly of speaker loadings as proposed by the exhibitors could cause an overstress in the existing single bolt connection at panel point 3. That is where the truss diagonals and the lower chord connection plates are joined by a single bolt. That bolt has to transfer the one diagonal loads through the truss, and back into the other diagonal to transfer shear forces, and to allow the chord member to accept any unbalanced horizontal forces in the two diagonals.

A computer model of the lower chord member with the proposed audio equipment located closely fitting the proposed installers plan (see attached) indicates the single $\frac{3}{4}$ " diameter cross bolt is stressed in bearing against the wood grain, slightly over the allowable limit. Now if that were the only forces on that bolt, it would probably be OK to proceed, assuming the overstress is marginal for a short time period of application.

However, since the design drawings are apparently not available, it is not possible to accurately evaluate the present truss to determine what the present normal forces are on that bolt and it's wood interface. Therefore it cannot be determined how much those two combined forces would be in excess of what the code would allow.

In view of this, I recommend that only one of the 900 lb. loads be permitted to be adjacent to any single truss lower chord panel point. Now if those two 900 lb. units are a single assembly and they cannot be separated, it may be necessary to rig a cross beam between two trusses, hang the two 900 lb. loads at mid span to reduce the loads by about 50%.

Yours truly,



Stuart C. Anderson PE

cc: file N0712-0823rpt-ltr

STRUCTURAL ENGINEERING MEMORANDUM

DATE: February 13, 2017

PROJECT C1703

To: Mr. Otto Maki

The Design Group

email: otto@thedgroup.org

SUBJECT: Structural engineering evaluation of the Grand Rapids I.R.A. Civic Arena West Venue Roof for the installation of a heavier air duct to the roof underside.

At the request of The Design Group's Mr. Otto Maki, we visited the subject structural bowed roof site to observe conditions and to take some measurements of the roof structure framing. The purpose of our site review was to determine if the present roof structure can safely support the loading of the new ductwork, which is slightly heavier than the existing duct.

SUMMARY AND CONCLUSIONS: We observed the subject building's roof structure, took limited measurements and performed load and stress analysis on limited members. **Based on our observations and analysis data, it is our professional opinion that the present roof structure of the West Venue of the I. R. A. Civic Center can carry the proposed load of the new duct work as proposed by The Design Group.** See analysis comments below.

OBSERVATIONS: The Grand Rapids I.R.A. Civic Arena West Venue roof is supported by wooden Bowstring Arch Trusses that are 20 feet on center and span 132 feet over the arena. Wood purlin beams (3 1/2" x 13") spaced about seven feet on center, span between the truss top chords. They in turn carry the roof decking that spans between the purlins at a 45 degree skew. The decking was 5" T & G of unknown thickness and lumber grade.

ANALYSIS: Our calculations and review of the AITC Timber Constr. Manual indicated that nominal 2" deck would be overstressed even for the 30 psf. roof snow load of the 1962 era the building was constructed. From this, we conclude the deck must be at least a minimum thickness of 3" nominal (2 1/2", 2 5/8") thickness, and our calculations and the AITC tabular data indicate it should safely carry the new duct load. We recommend the hanger eye bolts be a minimum of 1/4" and screwed in to a minimum depth of 1 3/4".

Those purlins are stressed pretty close to their limits, so we recommend that the duct hangers or their locations that are parallel to the arch trusses be limited in position to the purlin beam span be near the span quarter points, away from the middle of the purlin span.

The conclusions of this report represent our professional opinions. They are based on the limitations of observable items regarding the materials and procedures used in the construction. Our conclusions are also based on our research, experience, assumptions and judgment regarding comparable material and conditions of the construction.

The civil, structural and foundation engineering services performed for this project have been conducted in a manner consistent with that level of skill and care ordinarily exercised by other members of the profession currently practicing in this area under similar budgetary and time constraints. No other warrantee, express or implied, is made.

This report represents our completion of this project, based on our understanding of the scope of services requested, It is presented to the exclusive use of The Design Group & Grand Rapids City.

STUART ANDERSON PROFESSIONAL ENGINEERING SERVICES INC.

35840 Co Rd 238, Deer River, MN. 56636 tel. (218) 246-2396 fax same with auto switch

I hereby certify that this report-memo and related calculations were prepared by me and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

Stuart C, Anderson, P.E

Date 2/13/2017 Reg. Number 6721

November 9, 2017

Mr. John Erickson
DSGW Architects
2 West First Street, Suite 201
Duluth, MN 55802

Sent by Email to: jerickson@dsgw.com

**Re: Grand Rapids IRA Civic Center – West Venue Roof – Truss Web Failure
NCE Job No. 17-665**

Dear John,

This report is a follow-up on our previous report dated 11-06-17 and our discussions and observations from our visit yesterday.

We visited the facility on Wednesday 11-8-17 to attend a meeting to discuss the results of our analysis. Those results show that a 15 psf unbalanced snow load will cause the truss to be at design capacity and that with 20 psf of uniform snow loading, the roof trusses are at or above the design capacity. Note: 15 psf is approximately 8" to 10" of normal weight snow or maybe 6" of heavy wet snow.

When we arrived, we walked the east bleachers and discovered a failed member of the 5th truss from the north end of the building. Based on our analysis, the failed member is one of the members most effected by an unbalanced loading condition.

We directed the manager to get the truss shored up immediately and to get the truss member replaced.

After the meeting, we observed the roof snow loading from the adjacent roof and noted that only a few inches of snow appeared to be on the roof. We also observed the truss again and noted that the adjacent web members were bowed significantly and appear to be in danger of failure. We directed the Manager to close the West Venue and to shore the truss.

This report is a follow-up on that conversation and our previous report.

Recommendations and Opinions:

1. Shoring: The top and bottom chord of the truss should be supported from the ground surface until the repair is made. It is not sufficient to shore only the bottom chord.

2. Since very light snow loading appears to have caused this failure and the failure appears to have nearly caused the collapse of a roof truss that could have endangered the entire facility and the occupants, it is our professional opinion that some long-term deterioration of the wood capacity has occurred. Based on our discussions with NRR1 this morning, this deterioration can be due to long term alternating wet and dry environments that can affect the fiber strength of the material.

3. Since the design capacity is extremely deficient as noted in our previous report, any deterioration of the capacity of the wood members is extremely concerning in our opinion.

4. In our professional opinion the West facility should not be used in the winter months unless / until the building official, building ownership and other parties agree on temporary measures to protect the public and adjacent structures.

We are available for the above recommended discussions and would appreciate the opportunity to find solutions to this issue. (see attached photos)

Sincerely,



Jon E. Aamodt, PE
Principal Partner

Professional Certification:

I hereby certify that this plan, specification or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.



Jon E. Aamodt, P.E.
MN Reg. No. 24838

Date 11-09-17



November 15, 2017

Mr. John Erickson
DSGW Architects
2 West First Street, Suite 201
Duluth, MN 55802

Sent by Email to: jerickson@dsgw.com

**Re: Grand Rapids IRA Civic Center – West Venue Roof – Truss Web Member Repair
NCE Job No. 17-665**

Dear John,

This report is intended to follow up on our previous reports dated 11-06-17 and 11-09-17 and our discussions discussions with Matt Wegworth (City Engineer) and Travis Cole (City Building Official) yesterday. Also included is a requested detail for properly replacing the failed truss member discussed in our 11-09-17 report.

The Original report summarizing our analysis of the roof structure dated 11-06-17 stated that the existing roof design capacity of the west venue wood roof structure was determined to be approximately 20 psf based on member sizes and like new condition of the members. We stated that we consider this condition to be dangerous and should be corrected as soon as possible. Since the roof has adequately performed for over 50 years, we recommended that the snow load be controlled to less than 1 foot, that consideration be given to posting notice of the diminished capacity and to cancel events during a significant snow event.

The follow up report dated 11-09-17 discussed the issue regarding the failure of a significant truss web member under minimal loading. The failure apparently occurred between 10-24-17 and 11-08-17 with no large snow event and only approximately 3 inches of snow on the roof when we visited. In this letter, we concluded that the failure could potentially be a result of the wood being somewhat weakened by long term moisture changes in the material and that our confidence in the calculated capacity is diminished by the partial failure. This letter recommended that the truss be shored, and that the facility not be used unless and until the building official and the building ownership can agree to stringent maintenance procedures to adequately protect the public. In other words, we did not further prescribe these further procedures but leave this to the building official and the management to work out knowing the potentially very weak condition.

Based on our discussions and further review of the failure. We determined that the new web failure did occur in the same truss as a previous failure and repair in 2001. We also noted that the previous failure was in an adjacent web member. Since it is possible that the diminished capacity of the member could also be due to incorrect procedures in the previous repair, we informed the building official and staff of this possibility. This information along with their internal discussions and planning has apparently led them to instruct NCE to provide a repair detail for the broken web member.

Attached to this letter, please find our repair detail for the failed member. In our opinion, it is possible that this failure is partially due to improper repair in the past, but it is also possible that the failure under light loading is partially or completely due to reduced capacity due to long term moisture changes. Therefore, as we discussed, in our professional opinion:

1. The public should be informed and protected from snow loading on the roof as determined by the building official and the ownership with the knowledge that a partial failure has occurred with less than 6" of snow on the roof. The procedures for doing so are to be determined by the ownership and building official. Note: unbalanced loading (loading on one side of the roof only) is of particular concern as it causes members designed for tension to be placed into compression.
2. The repair of this member, if properly installed per the drawing, meets or exceeds the capacity of the original member, but does not improve upon the overall capacity of the truss or overall roof structure.
3. The venue should be closed during a significant snow event, with the understanding that a partial failure has occurred with less than 6" of snow on the roof.

Sincerely,



Jon E. Aamodt, PE
Principal Partner

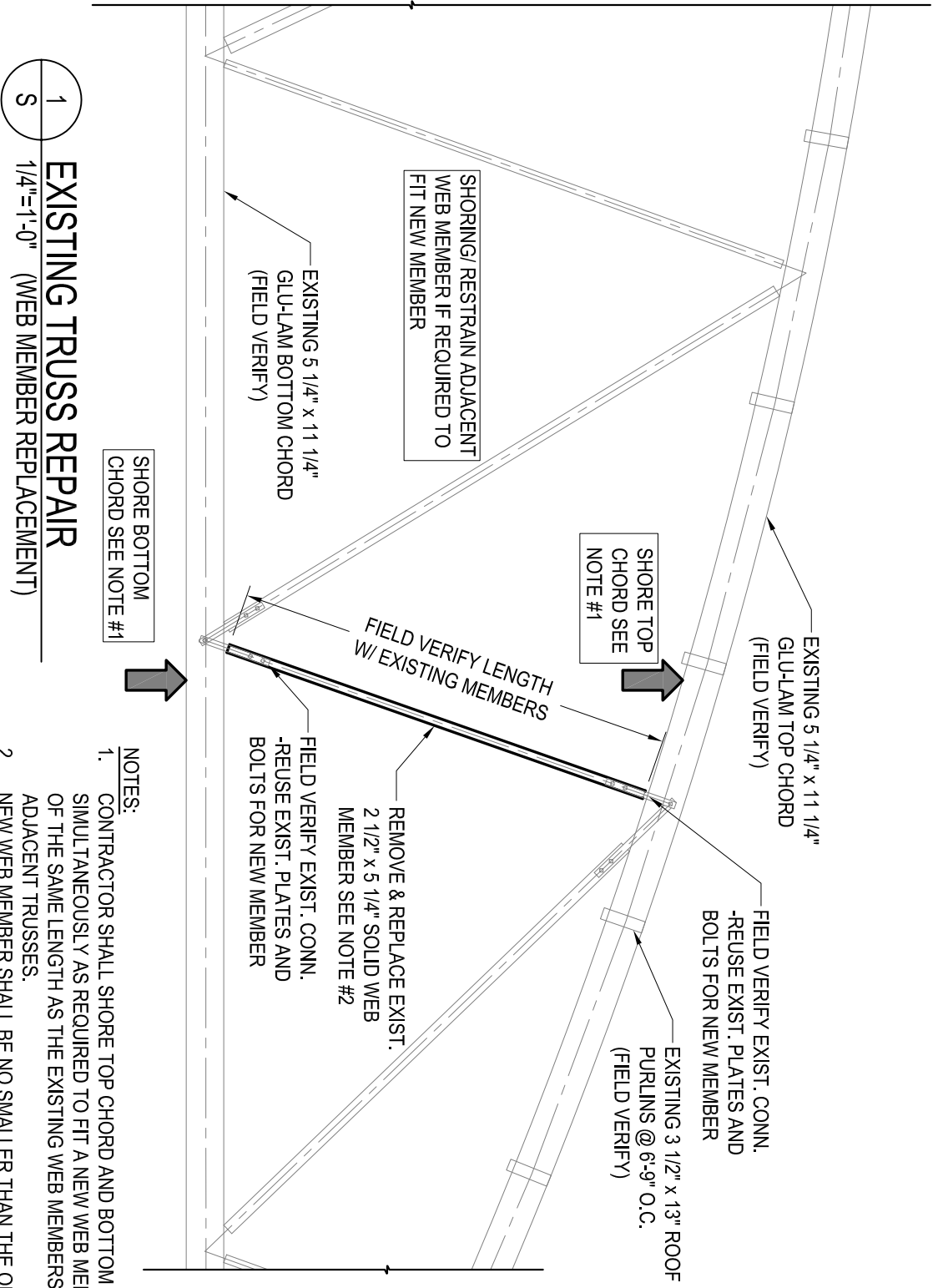
Professional Certification:

I hereby certify that this plan, specification or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.



Jon E. Aamodt, P.E.
MN Reg. No. 24838

Date 11-15-17



1
EXISTING TRUSS REPAIR
 1/4"=1'-0" (WEB MEMBER REPLACEMENT)

SHORE BOTTOM
 CHORD SEE NOTE #1

SHORE TOP
 CHORD SEE
 NOTE #1

- NOTES:**
- CONTRACTOR SHALL SHORE TOP CHORD AND BOTTOM CHORD SIMULTANEOUSLY AS REQUIRED TO FIT A NEW WEB MEMBER OF THE SAME LENGTH AS THE EXISTING WEB MEMBERS IN ADJACENT TRUSSES.
 - NEW WEB MEMBER SHALL BE NO SMALLER THAN THE ORIGINAL MEMBER AND SHALL BE D.F. #1 OR BETTER. ROSBORO X-BEAM GLU-LAM OF SIMILAR DIMENSIONS MAY BE USED.