BAR ENGINEERING

May 21, 2024 | Project No.: 24VR-600400

City of Salmon Arm
500 2 Avenue Northeast
Salmon Arm, BC
V1E 1J5

Phone: 250.803.4088
Email: dgerow@salmonarm.ca

Attn: Darin Gerow
Re: Structural Assessment & Life Cycle Assessment of the Memorial Arena – Final Report
100 - 30th Street SE, Salmon Arm, BC

BAR Engineering Co. Ltd. is pleased to submit this final report of the life cycle assessment of the Memorial Arena to the City of Salmon Arm.

This report includes the asset inventory condition assessment, rehabilitation and renewal forecast, cost estimates for demolition and replacement, and recommendations.

Should you have any questions regarding the contents of this report, please do not hesitate to contact the undersigned at (250) 541-9590 or via e-mail at joey.funk@bareng.ca.

Respectfully Submitted,
BAR Engineering Co. Ltd.

Per:

Joey Funk, P. Eng.
Senior Structural Engineer

Structural Assessment and Lifecycle Analysis of the Memorial Arena

Final Report

Engineering services for

City of Salmon Arm

May 21, 2024

Project No. 24VR-600400

City of Salmon Arm | **500 2 Avenue N.E. Salmon Arm, BC V1E 1J5**

Attn: Darin Gerow –Manger of Roads & Parks, City of Salmon Arm

Contact: Joey Funk, P.Eng.
Senior Structural Engineer

joey.funk@bareng.ca | 250.541.9590 | BAReng.ca
201-2540 53 Avenue Vernon, BC V1T 9W8

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Appendices

1.0 Introduction

1.1 Purpose

BAR Engineering Co. Ltd. (BAR) has prepared this lifecycle analysis of the Memorial Arena located at 351 – 3 Street SW in Salmon Arm, BC, for the City of Salmon Arm. The purpose of this life cycle analysis report is to identify the building components and their current conditions, estimate current and future costs of replacement, repair, and rehabilitation, and provide a recommendation on the most cost-effective manner of maintaining the Memorial Arena over time. More specifically, the following items have been included:

- Interim Report, issued March 25, 2024.
- Asset Inventory and Condition Assessment
  - Identification and description of each item or component.
  - Asset quantity, make, model, size, serial number, if applicable.
  - Asset condition.
  - Asset criticality.
  - Estimated installation year.
  - Estimated replacement cost.
  - Estimated remaining useful life.
- Rehabilitation and Renewal Forecast
  - Building component depreciation analysis to forecast future costs of building asset replacements, repairs, and rehabilitations.
  - Priority ranking of repairs.
  - Ultimate retirement date for the arena.
- Lifecycle Cost Estimate.
- Demolition and Replacement Cost Estimates.

1.2 Report Organization

This report has been organized as follows:

- Building Background: A summary of the building history and construction materials.
- Methodology: A summary of the processes used by BAR to complete the lifecycle assessment of the facility.
- Asset Inventory and Condition Assessment: A summary of the asset inventory and condition assessment.
- Rehabilitation and Renewal Forecast: A summary of the rehabilitation and renewal forecast.
- Demolition and Replacement Cost Estimates: A summary of the estimated costs to demolish the facility to green field conditions and a full replacement of similar size and function.
- Recommendations: Recommendations for the Memorial Arena.
- Conclusion: A summary of the report in its entirety.

2.0 Building Background

The Memorial Arena is located at 351-3 Street SW in Salmon Arm, BC. The facility has a footprint of approximately 26,500 square feet. It was built in 1957 with additions added to the East and West of the building in 1961 and 1966, respectively. A mezzanine supporting an additional office and additional storage was constructed on the east end of the arena around 1975. Significant fire and life safety upgrades were completed in 1991 including the removal of fixed spectator stands, installation of emergency exits, and installation of a fire alarm system.

The original building, the Arena, includes the artificial turf playing field and has an approximate footprint of 18,000 ft^2. The East Addition is approximately 3,350 ft^2 and includes a kitchen, reception area, washrooms, and offices. The West Addition is approximately 3,000 ft^2 and includes storage rooms, a workshop, a truck bay, two washrooms, and offices. The mezzanine, constructed in 1975, is approximately 1,250 ft^2 and includes additional office space and storage.

The ice plant was decommissioned in 1999 and the ice sheet converted to an artificial turf playing field. The ice equipment room was converted into office space and the change rooms were converted into a woodshop and additional storage.

For the purpose of this report, the Arena refers to the original building, and the east and west additions are referred to as the East Addition and West Addition, respectively.

2.1.1 General Construction

The general construction of the Arena, East Addition, and West Addition can be reviewed in Appendix I -Interim Report, Section 1.3.1.

3.0 Methodology

This section describes the approach and methodologies used to conduct the life cycle analysis (LCA). It includes information on the time horizon considered, rating and ranking systems, cost estimates, and any assumptions made during the analysis.

The objective of this LCA is to assess the financial implications associated with the repair, rehabilitation, and end-of-life costs of the Memorial Arena over an assumed remaining life span of 30 years. This analysis aims to provide stakeholders with comprehensive insights into the long-term economic viability of the asset and inform decision-making processes related to its management and investment.

3.1 Limitations

The scope of the LCA has been limited to the following cost categories:

- Replacement and Rehabilitation Costs: Covering expenditures related to component replacements, repairs, and upgrades, to maintain or improve the functionality and performance of the facility.
- End-of-Life Costs: Encompassing the costs associated with decommissioning, and disposal of the facility at the end of its useful life.

The initial construction costs of the facility have not been considered in this analysis. Due to the age of the structure it can be assumed that the structure has returned its initial construction costs through services provided throughout its lifespan. Additionally, the cost of replacing the facility has not been included in the life cycle analysis as a direct replacement of the facility is not guaranteed and the cost of replacement could vary substantially. An estimate of current replacement costs has been included within the report.

3.2 Data Collection

Data collection was completed by means of reviewing historical data provided by the City of Salmon Arm, discussion with stakeholders, and multiple site assessments.

3.2.1 Historical Data

A detailed summary of the historical and background data provided by the City of Salmon Arm has been included in the Interim Report. Please refer to Section 2.1 in Appendix I – Interim Report

3.2.2 Site Assessments

The on-site assessments were executed by Joey Funk, P. Eng. and Chris Thornton, E.I.T. from February 2024, to March 2024.

The assessment included a visual review of the building components during which an inventory of all building components and assets was collected. The assets were recorded using the ASTM Uniformat II Level 4 classification.

3.3 Component Evaluation

The prioritization of building components and assets for repair or replacement was determined utilizing the building's risk matrix provided by the City of Salmon Arm. Following the site assessment each component and asset were assessed by determining a condition rating, capacity rating, and criticality rating based on the risk matrix outlined below.

The prioritization score for each component was computed by calculating the average product of the asset condition and criticality, and the asset capacity and criticality. This scoring methodology yields a numerical value ranging from 1 to 25, with a score of 25 indicating components or assets that urgently require attention due to insufficient capacity and inability to fulfill intended functions.

BUILDINGS AND FACILITIES

Condition	Failing	Poor	Fair	Good	Excellent
	5	4	3	2	1
	Incapable of performing intended use	Requires immediate repairs in order to continue performing intended use	Requires some maintenance or repairs in the near future	Some maintenance and no repairs required in the forseeable future	Typical operations maintenance and no repairs required in the forseeable future
Consequence	High	Med-High	Medium	Med-Low	Low
	5	4	3	2	1
	Refer to consequence rating on SA Risk Framework				
Capacity	Failing	Poor	Fair	Good	Excellent
	5	4	3	2	1
	Incapable of supporting required capacity	Incapable of supporting existing desired capacity or incapable of supporting future capacity	Capable of handling existing capacity requirements, but at greater than 80% full; Future capacity issues noted	Capable of handling existing capacity requirements at less than 80% full; no future capacity issues noted	Capable of handling existing capacity requirements at less than 50% full; no future capacity issues noted
Consequence	High	Med-High	Medium	Med-Low	Low
	5	4	3	2	1
	Refer to consequence rating on SA Risk Framework				

Criticality
1 Facility service levels not impacted without asset
2 Minor service level disruption without asset
3 Moderate service level distruption without asset
4 Facility usage severly limited without asset function
5 Facility cannot provide service without asset

Figure 1: Buildings Risk Matrix [1]

Additionally, components were assessed based on the consequence of failure using the City of Salmon Arm Risk Framework. Each component was assigned a risk consequence ranking on a scale of 1 to 5, where 1 signifies insignificant consequences and 5 denotes catastrophic repercussions. The complete risk consequence ranking rubric is detailed below for clarity and consistency in the evaluation process.

Risk Consequence

Consequence	Injury	Service Interruption	Environment	Finance	Reputation	Legal/Regulatory
Insignificant (1)	Nil	< 4 hrs	Nil	<$20k	Nil	No Impact
Minor (2)	First Aid	Up to 1 day	Minor short term	$20k-$100k	Minor (local) Media	Visit from Government Body
Moderate (3)	Medical Treatment	1 day - 1 week	Wide short term	$100k - $500k	Moderate (Provincial) Media	Warning from G.B.
Major (4)	Disability	1 week - 1 month	Wide long term	$500k - $1M	High (national/ international) Media	Fine from G.B.
Catastrophic (5)	Fatality	More than 1 month	Irreversible long term	> $1M	Censure/ Inquiry	Loss of License

Figure 2: Risk Consequence [2]

3.4 Cost Estimates

DGH Engineering Ltd. (DGH) provided cost estimating services for the asset replacement and repair costs, facility demolition to greenfield site costs, and facility replacement costs.

DGH has prepared a Basis of Estimate which has been included in Appendix III. The Basis of Estimate includes the following three cost estimates:

- Appendix A – Demolish and Convert to Greenspace
- Appendix B – Equivalent Replacement
- Appendix C – Enhanced Replacement

End-of-life costs involve the anticipated expenses associated with decommissioning and demolishing the facility. This includes costs for disposal, including hazardous materials removal, and earthworks required to restore the site to greenfield conditions. These calculations are essential for understanding the full life cycle cost implications and ensuring responsible asset management.

4.0 Asset Inventory and Condition Assessment

The asset inventory and condition assessment has been prepared in table format (excel). The table has been attached in Appendix II. Please refer to columns 1 through 29 for the asset inventory and condition assessment.

The asset photos referenced in column 10 have been provided in Appendix IV.

The criticality score for each asset has been calculated by averaging the product of an asset's capacity and criticality, and an asset's condition and criticality. The criticality scores are provided in column 16.

The risk consequence for each asset is included in column 24.

In some cases, assets with low criticality scores are part of assemblies (roof, wall, floor, etc.) with high criticality scores, and repair or replacement is necessary to address the critical assets.

5.0 Rehabilitation and Renewal Forecast

The rehabilitation and renewal forecast has been provided in the second half of the life cycle spreadsheet in Appendix II. The forecast spans 30 years starting in 2024 and ending in 2054, columns 35 through 65.

The type of recommendation for each asset is provided in column 15. Assets requiring replacement have the replacement completed during the recommended replacement year provided in column 23. The replacement cost in the replacement year is based on the current asset replacement cost, column 34, adjusted for an annual inflation rate of three percent (3%). Assets requiring maintenance have cash allowances carried every 5 to 10 years.

The current replacement value of each asset has been provided in column 30. The costs include 7% consultant fees, 5% tax, disposal fees, and a 25% contingency. Since the sub-total current replacement values, column 30, are based on Class D estimates, they inherently include a 25% contingency and therefore the contingencies have not been sub-totaled separately.

Disposal fees are based on current landfill dumping fees for the City of Salmon Arm Landfill. No dumping fees have been included for assets with negligible dumping fees. Assets made of metal will be sold for scrap and have no dumping fees.

The following table summarizes the anticipated immediate, short-term, medium-term, and long-term expenditures for the proposed renewals and rehabilitations over the remaining life of the facility. The expenditures have been adjusted for an annual inflation rate of 3% and include the demolition cost in year 30, the retirement date of the facility.

Immediate Expenditures	$	2,933,712.06
Short-Term Expenditures (1-5 Years)	$	158,987.73
Medium-Term Expenditures (6-10 Years)	$	529,587.47
Long-Term Expenditures (11+ Years)	$	799,000.28
Demolition Cost in Year 30	$	2,410,817.77
Total Expenditures (2024 to 2054)	$	6,832,105.31

Table 1: Probable Cost Table for Proposed Renewals and Rehabilitations.

6.0 Demolition and Replacement Cost Estimates

The demolition and replacement cost estimates have been prepared by DGH. Please refer to Appendix III for the Basis of Cost Estimate and the cost estimates for the demolition to greenfield site, equivalent replacement, and enhanced replacement.

The following table summaries the demolition and replacement costs prepared by DGH.

Demolish and Convert to Greenspace	$	993,225.00
Equivalent Replacement	$	7,577,012.72
Enhanced Replacement	$	9,106,592.75

Table 2: Demolition and Replacement Cost Estimates

7.0 Recommendations

The Interim Report, attached in Appendix I, includes two repair recommendations for the Memorial Arena: Partial Occupancy Repairs and Full Occupancy Repairs.

The partial occupancy repair option includes limited repairs to permit occupancy of the facility between March and November and occupancy restrictions during roof snow accumulation and large windstorms. The anticipated immediate expenditure of this option is $89,700 and is expected to extend the service life of the facility for 3 to 5 years.

The full occupancy repair option substantially aligns with the repair and rehabilitation forecast included in the life cycle assessment. This option would extend the service life of the Facility by 30 years with an ultimate retirement date of 2054. The anticipated immediate expenditures to obtain full occupancy of the building is just shy of $3,000,000. The main factors contributing to the large upfront costs include the foundation, exterior wall, and building envelope replacement of the Arena; reinforcing of the Arena roof system to support current snow loading; and installation of a fire suppression system throughout the facility. In other words, the immediate expenditures will go towards 'keeping the facility standing', with limited rehabilitations of amenity areas in the East and West Additions.

The life cycle analysis considered an ultimate retirement date in the year 2054, with total expenditures exceeding $6,800,000 over the next 30 years. Nearly 50% of the total expenditures occurring immediately, 35% to demolish and return the site to greenfield conditions in year 30, and the remaining costs to maintain the facility between now and retirement of the building. Expenditures to fully rehabilitate the amenity areas and provide aesthetic upgrades to the facility interior have not been included in the repair and rehabilitation forecast.

It is the opinion of the undersigned that rehabilitating the building envelope and structure with the intent of extending the useful service life of the facility is not a financially feasible solution and demolition to a greenfield site or replacement should be considered. This opinion is based on the extensive effort and cost to replace the existing foundation and the flat roof sections of the Arena, the extensive Arena truss repairs, and the replacement of the building envelope. Furthermore, the extensive remediation will trigger the requirement to install a fire suppression system and upgrade the existing building to current codes in relation to fire and life safety.

Consideration should be given to implementing the partial occupancy repairs should the city decide to replace the facility. Facility planning, design, and funding can be expected to take 4-6 years, during which the existing facility could be partially occupied to host community events and programming. Demolition of the existing facility would commence only once the City of Salmon Arm was prepared to proceed with construction of the replacement facility. This option would provide the residents of Salmon Arm and surrounding communities with intermittent use of the existing facilities and minimize the 'down time' between the loss of the existing facilities and use of the new facilities.

8.0 Conclusion

The Salmon Arm Memorial Arena has been a landmark for city residents for many decades. It has served the City and past owners well since its original construction. The facility has exceeded its anticipated service life, evidenced by the need to replace and repair many crucial structural and building envelope components.

The main deficiencies of the facility pertain to the structure and envelope of the Arena. The roof structure and foundation have inadequate capacity to support the current design loading outlined in the British Columbia Building Code 2024. The lateral stability of the structure is inadequate to resist current wind and seismic loading. The roofing is near failure. And the exterior cedar plank siding is deteriorated and prone to water ingress.

As illustrated in this life cycle analysis, total expenditures nearing $7,000,000 can be expected to extend the service life of the facility by 30 years, with an ultimate retirement date in 2054. $3,000,000 of these expenditures occurring immediately, mainly to address the structural and building envelope deficiencies, and the remaining expenditures needed to maintain the building over the next 30 years and cover demolition after retirement.

The current demolition cost is anticipated to be $935,255, including conversion of the existing site to greenfield conditions.

The estimated cost to replace the existing facility with a new facility of same size and function is estimated to cost $7,580,000. It is likely that a replacement facility would exceed the current facility size and provide supplementary functions and services compared to the existing facility. Therefore, the true replacement cost will exceed this estimated cost.

Based on the detailed structural and building envelope review of the facility and this life cycle analysis it is the opinion of the undersigned that extending the service life should not be considered. As concluded in the Interim Report, if the Arena foundation and building envelope didn't require replacement, rehabilitation may have been considered feasible.

The undersigned recommends that the City of Salmon Arm demolish the existing facility and replace it with a new facility. Project planning, design, and funding is expected to take at least 4-6 years and therefore it is our further recommendation that the City consider the partial occupancy repairs in the interim. These repairs will permit intermittent use of the building by the general public until such time that the City is ready to proceed with construction of a new facility. As outlined in the Interim Report, implementing partial occupancy will include the following list of conditions:

- Occupancy limited to March through November.
- No occupancy permitted during snow accumulation on the roof.
- No occupancy permitted during forecasted and measured wind gust speeds exceeding 40 km/hr.
- Real-time data of roof video feed and wind speed monitoring broadcasted to the facilities operation manager.
- Updating the City of Salmon Arm's Operations Manual of the facility to include the conditions noted above.
- Annual visual assessment of the arena by a structural engineer, prior to occupancy following the winter season, to determine any significant changes in the building condition.

This final report is not intended to provide an opinion regarding responsibility of any party in causing or contributing to the observed condition. Any comments or conclusions within this report represent the opinion of the undersigned, which is based upon the historic documents provided, the site assessment, the structural evaluation, professional engineering judgement, and industry standards.

This report has been prepared for the exclusive use of the City of Salmon Arm and their authorized users for the specific application outlined in this report. Any use which a third party makes of this report, or any portion of this report, is the sole responsibility of such third party or parties. BAR Engineering and the undersigned accept no responsibility for damages suffered by any third party resulting from unauthorized use of this report.

We trust that the reader finds the information provided herein satisfactory. Please contact the undersigned regarding any questions.

Respectfully Submitted,
BAR Engineering Co. Ltd.

Per:

Joey Funk, P. Eng.
Senior Engineer
Oakangan Divison

Reviewed by:

PERMIT TO PRACTICE
BAR ENGINEERING CO. LTD.
PERMIT NUMBER: 1001776
Engineers and Geoscientists BC

Rick Collins, P. Eng.
Manager
Okanagan Division

9.0 References

[1] Buildings Risk Matrix – City of Salmon Arm, received by email on February 22, 2024 (Jennifer Wilson).

[2] Risk Framework – City of Salmon Arm, received by email on February 22, 2024 (Jennifer Wilson).

APPENDIX I

Interim Report

BAR ENGINEERING

March 25, 2024 | Project No.: 24VR-600400

City of Salmon Arm
500 2 Avenue Northeast
Salmon Arm, BC
V1E 1J5

Phone: 250.803.4088
Email: dgerow@salmonarm.ca

Attn: Darin Gerow
Re: Structural Assessment & Life Cycle Assessment of the Memorial Arena – Interim Report
100 - 30th Street SE, Salmon Arm, BC

BAR Engineering Co. Ltd. is pleased to submit this interim report of the detailed structural assessment of the Memorial Arena to the City of Salmon Arm.

This report describes our findings, analysis, and conclusions concerning the state of the building's structural integrity and building envelope, as well as recommendations for repair and associated costs.

Should you have any questions regarding the contents of this report, please do not hesitate to contact the undersigned at (250) 541-9590 or via e-mail at joey.funk@bareng.ca.

Respectfully Submitted,
BAR Engineering Co. Ltd.

Per:

Joey Funk, P. Eng.
Senior Structural Engineer

5237 70 Avenue
Lloydminster, AB, T9V 3N6
780.875.1683

info@bareng.ca
BAReng.ca

201-2540 53 Avenue
Vernon, BC, V1T 9W8
250.541.9590

Structural Assessment and Lifecycle Analysis of the Memorial Arena

Interim Report

Engineering services for

City of Salmon Arm

March 25, 2024

Project No. 24VR-600400

City of Salmon Arm | 500 2 Avenue N.E., Salmon Arm, BC V1E 1J5

Attn: Darin Gerow –Manger of Roads & Parks, City of Slamon Arm

Contact: Joey Funk, P.Eng.
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joey.funk@bareng.ca | 250.541.9590 | BAReng.ca
201-2540 53 Avenue Vernon, BC V1T 9W8

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Appendices

1.0 Introduction

1.1 Purpose

BAR Engineering Co. Ltd. (BAR) has prepared this interim report of the Memorial Arena located at 351 – 3 Street SW in Salmon Arm, BC, for the city of Salmon Arm. The purpose of this Interim Report is to report on the existing conditions of the building envelope and structural systems and provide recommendations for repair and associated costs to maximize facility usage while ensuring user safety. More specifically, the following items will be addressed.

- Provide an opinion on the structural condition of the deteriorated wood column, as identified in the Preliminary Structural Assessment prepared by R&A Engineering, and suitability of the current foundation at the northeast corner of the arena and provide recommendations for repair.
- Provide recommendations for repair of truss web members showing signs of distress and cracking at bolted connections, as identified in the Preliminary Structural Assessment prepared by R&A Engineering.
- Undertake a detailed structural assessment of the existing structure to verify the feasibility of repairing or replacing the existing main roof structure, and a review of the lateral load resisting system.
- Undertake a detailed building envelope and roof drainage assessment and provide recommendations for repair.
- Examine the condition of the arena truss connection bolts, by temporary removal, at random locations and assess the extent of corrosion.

In summary, the recommendations shall include the minimum repairs and associated costs to permit partial occupancy until a full structural rehabilitation can be completed. Furthermore, provide repair recommendations and associated costs to permit full occupancy year-round.

The building envelope and structural assessments were executed by Joey Funk, P. Eng., and Chris Thornton, E.I.T., between February 20, 2024, to March 4, 2024. This interim report has been prepared by Joey Funk, P. Eng., and Chris Thronton, E.I.T. Whit Saretsky, P. Eng. aided with structural analysis and peer reviews.

1.2 Report Organization

This report has been organized as follows:

- Methodology: A summary of the process used by BAR to complete a comprehensive condition assessment of the building envelope and structure.
- Building Information: A summary of the building envelope and structural components.
- Building Envelope Condition Assessment: A summary of the site observations related to the building envelope.
- Structural Condition Assessment: A summary of the site observations related to the building structure.
- Structural Evaluation: Summary of design analysis and results.
- Discussion: Detailed discussion regarding the building envelope and structural condition assessments and the structural evaluation.
- Recommendations: Outline of recommendations to achieve partial and full occupancy.
- Class D Construction Cost Estimate.
- Conclusion.

1.3 Building Background

The Memorial Arena is located at 351-3 Street SW in Salmon Arm, BC. The Arena has a footprint of approximately 26,500 square feet. The arena was built in 1957 with additions added to the East and West of the building in 1961 and 1966 respectively. A mezzanine supporting an additional office and additional storage was construction around 1975. Significant fire and life safety upgrades were conducted in 1991 including the removal of fixed spectator stands, installation of emergency exits, and installation of a fire alarm system.

The original building, the Arena, includes the artificial turf playing field and has an approximate footprint of 18,000 ft^2. The East Addition is approximately 3,350 ft^2 and includes a kitchen, reception area, washrooms, and offices. The West Addition is approximately 3,000 ft^2 and includes storage rooms, a workshop, a truck bay, two washrooms, and offices. The mezzanine, constructed in 1975, is approximately 1,250 ft^2 and includes additional office space and storage.

The ice plant was decommissioned in 1999 and the ice sheet converted to an artificial turf playing field. The ice equipment room was converted into office space and the change rooms were converted into a woodshop and additional storage.

For the purpose of this report, the Arena refers to the original building, and the east and west additions are referred to as the East Addition and West Addition, respectively. A site plan and floor plan illustrating the general layout of the facility has been included in Appendix A.

1.3.1 General Construction

The general construction of the Arena, East Addition, and West Addition is as follows.

1.3.1.1 Arena

Roof Construction: The Arena has an arched roof with two flat roof sections at the east and west ends. The roof construction is 1”x8” diagonal planking over 2”x12” Douglas Fir (D. Fir) rafters spaced at 16” on centre spanning between timber bowstring trusses spaced at 20’ on centre. The east and west ends of the arena have flat roofs. The truss top chords are 5”x14$^{5/8”}$ 9-ply D. Fir glulam members spanning 104’. The angle of curvature at the eaves is approximately 30 degrees. The bottom chords are flat and consist of 5”x9¾” 6-ply D. Fir glulam members. The truss webs consist of vertical and diagonal 2”x6” and 2”x8” D. Fir rough timber. The web to chord connections consist of single 4” diameter split rings. A roof framing plan of the arena has been included in Appendix A.

The gable trusses 20’ from each end of the arena are of similar construction with the exception of vertical steel tension rods spaced evenly along the length of the truss providing additional support of the bottom truss chord. The rafters of the east and west flat roofs are supported by the bottom chord of the gable trusses and timber beam and columns along each end wall.

The roofing includes torch-on SBS roll roofing.

The gable truss ends are clad with horizontal cedar lap siding over 2x4 studwall framing.

Wall Construction: The trusses are supported on 21’ tall 10”x16” D. Fir rough timber posts. The infill framing between the columns consists of 6”x10” D. Fir horizontal struts at the top and bottom of the walls, and two additional struts at approximately 8’ and 16’ above floor level; 4”x6” D. Fir rough timber diagonal bracing between the struts; and 2”x4” D. Fir studs spaced at 16” on centre.

The walls are clad with horizontal cedar lap siding.

Foundation: The side wall columns are supported on 18”x20”x36” concrete pilasters supported on 56”x56”x15” concrete spread pad footings. The end wall columns are supported on 18”x18”x36” pilasters over 24”x24”x12” concrete spread pad footings. The original arena foundation plan is included in Appendix B.

1.3.1.2 East Addition

Roof Construction: The east addition consists of an extension of the original flat roof of the arena and a lower flat roof. The lower roof is constructed of tongue and groove 3”x4” rough timber D. Fir planking. The planking is supported on 10”x14” D. Fir rough timber beams and 10”x10” D. Fir rough timber posts.

The roofing includes torch-on SBS roll roofing.

Walls Construction: The exterior walls of the East Addition are combination of the arena wall construction type and conventional studwall framing with exterior sheathing.

The exterior wall finish is painted stucco.

Foundation: The foundation consists of concrete frost walls over strip footings and spread pad footings below interior columns. The floor is a concrete slab on grade, contrary to the East Addition record drawings included in Appendix B.

1.3.1.3 West Addition

Roof Construction: The roof construction consists of wood rafters and TJI Joists. The size and spacing cold not be confirmed due to ceiling finishes. The roof framing is supported on concrete masonry unit (CMU) walls.

The roofing includes torch-on SBS roll roofing.

Wall Construction: The interior walls consist of CMU walls and conventional wood studwalls. The exterior walls are CMU.

The exterior wall finish is painted CMU.

Foundation: The foundation consists of frost walls on strip footings. The floor is a slab on grade.

2.0 Methodology

The building envelope and structural condition assessments were completed using the following processes:

- Background Investigation
- Site Assessment
- Structural Evaluation

2.1 Background Investigation

The following documents were provided by the City of Salmon Arm and were reviewed in detail. The documents are listed in approximate chronological order. Copies of the following documents are provided in Appendix B and C.

- Original Construction Drawings, Partial
- 1984 Arena Inspection Report
- 1994 Arena Review Report

- 1996 Proposed Building Geotechnical Review Report
- 1999 Arena Architectural Assessment
- 2000 Arena Fire Safety Recommendations Report
- 2016 Asbestos Materials Management Survey Report
- 2016 Lead Paint Bulk Sampling Results Letter
- 2016 Asbestos Clearence letter
- 2023 Preliminary Structural Condition Assessment

No construction records of the west addition were available.

The following is a brief summary of the documents reviewed.

2.1.1 Original Construction Drawings, Partial

Drawings include the following:

- Arena Foundation Plan and Details
- Arena Bracing Details
- Arena Wall Framing and Details
- Arena Truss section, specifications, and connection detailing.
- East Addition Floor Framing Plan, Foundation Plan, Exterior Elevations, and Sections
- East Addition Framing and Foundation Details

2.1.2 1984 Arena Inspection Report

The 1984 arena inspection report by Lowell A. Paul P.Eng. noted the trusses were in good condition however were under designed for the snow load of the time (49psf). The assessment recommended contacting the original truss designers for guidance on potential upgrades to the trusses. In the meantime, the report recommended the clearing of snow on both the arched and flat roofs to prevent exceeding the design snow load. The report noted decay at the base of the arena columns and along the exterior wall siding. The report recommended further investigation into the severity of the decay observed in the wood columns.

2.1.3 1994 Arena Review Report

The 1994 arena review report by Gordon Isaac noted the completion of major repairs to the roof columns and wood framing. Additionally, the report noted frost heave damage and decay on interior columns. The report recommends a structural engineer examine all exterior columns for decay and provide an estimated lifespan for the columns. Isaac noted decay particularly in areas in contact with or near to the ice surface. The report notes the exterior walls were installed without building paper which allowed extensive wetting of structural framing members. Discussion regarding life safety considerations, energy efficiency, accessibility requirements, and parking were also included.

2.1.4 1996 Proposed Building Geotechnical Review Report

A geotechnical report was prepared for the property south of the Memorial Arena and north of 5th Ave SW. The report notes the soil is composed of loose silt, soft to very soft clayey silt, and very loose silt with occasional deposits of sand. The report recommended replacing soil to a depth of 1m underneath footings, and slabs on grade with structural fill. The report noted a shallow ground water table with anticipated ability to rise above ground level with periods of high precipitation. Preloading of fill was recommended for the site for a duration of 6 months prior to construction.

2.1.5 1999 Arena Architectural Assessment

A summary of the property including building square footage, room uses, conditions, and life safety concerns for the building at the time. The ground floor level of the structure was observed to be level with exterior grade and the grade did not slope away from the building. Roof drainage was noted as adequate. The assessment listed ongoing problems with the prior repairs of some wood column bases, the strengthening of flat roof beams, and general water ingress. Some fire safety concerns were noted related to the uses of the rooms at the time.

2.1.6 2000 Arena Fire Safety Recommendations Report

Gage Babcock and Associates reviewed the structure to Part 3 of the 1998 British Columbia Building Code and provided recommendations for improving the level of fire and life safety. The report recommended the removal of the mezzanine installed in 1975, the removal of all ‘not in use’ mechanical and electrical equipment, and the removal of the enclosures around unused rooms. The report detailed the addition of fire exits to the main arena and the creation of a fire exit plan. The buildings spatial separation on the north face of the structure was found to be inadequate however upgrading the structure was deemed prohibitively expensive. Recommendations to reduce storage areas and remove sources of ignition were made.

2.1.7 2016 Asbestos Materials Management Survey Report

An asbestos materials management survey was conducted by APEX EHS Services. The survey found vermiculite in the west addition CMU walls. Additionally, asbestos was found in the vinyl kitchen flooring and kitchen sink mastic. The immediate abatement of the asbestos containing materials (ACMs) in the west addition was recommended.

2.1.8 2016 Lead Paint Bulk Sampling Results Letter

Per the request of Okanagan Restoration on behalf of The Salmon Arm & Shuswap Agricultural Association, sampling for suspect lead paint was conducted. Lead paint was detected in limited quantities on the exterior and interior faces of the CMU walls in the west addition. Recommendations included the development of safe work practices, exposure plan, and risk assessments if the lead paints were to be handled.

2.1.9 2016 Asbestos Clearence letter

A letter by APEX EHS Services indicating that air samples from the interior of the building was clear of asbestos contamination while abatement was ongoing. APEX EHS Services did not design or supervise the asbestos abatement.

2.1.10 2023 Preliminary Structural Condition Assessment

In January of 2023 a preliminary structural condition assessment of the arena was executed by R&A Engineering. The assessment included a visual examination of existing wood structural members. No destructive testing was conducted, and the structure foundations were not reviewed. The report noted the trusses were showing signs of distress with splits observed in the truss webs. Additionally, due to apparent foundation settlement, the bases of the wood columns had been exposed to moisture allowing decay. R & A Engineering recommended that a detailed structural assessment be conducted to determine required repairs to the structural elements and building envelope. A feasibility study of repairing or replacing the roof structure was also recommended. Additionally, R & A recommended a life cycle analysis be conducted and a geotechnical engineer be consulted to address the building settlement.

2.2 Site Assessment

The on-site assessments were executed by Joey Funk, P. Eng. and Chris Thornton, E.I.T. from February 2024, to March 2024.

2.2.1 Visual Assessment

The visual assessment included a review of the building envelope and structure. The roof structure was assessed using a 45-foot articulating boom lift which allowed close observation of the roofing, roof deck, and trusses.

Minor intrusive investigation methods were used to determine the condition of various building components. Random bolted truss connections were disassembled to visually assess the condition of the truss hardware and wood material at the connection. Foundations were excavated in two random locations to visually assess their condition and examine the soil bearing conditions. Lastly, cores were cut into the east addition roof assembly to verify the roof construction.

Each building component was recorded, and their condition assessed and documented.

The condition of each building envelope and structural component was rated using the following criteria:

- Good Condition – No visual defects, component performing as intended.
- Fair Condition – Minor defects, component performing as intended.
- Poor Condition – Moderate defects, component not performing as intended, repair or replacement recommended.
- Failed Condition – Major defects or complete failure. Repair or replacement required.

2.2.2 Interior Alignment Survey

The interior alignment of the structural components of the arena was completed using a digital level, laser level, and visual sighting methods.

The general alignment of the trusses, associated bracing, and columns was completed by visually looking along the length of each member to determine how straight and plumb they were. Gross discrepancies were recorded.

A digital level and a laser level were then used to measure the alignment and plumbness of the trusses, associated bracing, and columns.

Lastly, the camber and sag of the trusses were surveyed.

The straightness and plumbness of structural members, and the deflection of the trusses, were evaluated against standard codes and engineering judgment based on the type and size of loading supported by the structural member. The Canadian wood design standard, CSA-086, does not include erection tolerances for timber columns. Therefore, the steel column erection tolerances outlined in the steel design standard, CSA S16-2014, were used as guidance in evaluating the plumbness of the timber columns.

2.3 Structural Evaluation

The structural evaluation consisted of field verification, computed-aided modelling, and design analysis of the existing structural members.

The structural members were measured on site to determine size, orientation, material, and end support conditions and compared against the original structural drawing specifications.

After verifying the truss layout and member sizes, a computer-aided model of the truss was created using Staad-Pro and design loads applied to determine member and connection forces.

The calculated member and connection forces were then compared to the member and connection capacities and their utilization reported.

3.0 Building Information

The following tables summarize the components of the building envelope and structure. The general condition of each component including photographic representation of the conditions observed are provided in the subsequent building condition assessment and structural assessment.

Table 1: Building Envelope Components	
Roofing	Modified bitumen SBS torch on roll roofing
Exterior Siding	Horizonal cedar lap siding Horizontal Hardi-plank lap siding Vertical metal cladding Plywood sheathing Stucco Painted CMU Soffit & Facia
Windows	Wood frames PVC frames Glazing
Doors	Solid wood Insulated steel Hollow steel Steel overhead door Glazing
Flashings and Sealants	Painted aluminium flashing Aluminum gutters Silicone, bitumen, and polyurethane sealers
Insulation	Vermiculite Fibreglass batt Polyurethane spray foam
Vapour Barrier	Unknown.

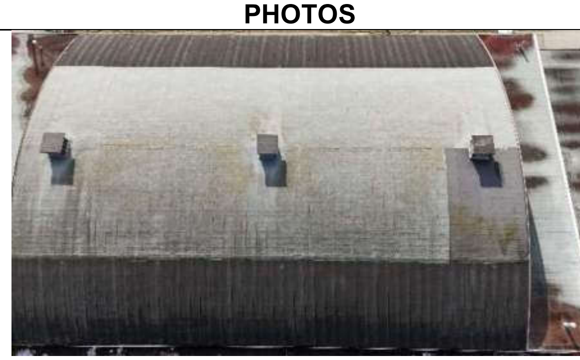
Table 3: Structural Components	
Roof Deck	Tongue and groove planking Plywood sheathing
Rafters	Douglas Fir 2x12 TJI engineered joist, unknown depth
Roof Trusses	Timber bowstring trusses with glue-laminated top and bottom chords and rough sawn timber webs and bracing.
Struts	Rough sawn D. Fir timber
Columns	Rough sawn D. Fir timber
Exterior Wall Framing	Concrete Masonry Unit (CMU) walls Wood framing

Interior Wall Framing	Concrete Masonry Unit (CMU) walls Wood framing
Mezzanine Framing	Tongue and groove planking on wood joist
Slab on Grade	Cast-in-place concrete.
Foundation	Concrete foundation wall on continuous strip footings Concrete pilaster on spread pad footings

4.0 Building Envelope Condition Assessment

4.1 Roofing

OBSERVATIONS	PHOTOS

SBS Torch-on Roll Roofing – Arena
• Center third of arched roof area
  ○ Roofing appears to be in fair condition. Photo 1.
  ○ Sanded surface appears weathered but still intact.
• Outer thirds of arched roof area (along eaves)
  ○ Roofing appears to be in poor condition.
  ○ Sanded surface has deteriorated and only the SBS membrane remains. Photo 2.
  ○ Large portions of the aggregate from the sanded surface have accumulated in the eavestroughs along the eaves of the arena.
  ○ Edge and end laps appear adequate, and the thermally welded edges are mainly intact.
  ○ Roofing terminates along eaves with a welded connection to the aluminum eave flashing. The eave flashing has a ¾” vertical upturn such that debris and water remain trapped at the eave, restricting natural drainage into the eavestroughs.
  ○ The roofing at the southeast corner is folded and is susceptible to water ingress. Photo 2.
	
	Photo 1: Overview of arched roof area.
	
	Photo 2: Folded SBS at southwest corner. Note lack of sanded surface.

- East and west flat roof areas
    - Roofing appears to be in fair condition. Photo 3 and 4.
    - Sanded surface is intact.
    - End and side laps of roll roofing are intact and watertight.
    - Membrane extends vertically up rake walls, behind gable wall lap siding.
    - Standing water observed in numerous locations as a result of inadequate roof slopes. Photo 3 and 4.
    - 2-3 inches of standing water at the northwest corner of the roof
    - The scuppers are clear of debris.
    - Roof penetrations appear to be adequately sealed.

SBS Torch-on Roll Roofing – East Addition

- Roofing is in fair condition.
- Entire lower roof area was flooded at time of review. No leaks apparent within building interior suggesting roofing is watertight. Photo 5.
- Significant staining observed throughout.
- Poor drainage.

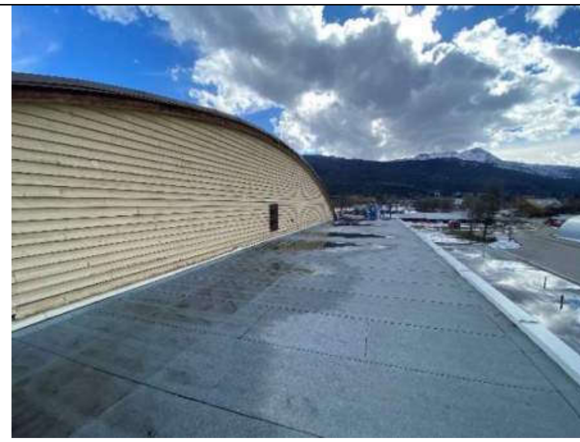


Photo 3: Flat roof at west end of arena. Ponding water observed.

Photo 4: East arena flat roof.

Photo 5: East addition lower roof flooded.

SBS Torch-on Roll Roofing – West Addition

- Roofing appears to be in poor condition. Photo 6.
- Sanded surface is deteriorated and has micro cracking throughout. Photo 7.
- End and side laps of roll roofing are intact and watertight.
- Membrane extends vertically up rake wall and is thermally welded to the painted plywood gable end. Bond failure between the roofing and painted plywood was observed. Photo 7.
- Standing water observed in numerous locations as a result of inadequate roof slopes. Photo 6.
- The scuppers are clear of debris.
- Roof penetrations appear to be adequately sealed.
- Large membrane blisters observed along the rake wall. Photo 7.

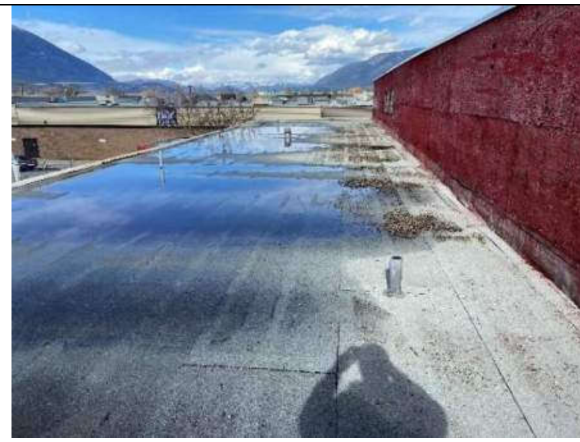


Photo 6: West addition upper roof typical condition

Photo 7: Microcracking, bond failure, and blistering observed.

4.2 Exterior Siding

OBSERVATIONS

Horizontal Cedar Lap Siding

- The cedar siding covers the east, west and north sides of the arena, including the gable truss ends.
- The lap siding is in failed condition throughout.
- The painted finish is in failed condition and is peeling throughout. Photo 8.
- Deterioration and decay observed throughout.
- Severe deterioration and decay within 48 inches of grade. Photo 9.
- Visible light shines through deteriorated lap siding throughout the arena. Photo 10.
- Lap siding is fastened directly to studs with no sheathing or air barrier installed.
- Lap siding extends below grade along east half of north wall. Photo 9.

PHOTOS

Photo 8: Typical condition of ceder lap siding.

Horizontal Hardi-plank lap siding

- The Hardi-plank lap siding is limited to the south wall of the arena and extends from grade to approximately within 5 feet of the eave. Photo 11.
- The siding is in fair condition throughout.
- The siding extends to grade along the east end of the arena. The manufacturer requires minimum 6-inch clearance from grade.
- The siding was installed over WRB and OSB sheathing.
- The bottom 6 inches of the OSB sheathing was saturated and has severe deterioration along the east half of the south wall.

Photo 9: Ceder lap siding severely deteriorated within 48” of grade.

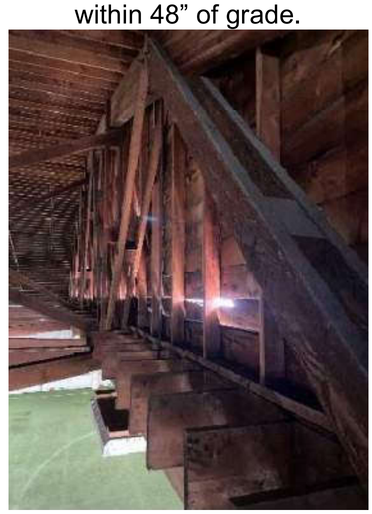


Photo 10: Light entering through ceder lap siding.

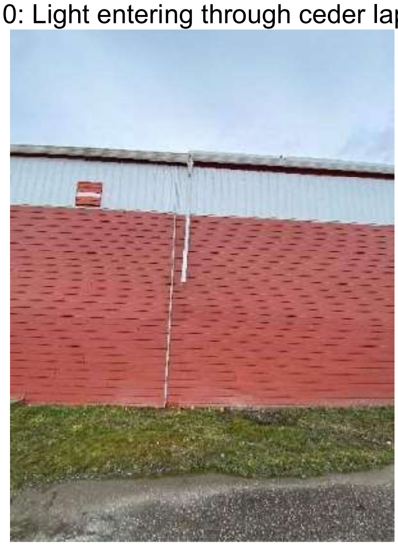


Photo 11: Typical condition of Hardi-plank lap siding.

Vertical metal cladding

- The vertical metal cladding is limited to the upper 5 feet of the south wall of the arena.
- The metal cladding and associated trims and flashings are in good condition. Photo 12.
- The siding was installed over air barrier and OSB sheathing.

Plywood sheathing

- The west gable of the west addition is finished with painted plywood.
- The plywood is deteriorated and in poor condition. Photo 13.
- The paint is peeling throughout.
- Cut outs in the plywood exposing structural timber and bolted connections were observed.

Stucco

- The stucco siding is limited to the east addition of the building only.
- The stucco appears to be in fair condition with no significant cracking, delamination, or spalling. Photo 14.
- The stucco has a painted finish.

Photo 12: Vertical Metal Siding.

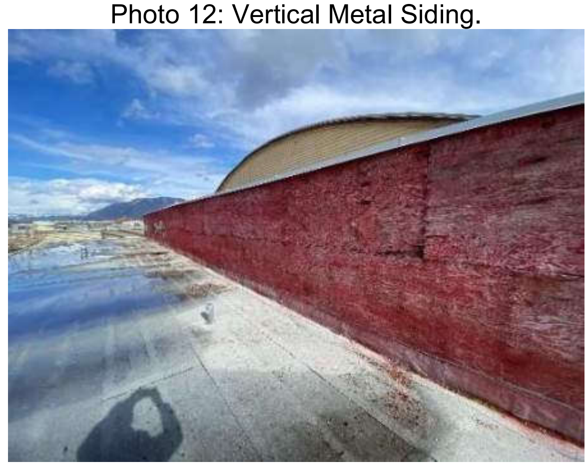
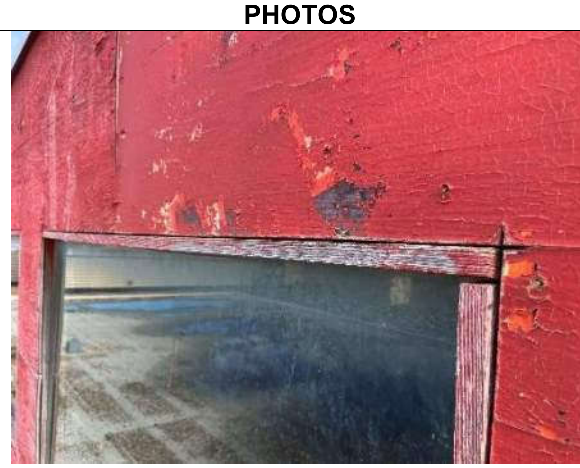


Photo 13: Typical condition of plywood sheathing.

Photo 14: Typical condition of stucco siding.

Painted CMU • The west addition is finished with painted CMU. • The paint is in fair condition throughout. Photo 15. • Minor impact defects noted in numerous locations. • Peeling observed near grade. • The condition of the CMU block is further discussed in the structural condition assessment later in this report. **Soffit & Facia** • The soffit and Facia along the gable end of the arena is painted wood. • The paint is in failed condition and the wood is deteriorated. Photo 16.	 Photo 15: Typical condition of painted CMU.  Photo 16: Deterioration present on painted facia board.

4.3 Windows

OBSERVATIONS	PHOTOS
Wood Frames • The exterior windows throughout the arena and the east side of the east addition are wood framed. • The wood frames are deteriorated and in poor condition. Photo 17. • Water staining on the interior side of the window frames suggests water ingress.	 Photo 17: Typical wood window frame condition

PVC Frames • The east and west addition windows are mainly PVC framed. Photo 18. • The frames are in fair condition with the exception of the south window in the east addition which is in poor condition. • The interior windows throughout the building are mainly PVC and are generally in fair condition. **Glazing** • The window glazing throughout the building are single pane and double pane sealed units. • The glazed units in PVC frames are generally in fair condition and well sealed. • The glazed units in wood frames are in poor condition and poorly sealed.	 Photo 18: Typical PVC window frame condition

4.4 Doors

OBSERVATIONS	PHOTOS
Solid Wood • The solid wood doors are located sporadically throughout the building interior. The doors are generally in fair condition. Photo 19. • The southwest corner of the wood shop has a sliding wood door. The door is in poor condition and is difficult to operate. • The south wall of the arena has a large exterior barn style sliding door. The door is wood framed with a painted plywood exterior. The paint is peeling, and the wood is deteriorated. The door hardware appears to be in fair condition.	 Photo 19: Solid wood door

Insulated Steel

- The exterior doors in the east and west addition are insulated steel doors.
- The doors and frames are generally in fair condition. Photo 20.
- The door seals, weather stripping, and door hardware are in fair condition.

Photo 20: Insulated steel door

Overhead Door

- The overhead door serving the vehicle bay in the west addition is in poor condition. Photo 21.
- The overhead door opener is not operational.
- The latch hardware for locking the overhead door is not operational.
- The door does not seal to the shop floor or door jamb.
- The door has mechanical damage from impact.

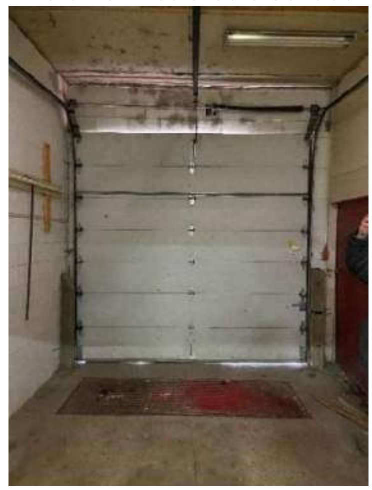


Photo 21: Overhead door

Hollow steel

- The emergency exit doors serving the arena and the west addition are hollow steel doors. Photo 22.
- The doors are in failed condition.
- The doors seals and weather stripping are in failed condition and/or missing.
- Door hardware failure was observed throughout.

Photo 22: Hollow steel door

Glazing

- The main entrance doors on the east wall of the east addition and the entrance door at the southeast corner of the east addition have glazing.
- The glazing is in fair condition. Photo 20.

4.5 Flashing and Sealants

<table>
<tr><th>OBSERVATIONS</th><th>PHOTOS</th></tr>
<tr><td>

Flashing

- Aluminum flashing exists along the eaves and gable ends of the roofs.
- The flashings are generally in poor condition. Photo 23.
- Flashing fasteners missing in some locations.
- Gable flashing at southwest corner of arena roof does not fully cover wood. Wood has deteriorated and flashing fasteners no longer engaged into substrate materials. Photo 23.
- No head and sill flashing observed at windows and doors with the exception of the south wall of the arena.
- Flashing at material transitions don’t adequately protect against water ingress.
- The metal flashings along the south wall of the arena, where the exterior siding has been replaced with metal cladding, are in fair condition. Photo 24.

Aluminum Gutters & Downspouts

- The arena is fitted with aluminum gutters along the eaves with 4 downspouts evenly spaced along each eave. Photo 24.
- The gutters are generally in poor condition.
- The gutters have 2-3 inches of debris accumulation. Much of the debris includes the aggregate surface of the SBS roofing system. Drain holes at downspouts were mostly clogged.
- The downspouts are in failed condition. Downspouts missing or damaged in numerous locations. (Photo 25).
- Downspouts no longer drain into designated stormwater piping.

Silicone, Bitumen, and Polyurethane sealers

- Majority of the exterior wall penetrations have not been sealed or flashed.
- The caulking of the Hardi-plank siding on the south side of the Arena is in fair condition.

</td><td>

Photo 23: Gable flashing with deteriorated substrate materials

Photo 24: Aluminium gutters and flashing along south arena wall.

Photo 25: Typical condition of gutter downspouts

</td></tr>
</table>

4.6 Insulation

OBSERVATIONS	PHOTOS
Vermiculite • Based on the background data reviewed the west addition CMU walls are filled with vermiculite insulation. • The insulation could not be visually assessed. **Fibreglass Batt** • Fibreglass insulation in the rafter bays of the west addition was confirmed, but its condition could not be assessed due to ceiling finishes. • The east addition is assumed to have insulation between the roofing membrane and wood decking. Its condition could not be confirmed. • The east addition exterior walls are assumed to be insulated with fibreglass batt insulation. Its condition could not be confirmed. **Polyurethane Spray Foam** • The rafters above the offices at the northwest corner of the west addition are insulated with polyurethane spray foam insulation. • The insulation is in good condition. Photo 26.	 Photo 26: Polyurethane spray foam and TJI joists

4.7 Vapour Barrier

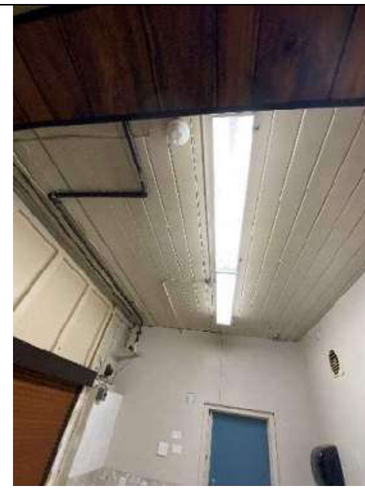
OBSERVATIONS	PHOTOS
• No vapour barrier was observed in the building wall and roof assemblies.	

5.0 Structural Condition Assessment

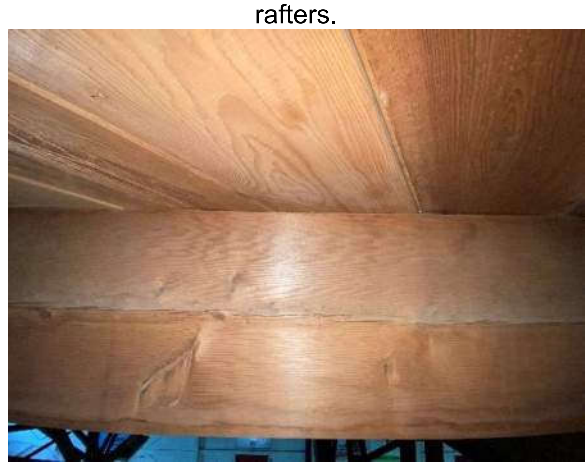
The following is a summary of the general condition of the structural components. The detailed arena truss assessment data collected in the field has been included in Appendix D for reference.

5.1 Roof Deck

OBSERVATIONS	PHOTOS
Tongue and Groove Planking • Arena Roof: o 1”x8” D. Fir diagonal planking throughout. o Planks are in fair condition. o Water/moisture staining was observed sporadically throughout. o No decay or rot was observed. o Shrinkage was observed throughout such that the tongue and grooves are not tight fitting. o Gaps up to ½” in width were observed in some locations in which the asphalt roofing could be seen. o Larger gaps and holes due to past deterioration of the planking have been covered with metal sheathing during past roofing repairs. Photo 28. o Moisture content of the planking throughout the arena ranges from 7-10%.	 Photo 27: Typical condition of arena roof planking  Photo 28: Metal sheahting over gaps in planking.

• East Addition
  o Planking is in fair condition. Photo 29.
• West Addition
  o The roof decking was not assessed due to roof and ceiling coverings.
 	
Plywood Sheathing
• Plywood sheathing was observed through numerous gaps in the arena roof planking.
• The extent and condition of the sheathing could not be observed.
	
	Photo 29: T&G planking, east addition.

5.2 Rafters

OBSERVATIONS	PHOTOS
2x12 Douglas Fir Rafters • The rafters are generally in good condition. • Water/moisture staining throughout. Photo 30. • The rafters are straight. • No excessive deflections were observed at the time of the assessment. Note that there was no snow accumulation on the roof at the time of the assessment. • Splitting of several rafters was observed. Refer to Appendix D which illustrates the approximate locations of rafters with observed splitting. Photo 31. • The bearing locations of the rafters on the top chord of the trusses are in good condition. No signs of bearing failure or lateral movement of the rafters were observed. • The rafters in the west addition were not visually assessed due to coverings. The rafters are assumed to be 2"x12" D. Fir members. **TJI Joists** • The rafters above the offices at the northwest corner of the west addition are TJI joists spaced at 16" on center. • The condition of the TJI joist could not be assessed as they are fully encapsulated by spray foam insulation with the exception of the bottom chords of the joists.	 Photo 30: Water staining observed on arena rafters.  Photo 31:Observed splitting in rafter

5.3 Roof Trusses

OBSERVATIONS	PHOTOS

OBSERVATIONS

Top Chord

- The top chord of the bowstring trusses are 9-ply 5”x$14^{5/8}$” glulam arched members with an approximate curvature radius of 54’-$7^{3/16}$”. Photo 32.
- The top chords are generally in good condition.
- Water/moisture staining was observed throughout, but no signs of decay or rot were observed.
- Moisture content ranged from 8-10% throughout the arena.
- No crushing failures at rafter bearing locations were observed.
- Minor checking was observed on most of the truss top chords.

Bottom Chord

- The bottom chord of the bowstring trusses are 6 ply 5”x$9^{3/4}$” glulam members. Photo 33.
- The bottom chords are generally in good condition.
- Water/moisture staining was observed throughout, but no signs of decay or rot were observed.
- Moisture content ranged from 8-10% throughout the arena.
- No crushing failures at rafter bearing locations were observed.
- Minor checking was observed on most of the truss bottom chords.

Truss Webs

- The truss webs are rough sawn D. Fir members. Web sizes include 2”x6” and 2”x8” members.
- Approximately 50% of the webs are in fair condition while the remaining 50% considered failed due to full depth splits at web to chord connections. Photo 34.
- The truss webs are nominally straight with limited lateral bowing.
- Approximately 25% of the web members have checking. Photo 35.
- Moisture content ranged from 8-10% throughout the arena.

PHOTOS

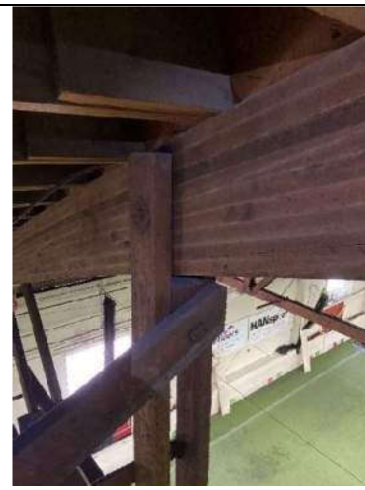


Photo 32: Typical condition of top chord.

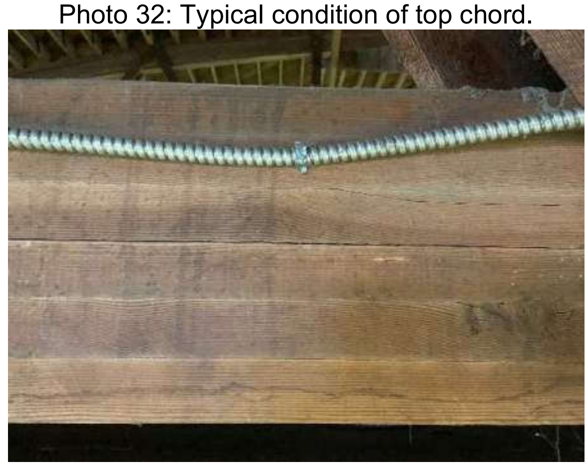


Photo 33: Typical condition of bottom chord. Note minor checking in 3rd lamnination.

Photo 34: Typical splitting at web to chord connection.

Top Chord Splice

- The top chord is spliced at the peak of the arched roof.
- The splice connections are in good condition with no signs of stress related failures or deterioration. Photo 36.

Bottom Chord Splice

- The bottom chords are spliced at midspan with glulam splice blocks.
- The splice blocks are in good condition. Photo 37.
- Minor checking was observed throughout.

Photo 35: Web member with checking along entire length.

Photo 36: Typical top chord splice.

Photo 37: Typical bottom chord splice.

Truss Bearing

- The truss bearing locations appear to be in good condition with no signs of deterioration. Due to the nature of the steel saddle connection at the truss bearing locations the actual condition of the timber within the saddle could not be reviewed in detail. Photo 38.1.

Bottom Chord to Top Chord Splice

- The steel splice plates are in good condition. Photo 38.1.
- The bottom chords within the splice plate connection could not be observed and end splitting could not be confirmed.
- Gaps between the top chord and bottom chord at the splice location ranged from ¼" to ½". This may have caused some end splits in the bottom chord which could not be confirmed.

Knee Braces

- The knee braces at the ends of the trusses are generally in fair condition with the exception of a few knee braces at the east end of the building which are in a failed condition due to end splitting. Photo 38.2.

Truss Bolts

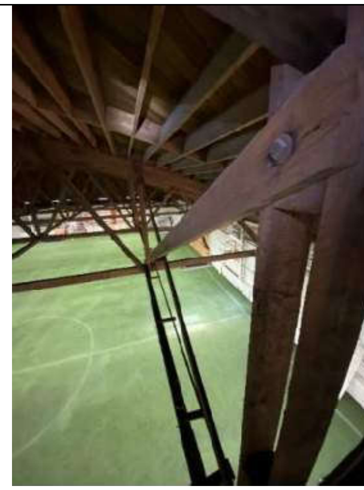
- Surface corrosion on the connection hardware was observed throughout. Photo 39.
- Random connection bolts were removed and assessed. The bolts reviewed are all in good condition. Photo 40.

Photo 38.1: Truss bearing plate.

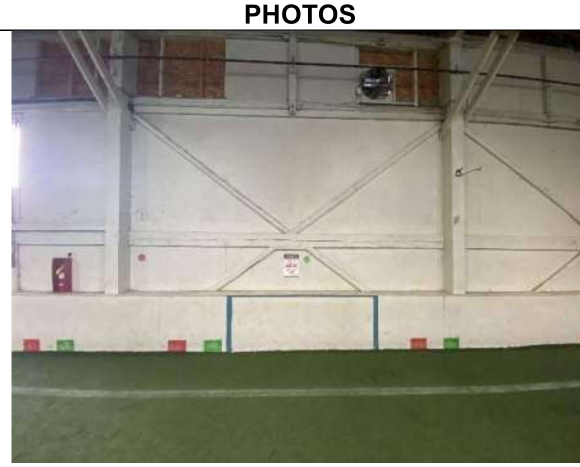
Photo 38.2 Typical knee brace.

Photo 39: Typical surface corrosion on connection hardware.

Photo 40: Typical bolt surface corrosion.

Truss Sway Bracing • The truss bracing is generally in fair condition with the exception of the east braced bay (between trusses G and H). • The sway braces in the east bay are laterally displaced 2 inches, at midspan, toward the south due to buckling. • Brace to truss connections are generally in fair condition. **Truss Bottom Chord Bracing** • The bottom chord bracing consists of cross-bracing in the east and west bays and strut braces in the remaining bays. • The strut braces are in fair condition throughout. • The cross-bracing in the end bays are in poor condition. Due to their long span, they have sagged 2-3 inches. Photo 42. • Brace to truss connections are in fair condition with limited splitting.	 Photo 41: Typical sway bracing.  Photo 42: Typical bottom chord cross bracing.

5.4 Wall Struts and Bracing

OBSERVATIONS	PHOTOS
Struts • The struts span between the arena columns and are located at the bottom of the walls, approximately 6 feet above the arena floor, at the knee brace to column connection, and at the top of the walls. Photo 43. • The struts are generally in poor condition. • The struts located at the column bases on the east half of the arena are in failed condition due to severe decay. • Severe checking was observed throughout with checks measuring greater than ½” in width and 3” in depth. • Significant warping and twisting was observed throughout. • Strut to column connections have failed in many locations. The wood has split at	 Photo 43: Typical wall struts and bracing.

toenail locations and the members are disjointed from one another. Photo 44. **Bracing** • The walls of the arena have 4”x6” cross bracing. Photo 43. • The braces are generally in fair condition. • Checking was observed throughout. • Several brace to column and brace to strut connections have failed due to building movement and member twisting and warping.	 Photo 44: Strut to column connection.

5.5 Columns

OBSERVATIONS	PHOTOS
Arena Sidewall Columns • The columns are 10” x 16” D. Fir and extend from the foundation to the underside of the roof trusses. • The columns are generally in very poor condition. • Severe checking was observed throughout. Checks measured as large as ¾” in width and 3” in depth. Photo 45. • Some columns have end splitting at the base connection. • Moisture staining was observed throughout. Photo 45.2/ • The base of the columns on the east half of the arena are at/below grade and exposed to moisture. These columns have varying degrees of decay. The base steel brackets at these columns are corroded. Photo 46.	 Photo 45.1: Typical arena columns. Checking observed.  Photo 45.2 Typical end split at base of column.

	 Photo 46: Severe column base deterioration on east side of arena.

5.6 Exterior Wall Framing

<table>
<tr><th>OBSERVATIONS</th><th>PHOTOS</th></tr>
<tr><td>
Concrete Masonry Unit (CMU) Walls

The west addition CMU walls are generally in fair condition.
Minor step cracking was observed above the exit door at the southwest corner of the west addition.
Minor step cracking was observed on the west wall of the wood shop.
The CMU walls are straight and plumb.

Wood Framing

East Addition

The exterior walls are generally in fair condition.
The exterior wall framing could not be visually assessed due to wall finishes.
Based on the background data reviewed and the site conditions, it is expected that the exterior walls are framed using conventional wood framing.
The walls are straight and plumb.
No concerning defects were observed.

Arena

The infill framing between the columns, struts, and bracing is 2”x4” D. Fir studs spaced at 16” on center with plywood sheathing on the interior face.
The infill framing is generally in poor condition. Photo 48.

</td><td>

Photo 47: Typical condition of exterior CMU walls.

Photo 48: Typical infill studwall framing seen on left, bearing on bottom strut, partially below grade. Studs clad with horizontal lap siding. No sill plate observed. Moisture staining throughout.
</td></tr>
</table>

○ The south wall was sheathed with OSB on the exterior face during a previous siding replacement. ○ The bottom 6 inches of the OSB on the east half of the south wall is decayed due to moisture exposure.	

5.7 Interior Wall Framing

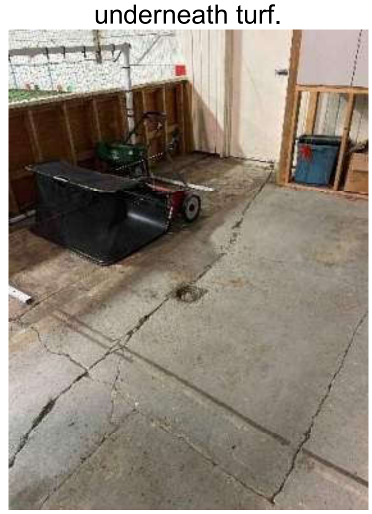
OBSERVATIONS	PHOTOS
Concrete Masonry Unit (CMU) Walls • The interior partitions in the west addition are CMU framing. • The walls are generally in good condition. • The walls are plumb and straight. **Wood Framing** • The interior partition framing could not be visually assessed due to wall finishes. • The walls are plumb and straight. • No significant cracking in wall finishes was observed. • The beam and column roof supports in the east addition are in fair condition with the exception of the south column which has mechanical damage at the base. Photo 50. • Checking in both the beams and columns was observed.	 Photo 49: East addition beam and column roof supports.  Photo 50: East addition south column with mechanical damage at base.

5.8 Mezzanine Framing

OBSERVATIONS	PHOTOS
• The mezzanine floor framing at the northeast and southeast corners of the arena could not be visually assessed due to floor and ceiling finishes.	

• The floors are generally level and stiff. • No significant defects in floor and ceiling finishes were observed.	

5.9 Slab on Grade

OBSERVATIONS	PHOTOS
• The floor slabs are generally in fair condition throughout the building. Photo 51. • Typical shrinkage and temperature cracking was observed throughout. • The concrete slab at the east end of the arena is in poor condition with cracking throughout and differential settlement. Photo 52. • The slabs throughout the east and west additions are nominally level. • The arena slab was surveyed. The slab elevation varies 1.5”.	 Photo 51: Observed sample section of slab underneath turf.  Photo 52: East end of the arena typcial slab cracking

5.10 Foundation

OBSERVATIONS	PHOTOS
East and West Additions • The additions are assumed to be supported on concrete frost walls and continuous strip footings. • Only the portions of the foundation walls above grade could be observed. • The foundation walls appear to be in fair condition. Photo 53. • No significant signs of differential settlement or cracking were observed. **Arena** • The arena foundations consist of concrete pilasters on spread pad footings. • Two pilasters and pads were partially excavated and confirmed to match the original design drawings. The pilaster and pads appeared to be in fair condition. No cracking or signs of instability were observed. Photo 54. • The top surface of the east half of the building pilasters are at or below grade. Photo 55.	 Photo 53: Foundation wall observed at grade.  Photo 54: Arena pilaster and pad footing.  Photo 55: Top of pilaster roughly 4” below grade.

5.11 Interior Alignment Survey

The interior alignment survey included checking the arena trusses, associated bracing, and support columns for plumbness and straightness. Furthermore, the trusses were surveyed to determine truss deflections.

The measurements were taken while the arena roof was free of any snow accumulation. Recorded data such as truss deflections and bowing of compression members are likely to increase under heavier loading conditions. Refer to Appendix D for collected survey data.

The trusses are generally plumb and straight throughout the arena. The truss camber ranged from 2.125" to 3". The original truss design drawings specified a 3" truss camber.

The bottom chord cross bracing located in truss bays A-B and G-H are sagged approximately 3 inches.

The vertical sway bracing in the truss bays G-H are bowed southwards approximately 2 inches.

The elevation of the truss bearing locations along the north side of the arena are within ¼" with the exception of truss H which is approximately 4" lower. The elevation of truss bearing locations along the south side of the arena are within ½" with the exception of truss E, G, and H which are 1.5", 2", and 2.25" lower, respectively.

The columns throughout the arena are nominally plumb in the east-west direction. Majority of the columns are slightly out of plumb in the north-south direction. Most notably, the north and south columns at truss G and H are 1.5"-2" out of plumb.

6.0 Structural Evaluation

The following discussion summaries the original loads used in the design of the structure and the design loading currently required by the British Columbia Building Code 2024 (BCBC 2024) and National Building Code of Canada 2015 (NBCC 2015).

6.1.1 Design Loads

The arena roof was originally designed for a uniform snow load of 40psf. Trusses A and H were designed for a dead load of 15psf whereas the remaining trusses were designed for 12psf.

The as-built weight of the arena roof has been estimated based on the observed roof assembly. The roof assembly observed is as follows with respective weights noted:

• 2-ply SBS roll roofing	2 psf
• 3/8" plywood	1.1 psf
• Asphalt Shingles	2.5 psf
• ¾" tongue and groove planking	2.2 psf
• 2x12 rafters @ 16" o/c	3.3 psf
• Trusses @ 20ft o/c	2 psf
Total Estimated Dead Load	15 psf

The total estimated dead load is conservative as it could not be confirmed if the original shingles had been removed prior to re-roofing with SBS. Furthermore, it is unknown whether the entire roof was sheathed with plywood prior to the SBS installation.

The design standards at the time did not include any provisions for earthquake loading. Furthermore, the National Building Code of Canada 1953 did not require buildings less than 50 feet tall to be designed for wind loads assuming they were constructed with a bracing system. The current building codes require that structures be designed with Lateral Force Resisting Systems (LFRS) to support these loads.

The snow, wind, and earthquake loads used to evaluate the structure have been calculated based on the BCBC 2024 and the application of Commentary L of the NBCC 2015.

6.1.1.1 NBCC 2015 Commentary L

Commentary L outlines the application of Part 4 of the NBCC for the structural evaluation and upgrading of existing buildings to ensure a level of performance that is consistent with the intent of the current NBCC.

Buildings which have been evaluated against the guidelines outlined in this commentary are generally considered acceptable even though they may not specifically meet all aspects of the current building code.

The commentary outlines a systematic approach to determine the minimum reliability level for a building based on its use, occupancy, and past historical performance. Based on the determined reliability level, the load factors are relaxed, while meeting the basic requirements for life safety and building performance as outlined within Part 4 of the NBCC.

Based on the building having an Assembly occupancy, a maximum number of people exposed to risk associated with structural failure exceeding 100, and a record of satisfactory past performance, the building is assessed to have a level 4 reliability level.

The following table is referenced from Commentary L, NBCC 2015, and outlines the load factor relaxations considered in the structural evaluation. Note that reliability level 5 is the highest and represents conformance to the current building code.

Principal Load Factors for the Structural Evaluation of Existing Buildings Other than Post-disaster Buildings

Reliability Level(1)	Load Type				
	Dead Load		Live Load(2) or Snow Load	Wind Load	Earthquake Load
	Active	Counteractive(3)			
	Principal Load Factors				
5	1.25	0.90	1.50	1.40	(4)
→ 4	1.20	0.92	1.40	1.30	(4)
3	1.15	0.95	1.30	1.20	(4)
2	1.11	0.97	1.20	1.10	(4)
1 or 0	1.08	1.00	1.00	1.00	(4)

(1) The reliability level is the sum of the indices for system behaviour, risk category and past performance in Table L-4.
(2) A reduction in the live load factor may be justified if the live load in question is controlled (e.g., a liquid in a storage tank); however, the reduced load factor must not be less than the smallest value in the Table.
(3) The counteractive value applies when the dead load acts to resist failure.
(4) See the Commentary section titled Earthquakes (Paragraphs 39 to 48) for guidance on earthquake loads.

Based on the above table, the load factors used in the structural evaluation are as follows:

- Dead Load Active 1.2
- Dead Load Counteractive 0.92
- Live/Snow Load 1.4
- Wind Load 1.3
- Earthquake Load 1.0

At present there is no requirement in Canada to seismically upgrade a building that is not being renovated. Buildings undergoing large renovations are required to be upgraded to various seismic force levels depending on their existing force resistance level.

6.1.1.2 Snow Loading
The current building codes require arched roofs to be designed for balanced, unbalanced, and partial loading.

The balanced load case assumes equal snow load across the full area of the roof.

The unbalanced load case assumes the displacement of snow from the windward side of the roof peak to the leeward side of the roof peak such that the snow load is zero on the windward side and increases from the peak to the eave of the leeward side of the roof.

The partial load case assumes 50% of the uniform snow load on half the roof and 100% of the uniform snow load on the remaining half of the roof.

Additional snow loading conditions are mandated where multi-level roofs are present due to drifting snow.

The arena roof steps down to a flat roof at each end. The east and west additions step down further from the flat arena roofs resulting in significant drift loads.

The snow loads used in the structural evaluation are as follows:

- Balanced 60.6 psf
- Unbalanced 0 psf @ peak increasing to 148.3 psf on leeward eave.
- Partial 30.3 psf on windward half, 60.6 psf on leeward half.
- Drift Loads
    - At arena arched to flat roof 237.7 psf with a drift length of 37’-6”
    - At arena flat roof to additions 106.5 psf with a drift length of 12’6”

6.1.1.3 Wind Loading
The wind loads were calculated based on the provisions of Commentary I in the BCBC 2018 using an hourly wind pressure having the annual probability of occurrence of 1-in-50 for the City of Salmon Arm equalling 8.1 psf.

6.1.1.4 Seismic Loading
Figure L-1 in Commentary L was used as the basis for earthquake loading considerations in the structural evaluation. Level 3 forces, which correlates to the use of spectral response acceleration values with a probability of exceedance of 5% in 50 years (1/1000 per year) were used.

The spectral response acceleration values used are as follows:

- Sa(0.2) 0.161
- Sa(0.5) 0.174
- Sa(1.0) 0.135
- Sa(2.0) 0.0895
- PGA 0.0654
- PGV 0.137
- Site Class E

6.1.2 Design Results

The structural evaluation of the arena trusses and columns, with the application of the design loads discussed above, have been included in Appendix E.

6.1.2.1 Superstructure

The evaluation of the superstructure compared the tension, compression, and bending moment capacities of the structural members against the respective factored forces. The general design analysis results of the structural members are summarized below:

- Arena
  - The rafters are 50% over capacity in bending during balanced loading. The remaining load cases exceed this value.
  - The truss top chords are generally 50% over capacity in compression and 200% over capacity is combined bending and compression.
  - The truss bottom chords are generally 75% over capacity in tension and 200% over capacity is combined bending and tension.
  - 70% of the truss webs fail in either compression or tension.
  - 75% of the truss web to chord connections fail.
  - The columns have sufficient capacity to support the current design loads.
  - The column to truss connections are 50% over capacity in tension.
  - The column to foundation connections are 80% over capacity in tension.
- East Addition
  - The tongue and groove roof planking are structurally adequate to support the current uniform snow load conditions, but not the drift load cases.
  - The roof beams have adequate strength to support the current design loads.
  - The roof beam support columns have adequate strength to support the current design loads.
- West Addition
  - The rafters are structurally adequate to support current uniform snow load conditions, but not the drift load cases.

The CMU walls of the west addition and the wood framed exterior walls of the east addition are generally in good condition and can adequately support the current design loads.

6.1.2.2 Foundation

The bearing pressure of the soil assumed in the original design of the arena was back calculated based on the original design loads and foundation pad sizes. A factored soil bearing pressure of 4000 psf would have been required to support the original design loads. Based on the current design loads a factored bearing capacity of 5,300 psf is required.

The foundation excavations and underlying in-situ soils exposed during the site investigation were reviewed by Evertek Engineering. Based on Evertek's investigation the in-situ soils have an estimated factored bearing strength of 2,500 psf. Therefore, the existing footing pads are approximately 200% over capacity under current design standards.

The preliminary geotech report prepared by Evertek has been included in Appendix F. The report provides further discussions regarding expected foundation settlements.

The east and west addition foundation sizes are unknown and therefore have not been evaluated in detail.

7.0 Discussion

The building envelope of the arena is in a failed condition and requires a full replacement. The building envelope of the east and west additions are in fair condition and can likely provide an additional 5-10 years of service with regular maintenance.

The arena structure is in very poor condition requiring immediate interim repairs to prevent collapse, and significant structural repairs and reinforcing throughout to meet current building code regulations. The east and west addition structures are in fair condition and can likely provide an additional 10-15 years of service with reinforcing and regular maintenance.

7.1 Building Envelope

7.1.1 Arena

The building envelope components are beyond the expected service life and replacement is necessary to provide protection of the structure.

The outer thirds of the arena roof membrane have no remaining sanded surfacing. The sanded surface is intended to protect the bitumen membrane from mechanical damage, weathering, and UV exposure. The rate of deterioration can be expected to increase due to the loss of the sanded surface.

The flat roof sections are generally in fair condition with the exception of some blistering, bond failure at headwalls, and poor drainage. Differential settlement of the building has resulted in poor roof drainage causing rain and snowmelt to accumulate on the roof.

The cedar lap siding does not provide adequate protection of the structural components of the exterior walls which has led to significant deterioration of primary structural elements. The lack of a weather resistant barrier (W.R.B.) between the structure and the lap siding has exacerbated the deterioration of the structure.

The lack of a perimeter foundation wall and adequate waterproofing along the base of the exterior walls has led to water ingress causing decay of the lap siding, primary structural members, and secondary structural members.

The painted plywood gable end sheathing of the flat roof sections is deteriorated and does not adequately protect the primary and secondary structural elements from moisture ingress.

The updated lap siding on the south side of the arena is in fair condition with the exception of the area within 12 inches of grade. Hardi-plank siding in not intended to be installed within 6 inches of grade. Premature deterioration of the siding within this area can be expected. Early signs of deterioration are evident along the base of the wall as the OSB backing to which the siding is mechanically fastened to has significant decay.

7.1.2 East Addition

The east addition building envelope components are generally in fair condition.

The SBS roof appears to be watertight as no leaks were observed inside the building. Poor drainage is evident as water and snowmelt accumulates on the roof. Poor drainage is of moderate concern as it can lead to premature failure of SBS roofing systems.

The exterior stucco finishes are intact and provide adequate protection against water ingress.

7.1.3 West Addition

The west addition building envelope components are generally in poor condition.

The vertical upturn of the membrane at the headwall of the arena gable has delaminated from the plywood sheathing and blistering of the membrane has occurred. Blistering occurs when the membrane delaminates from the substrate and a ‘bubble’ of air is formed between the membrane and the substrate.

The SBS roof appears to be watertight as no leaks were observed inside the building. Poor drainage is evident as water and snowmelt accumulates on the roof. Poor drainage is of moderate concern as it can lead to premature failure of SBS roofing systems.

The CMU walls are mainly intact and provide adequate protection against water ingress.

7.2 Structural

7.2.1 Arena

The split rafters observed randomly throughout the arena can be attributed to drying shrinkage and bending stress failures.

Drying shrinkage refers to the shrinkage of wood members during the drying process. During drying, stresses develop in the wood fibres resulting in cracks. Common types of cracking include checks and splits.

Checks occur lengthwise along a wood member, typically parallel to the grain, and are usually shallow. Checks are typically superficial and do not significantly affect the strength of wood.

Splits involve the separation of wood into two separate pieces of wood and cause a significant reduction in strength. Splits generally occur as a result of overstressing, impact, or defects such as knots, however, excessive drying can also cause splitting.

The split rafters were located randomly throughout the arena, not concentrated, suggesting that the splits are related to excessive drying and defects rather than overstressing. Most of the rafters have performed well throughout the life of the building with no visible defects besides staining.

The defects observed on the top and bottom chords of the trusses include minor checking and delamination. The checking observed can be attributed to drying shrinkage as discussed above.

Glue-laminated timber, also known as glulam, are structural engineered wood products made up of multiple layers of wood glued together. Delamination of glulam members refers to the separation of the layers resulting in loss of structural integrity. Common reasons for delamination include moisture exposure, manufacturing defects, mechanical damage, over stressing, and decay.

The minor delamination observed in the glulam truss chords can be attributed to past moisture exposure and possible overstressing.

Moisture staining was observed throughout the arena. Based on the background information, the arena has a history of dealing with condensation related to the ice rink. The staining observed can likely be attributed to the historic condensation challenges and past roof leaks.

Delamination of the chords due to overstressing is also probable as the allowable material stresses used in the 1950s for the design of glulam members was overestimated by approximately 30% This has led to numerous failures of large span timber trusses in the past.

Approximately 50% of the truss webs were observed to have end-splits. The splitting can be attributed to drying shrinkage, as discussed earlier, and stress related failures. The ends of the webs are cut perpendicular to the grain increasing drying potential and therefore more susceptible to splitting. Furthermore, the split-ring connections between truss chord and webs concentrate the stresses near the end of the webs often resulting in splitting.

The capacity of split ring connections is reduced in locations where the split in the wood member passes within ¼" of the split ring, the split extends through the full depth of the member, and measures 3/64" (1.2mm) in width at the split ring. The capacity reduction is related to the loss of bearing area between the split ring and the wood member. The splits observed in the truss webs generally pass through the split rings, extend through the full depth of the member, and measure at least 3/64" in width at the split ring.

The checking of the arena columns can be attributed to drying shrinkage as discussed earlier. The checking is extensive and of moderate concern since the checks extend to the end of the members propagating into splits at the end connections.

The splitting observed at the end of several columns can be attributed to drying shrinkage and overstressing. Like the truss webs, the cut ends of the columns have a higher drying potential due to being cut perpendicular to the grain. The bolted connections at the base of the columns and the split ring connections at the top of the columns are areas of stress concentrations which have exacerbated the splitting at the column ends. Splits extending through the bolted and split ring connections can be expected to decrease the capacity of the connection as discussed earlier.

The deterioration of the structural column bases and grade level struts spanning between columns is a direct result of inadequate ground clearance and protection against moisture. The struts and column bases on the east half of the arena are at or below grade, exposing them to moisture, and leading to decay. Furthermore, the damaged downspouts discharge water directly against the base of the building exacerbating moisture exposure.

As discussed in the geotechnical report, the anticipated settlement of the foundations throughout the life of the building is approximately 4 inches. Although not addressed in detail in the background documents reviewed, the settlement has likely been a contributing factor to the column base elevations and poor site drainage.

The decay of the column bases has resulted in the settlement of the trusses on the east half of the building. Most notably, truss H, which has settled approximately 4 inches. The differential movement between truss G and truss H has resulted in stress concentrations in the truss bracing resulting in localized buckling of the vertical sway bracing. The settlement has also been the cause for poor roof drainage discussed earlier.

The moisture content of the wood framing throughout the arena generally ranged from 8-10%, well within the 19% maximum outlined in the building code. The base of the columns on the east half of the arena had moisture readings over 28%, the average fibre saturation of wood. Generally, decay and fungi growth will begin as moisture content exceeds fibre saturation.

7.2.2 East Addition

The roof and wall framing of the east addition could not be observed due to exterior and interior finishes. In these cases, the wall and ceiling finishes are reviewed for defects and misalignment. Structural issues will typically manifest as cracks and defects in the wall and ceiling finishes. Misalignment and large deflection of walls and ceilings typically raise concerns regarding the structure.

Based on limited observed defects in the ceiling and wall finishes, the structure is considered to be performing well.

The damage of the south timber roof support column in the reception area appears to be related to mechanical damage. The damage has reduced the cross-sectional area of the column decreasing its axial capacity.

7.2.3 West Addition

The roof framing could not be observed due to exterior and interior finishes. The roof and ceiling finishes were visually reviewed for defects, misalignment, and excessive deflections. Based on limited observed defects in the ceiling and roofing finishes, the roof structure is considered to be performing well.

Defects of CMU walls considered to be of structural consequence typically include step cracking along grout joints, dislodged blocks, lateral and vertical movement of walls, and grout deterioration.

The minor step cracking observed in the west addition walls is likely a result of differential foundation moments and not of structural consequence.

7.3 Roof and Site Drainage

The roof drainage throughout the facility is in failed condition due to building settlement, column decay, lack of maintenance, and vandalism.

The eavestroughs serving the arena have significant debris accumulation reducing flow capacity and clogging the discharge openings into the downspouts. The eavestroughs are prone to overflowing during intense rainfalls.

The downspouts are damaged throughout with many missing sections within 10 feet of grade. The missing sections can likely be attributed to vandalism and mechanical damage during large windstorms. The damaged downspouts at the northeast corner of the arena are a result of settlement of the building due to column decay discussed earlier.

Several of the downspouts no longer drain directly into the stormwater collection system along the north and south sides of the building. The roof runoff drains directly against the base of the building resulting in ponding due to inadequate site grading.

The grading around the east and west additions generally slopes away from the building. The grading along the north side of the arena is relatively flat with minimal positive drainage away from the building. The grade along the south side of the area slopes towards the building. The poor grading on the north and south sides of the arena results in the accumulation of rainwater and snowmelt along the building. The 1999 building information report noted that the site was relatively flat without positive drainage away from the building. This would suggest the site drainage has likely been inadequate throughout the life of the building. The building settlement discussed in the geotech report has also been a contributing factor to the poor site drainage.

The inadequate roof and site drainage has exacerbated the deterioration of the building envelope and primary and secondary structural elements.

7.4 Structural Evaluation

The facility does not meet the current design standards for resistance against vertical and lateral loads.

The structural capacity of building materials and design loads on structures have undergone significant changes in the last 7 decades as building codes and standards developed through analysis of historical data, testing, research, and development.

The most notable changes in relation to the arena include the decrease in allowable stresses of graded timber, unbalanced snow loading conditions, and consideration of snow accumulation at stepped roofs.

Notwithstanding the application of reduced load factors based on Commentary L of the NBCC 2015, the building is structurally inadequate to support current snow, wind, and earthquake loading.

As discussed earlier, Commentary L allows the consideration of past performance in the evaluation of existing structural members. Considering the past performance of the arena rafters, their current condition, the impact of a local failure, and the associated risk to occupants, the arched roof rafters can be considered acceptable. A local failure of the rafters is not expected to cause a catastrophic rupture or collapse of the building. Notwithstanding, the rafters are under capacity and can be expected to undergo significant deflections and deformations under high snow loads which could cause damage to the roof membrane.

The design results for the typical arena truss, trusses B through G, have been presented in Appendix E. The results for trusses A and H have been omitted as these two trusses are grossly inadequate due to the drift loading on the flat roof areas of the arena. Reinforcing of trusses A and H is not feasible, and replacement of the flat roof areas will be required.

The design results include the original truss forces, new truss forces, truss capacities, truss unity checks, and connection unity checks. Structural elements with a unity of 1.0 or less are considered to be structural adequate. Structural elements with a unity greater than 1.0 are considered to be structural inadequate.

Two unity checks have been included for the tension and compression load cases, axial and combined axial and bending. The axial unity checks consider axial member forces only, axial and bending unity checks consider axial and bending forces occurring simultaneously. The latter case would most notably apply to the top truss chord since the rafters bearing on the chord cause bending forces.

It should be noted that a unity between 1.0 and 2.0 doesn’t necessarily represent a member failure. The calculation of the applied loads includes load factors, as discussed earlier, which inflate the design loading to provide a level of safety. Furthermore, member capacities are reduced with the application of resistance factors, further increasing the level of safety. The resulting range between the ‘design capacity’ and ‘failure capacity’ can be considered a ‘no go zone’. Once in this zone, structural members can undergo excessive deflections and other serviceability issues prior to reaching their failure point. An example of this would be the buckling observed on the vertical sway bracing between trusses G and H. Although the braces have not physically broken, they are considered to be failing due to excessive deflection related to buckling.

Further consideration must also be given to members with unity checks exceeding 2.0. Failure of these members would be expected should a historic snow event be followed by high winds. The reported unity checks are based on worst case loading conditions which would represent a 1-in-50-year snowfall event followed by a 1-in-50-year wind event. Statistically, this weather scenario would have occurred 1.34 times since the original construction of the arena. Considering a catastrophic failure related to roof loading has not been historically recorded or observed, the building has likely never experienced this worst-case loading scenario.

Failure of a truss would likely result in the catastrophic failure of a large portion of the arena roof or possibly a complete collapse. Therefore, reinforcing of the trusses to meet the current building code design loading would be required. Cost effective reinforcing methods of bowstring timber trusses include post-tensioning of the bottom chord with steel cables, replacement of webs with split ends, increasing the net area of the truss members, increased truss bracing, and adding gussets to connections.

The arena columns are generally structurally adequate with the exception of the columns with observed decay. The decayed posts will require repair/replacement and the foundation raised to bring the column base elevations above grade.

Considering the level of effort required to replace the decayed columns, historic evidence of column decay and replacement, the root cause of the decay being related to the elevation of the foundations, and the inadequacy of the foundation based on soil bearing capacities, replacement of the foundation should be considered. In addition to increased pad and pilasters sizes at each column, continuous frost walls on strip footings should be used to elevate the base elevations of the exterior walls above grade and provide adequate support.

The arena roof and wall bracing do not meet the current code requirements for a LFRS. Additional truss bracing and a combination of braced bays and shear wall segments will be required to provide the necessary LFRS.

8.0 Recommendations

Based on the background information, detailed site assessment, structural evaluation, and the topics discussed herein, the building envelope and structural systems are generally considered to be in poor condition and at the end of their useful service life. Significant investment into the building envelope and structure is required prior to interim occupancy and extending the useful life of the facility.

It is the opinion of the undersigned that permanent repairs to the building envelope and structure with the intent of extending the useful service life of the facility is not a viable solution and replacement should be considered. This opinion is based on the extensive effort and cost to replace the existing foundation and the flat roof sections of the arena, the extensive truss repairs, and the replacement of the building envelope. Furthermore, the extensive remediation will trigger the requirement to upgrade the existing building to current codes in relation to fire and life safety. Costs related to the latter are not considered in this report as they will be further analyzed in the next phase of the project, the Life Cycle Assessment.

Two repair recommendations have been prepared. Repair recommendations for partial occupancy and repair recommendations for full occupancy. The intent of partial occupancy is to maximize the occupancy with the minimum level of repairs which will allow safe use of the building within certain weather conditions and seasons. The intent of full occupancy is to provide the minimum level of repairs which will allow safe use of the building year-round.

8.1 Repair Recommendation - Partial Occupancy

The following are the minimum structural repairs required prior to permitting occupancy in the arena:

- Jacking of truss H to be within 1 inch of truss G elevation.
- Installation of shoring at the north and south ends of trusses F, G, and H.
- Installation of cable cross bracing along each side of the arena.
- Clean eavestroughs along the north and south sides of the arena.
- Replace downspouts on north and south sides of arena and tie into existing stormsewer system.
- Installation of video surveillance on all roof areas.
- Installation of wind speed monitoring system.

Notwithstanding the implementation of the repairs outlined above, partial occupancy would be restricted to the following conditions:

- Occupancy limited to March through November.
- No occupancy permitted during snow accumulation on the roof.
- No occupancy permitted during forecasted and measured wind gust speeds exceeding 40 km/hr.
- Real-time data of the roof video feed and wind speed monitoring broadcasted to the facilities operation manager.
- Updating the City of Salmon Arm’s Operations Manual of the facility to include the conditions noted above.
- Annual visual assessment of the arena by a structural engineer, prior to occupancy following the winter season, to determine any significant changes in the building condition.

- The implementation of permanent repairs outlined in Section 8.2 below by the year 2030.

It should be noted that the shoring of trusses F, G, and H will impede on the playable turf area. It is anticipated that the shoring will take up approximately 240 square feet of the turf area at both the northeast and southeast corners of the playing field.

The expected cost related to the minimum repairs outlined above is $89,700. A class D estimate has been attached in Appendix G.

8.2 Repair Recommendation - Full Occupancy

The following are the minimum structural repairs/replacements required prior to permitting full occupancy of the building year-round. Due to the nature and extent of the structural repairs required in this recommendation, building envelope repairs/replacements have been included. The repairs/replacements outlined below will be considered a 'major renovation', as defined by the building code, triggering the requirement to upgrade the fire and life safety systems. A brief list of the fire and life safety upgrades that can be expected has been provided but is not considered to be an exhaustive list. Furthermore, the opinion of costs provided does not include the costs associated with the fire and life safety upgrades. These costs will be analyzed in the Life Cycle Assessment.

- Structural Repairs
    - Arena Roof
        - Reinforce all defected arena rafters.
        - Reinforce truss top chords.
        - Post-tension bottom truss chords.
        - Replace truss webs with split ends.
        - Reinforce truss webs.
        - Install gussets at web to chord connections.
        - Replace flat roofs on east and west ends of arena.
    - Arena Exterior Walls
        - Remove exterior walls along north and south sides of the arena.
        - Install new stud framed shearwalls supported on new concrete frost walls along north and south sides of the arena.
        - Install steel braced frames at east and west ends of arena.
    - Arena Foundation
        - Remove 10 feet of interior slab on grade along north and south sides of the arena.
        - Remove existing pilasters and pad footings.
        - Install new pad footings.
        - Install new continuous strip footings.
        - Install new frost walls along the north and south sides of the arena extending a minimum of 6" above grade.
        - Repair interior slab on grade.
        - Replace stormwater system and add weeping tile system.
    - East and West Additions
        - Reinforce rafters.
        - Replace damaged column in reception area of east addition.

- Building Envelope Repairs
    - Arena
        - Replace SBS roof membrane.
        - Replace eave and gable flashing.
        - Replace soffit and facia.

        - Replace eavestroughs and downspouts.
        - Replace plywood sheathing on truss gable ends.
        - Install weather resistant barrier on new exterior walls and gable ends.
        - Install new lap siding on exterior walls and gable ends.
        - Replace all doors and windows.
    - East and West Additions
        - Replace SBS roof membrane.
        - Replace parapet flashing, scuppers, and downspouts.
        - Repair wall penetrations.
        - Selective replacement of windows and doors.
        - Repaint exterior stucco and CMU surfaces.

- Anticipated Fire and Life Safety Upgrades
    - Installation of a fire sprinkler system.
    - Upgrades to the existing fire alarm system with integration of the sprinkler system.
    - Provisions for barrier free access.
    - Replacement of the emergency lighting system.
    - Upgrade of walls requiring fire-resistance ratings.

The expected cost related to the repairs outlined above is $2,778,000. A class D estimate has been attached in Appendix G. As previously mentioned, the costs related to the fire and life safety upgrades have been excluded from this cost estimate.

The building envelope repairs, and opinion of cost outlined above have not considered the heritage status of the facility. It is assumed that since the south side of the arena has been re-clad with Hardi-plank, no reservations exist against the replacement of the remaining exterior cladding with Hardi-plank lap siding. Should the heritage registrar require the exterior cladding replacement to be cedar plank siding, the recommendations and associated cost estimates would be amended accordingly.

9.0 Conclusion

Based on the historical data reviewed, site observations made, and the structural evaluation, the building envelope and structure are generally in poor condition and beyond their intended service life. Significant financial investment will be required to extend the useful service life of the building.

As stated in Section 8.0 Recommendations, it is the opinion of the undersigned that repairs to the building envelope and structure is not a viable option and demolition or replacement should be considered. Had the existing arena foundation consisted of a conventional perimeter foundation wall extending a minimum of 6 inches above grade, replacement of the foundation may not have been required. This would have increased the feasibility of repairs to extend the service life of the facility.

The repair recommendations and conditions outlined in Section 8.1: Repair Recommendation – Partial Occupancy, shall be designed and construction reviewed by a structural engineer prior to re-opening of the Memorial Arena. Use of the offices and welfare areas in the east and west additions will be granted once truss F, G, and H have been shored and all access points into the arena have been locked and barricaded. Access into the arena will only be granted once the shoring and bracing has been completed.

BAR Engineering and the undersigned reserve the right to amend the opinions outlined in this interim report following the completion of the Life Cycle Analysis, Demolition Estimate, and Replacement Estimate scheduled to be completed at the end of April 2024.

This interim report is not intended to provide an opinion regarding responsibility of any party in causing or contributing to the observed condition. Any comments or conclusions within this report represent the opinion of the undersigned, which is based upon the historic documents provided, the site assessment, the structural evaluation, professional engineering judgement, and industry standards.

This report has been prepared for the exclusive use of the City of Salmon Arm and their authorized users for the specific application outlined in this report. Any use which a third party makes of this report, or any portion of this report, is the sole responsibility of such third party or parties. BAR Engineering and the undersigned accept no responsibility for damages suffered by any third party resulting from unauthorized use of this report.

Respectfully Submitted,
BAR Engineering Co. Ltd.

Per:

J.L.Funk
54272
BRITISH COLUMBIA
ENGINEER
2024-03-25

Joey Funk, P. Eng.
Senior Engineer
Oakangan Divison

Reviewed by:

PERMIT TO PRACTICE
BAR ENGINEERING CO. LTD.
PERMIT NUMBER: 1001776
Engineers and Geoscientists BC

Whit Saretsky, P. Eng.
Senior Engineer
Buildings Division

10.0 References

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[9] Design Guide for Timber Roof Trusses, Timber Frame Engineering Council, Alstead, NH, 2020