STRUCTURAL CONDITION ASSESSMENT

SUDBURY COMMUNITY ARENA
240 ELGIN ST
SUDBURY, ONTARIO

Our Project No.: 23131A

October 16, 2023

Prepared for:
City of Greater Sudbury
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TABLE OF CONTENTS

1. GENERAL DESCRIPTION

The Sudbury Community Arena is a steel and reinforced concrete-framed, 2-storey building clad with uninsulated, composite masonry walls that was originally constructed c.1951. Significant additions were completed to the arena c.2000 (new stair additions in each corner) and c.2006 (club seating and lounge expansion). Numerous, relatively minor renovations and repairs have been completed throughout the building's history.

2. EXECUTIVE SUMMARY

The original arena structure is in fair condition with several deficiencies noted requiring repair, compensating construction, and/or replacement to maintain the performance level of the structural elements. Recommendations for additional investigation to better define some deficiencies, or to expose anticipated deficiencies, are outlined.

Structure associated with the additions was generally found to be in good condition where reviewed.

The original roof areas were designed and constructed to an older version of the National Building Code of Canada that did not require consideration of snow accumulation loads adjacent to high roofs and/or obstructions. Most of these areas are currently under snow watch (i.e., snow depths are regularly reviewed during the winter and snow is removed when depths exceed recommended safe limits) to defer the cost associated with the anticipated compensating construction.

Roof leakage is widespread and manifesting in staining at the underside of the wood deck at the high roof, deterioration at the northwest corner, and peeling paint on some of the perimeter masonry walls. Leaks should be identified and repaired in the short term with the understanding that wholesale replacement may be warranted.

Water infiltration through cracks and/or joints in the foundation walls was evident around the perimeter of the building and is anticipated to require extensive excavation, concrete repair, and waterproofing to mitigate continued water infiltration and structural deterioration.

Relatively minor brick repairs (i.e., replacement of cracked units and repointing of joints) is required at several locations on the building exterior to maintain the structural capacity of the wall and mitigate water infiltration and associated deterioration.

Further review and testing of the Main Entrance Lobby floor structure is recommended to address concerns regarding exposure to excessive levels of moisture and chlorides.

The precast panels at the Main Entrance are in poor condition and warrant replacement.

3. SCOPE OF WORK

3.1 Authorization

This report was prepared by Steve Cairns, P.Eng. of A2S Consulting Engineers at the request of Nick Zinger of the City of Greater Sudbury for the purpose of determining the general condition of the existing building structure.

3.2 Mandate

The purpose of our review is to complete a walkthrough of the existing building to facilitate a visual inspection of a rational sampling of building finishes, components (where applicable) and structural elements (where possible) so as to develop an opinion on the condition of the existing structural systems based on previous and current uses. This scope of work does not include an exhaustive review of observed conditions against all building code requirements, by-laws or other legislative requirements, all of which can change over time and may or may not retroactively apply to the building.

Our review does not include the removal of material (including finishes), exploratory probing or the use of specialty equipment unless specifically noted in our report.

Unless specifically noted, no structural analyses were performed on any component of the existing building structure. A2S Consulting Engineers assumes no responsibility or liability for the adequacy of the original structural design or the current capacity of the structural systems.

Only conditions observed and noted in our report can be assumed to have been reviewed during our walkthrough. All conclusions and/or recommendations pertaining to the condition of the building structure are based on extrapolations and interpolations of the conditions observed.

This report is intended to be read in its entirety, including the scope of work, limitations, and all appendices. No part of this report should be read in isolation or taken out of the context of the complete report.

3.3 Survey Method

The building was reviewed by Steve Cairns, P.Eng. of A2S Consulting Engineers on August 8, 2023. During our review, the weather was generally clear with an ambient air temperature of 19°C.

3.4 Information Provided

The following drawings were available for our review:

DATE	DRAWING TITLE/DESCRIPTION	AUTHOR
1950	Sudbury Community Arena – Architectural (incomplete)	J. B. Sutton
1974	Sudbury Arena Renovations – Structural (incomplete)	Morrison, Hershfield, Burgess & Huggins, Ltd.
Feb 2000	Stair Additions and Life Safety Retrofit – Structural	Halsall Associates Ltd.
Mar 2000	Sudbury Arena Floor Replacement – Structural	Northland Engineering Ltd.
Sept 2006	Event Enhancement Project – Structural (prelim or incomplete)	CDCD Engineering Ltd.
Sept 2006	Catered Lounge Renovation – Structural	Northland Engineering Ltd.
Nov 2013	Arena Ramp & Slab Repairs – Structural	J. L. Richards & Associates Ltd.
Dec 2015	Sudbury Arena Platform Upgrades	A2S Consulting Engineers
Jul 2016	2016 Emergency Repairs (Northeast Stair Addition)	A2S Consulting Engineers

DATE	DRAWING TITLE/DESCRIPTION	AUTHOR
Feb 2017	Steel Framing Repairs (Northeast Stair Addition)	A2S Consulting Engineers
May 2018	Sudbury Arena Entrance Repairs (Northwest Stair Addition)	A2S Consulting Engineers
Mar 2021	Precast Cladding Restoration	A2S Consulting Engineers

The following documents were available for our review:

DATE	DOCUMENT	AUTHOR
Mar 2016	Zamboni Slab Surface Repairs	A2S Consulting Engineers
May 2016	Review of Stair Additions	A2S Consulting Engineers
Nov 2018	Structural Review for Partial Re-Roofing Project	A2S Consulting Engineers
Mar 2019	Main Entrance Structural Review for Partial Re-Roofing	A2S Consulting Engineers
Apr 2019	Main Entrance Precast Concrete Panel Review	A2S Consulting Engineers
Jun 2023	CA Report	Asset Planner

Building Staff accompanied us during our review and provided commentary on issues related to building maintenance and/or their observations of current building performance. We cannot attest to the integrity, knowledge or accuracy of the persons interviewed.

4. OBSERVATIONS AND DISCUSSION – ORIGINAL BUILDING C.1951

4.1 High Roof

The original, high roof structure above the ice, Grandstands, and Concourse Level generally consists of nailed-laminated timber (NLT) decking spanning between structural steel beams and custom, structural steel trusses that span the width of the arena in the north-south direction.

4.1.1 NLT Deck

Unless noted below, the NLT deck was generally found to be in good to fair condition where reviewed.

We noted several areas of apparent staining at the underside of the NLT deck on all sides of the building, closer to the low side of the roof, suggesting excessive exposure to water from above. Continued and prolonged exposure to moisture will lead to rot in the wood, reducing the performance level of the structure. Further investigation is recommended to identify if exposure has damaged the wood members, which could warrant localized replacement.

Building Staff indicated that the High Roof leaks persistently at the southwest corner, where we observed staining at the underside of the NLT deck and peeling paint on the masonry wall below. It is critical that active leaks be addressed in a timely manner to mitigate the risk of prolonged exposure and associated deterioration of structural elements.

Approximately eight (8) openings have been cut through the NLT decking around the perimeter of the high roof. The purpose of the openings is not immediately obvious but may have been intended to act as passive vents to help keep the roof cooler in the winter and mitigate ice-damming. The size of the openings is such that new support beams around each opening, spanning between the steel roof trusses are warranted to reinstate the capacity of the NLT deck at these locations.

4.1.2 Structural Steel

The structural steel beams, trusses, and associated bracing members were generally observed to be in fair to good condition with no obvious signs of structurally significant deterioration or distress (i.e., excessive deflection, warping, buckling… etc.).

Light corrosion was observed on most structural steel members in the High Roof. None of the corrosion observed is indicative of an appreciable reduction in the performance level of the structure, in our opinion. Cleaning and repainting the steel structure, while not immediately or urgently required at this time, would improve the long-term durability of the members, and is recommended.

4.1.3 Steel Framing Embedded in Perimeter Walls

Steel columns around the perimeter of the building appear to have been encased within masonry pilasters, which are visible from Concourse Level on the interior (refer to *Figure 1*, below). Existing details describing the masonry wall assemblies are limited, but generally indicate that they are an uninsulated, composite masonry consisting of 90 mm clay brick and 140 mm and 190 mm concrete blocks.

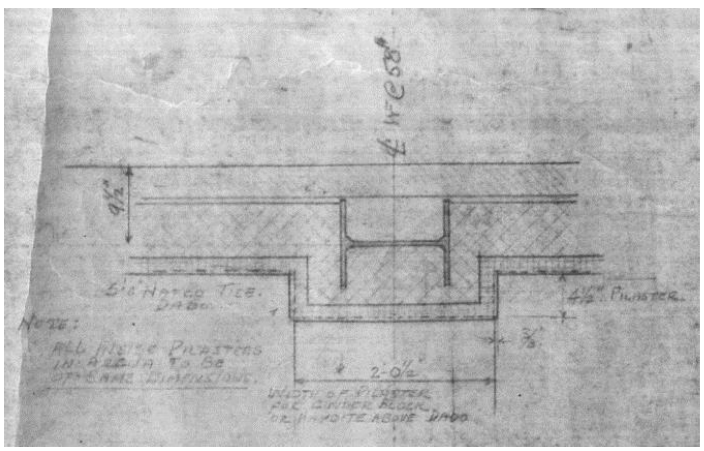

Figure 1: Anticipated masonry pilaster detail at perimeter steel columns (c.1951).

The lack of insulation in the walls and around the columns will allow the steel to cool as temperatures drop during the winter. Water vapour from the building interior that comes into contact with the steel will condense on the cold surface, creating an environment that could potentially promote accelerated deterioration of the steel and surrounding masonry.

Steel beams around the perimeter are similarly at risk. Where reviewed on the interior, we did not identify any obvious signs of deterioration or excessive condensation.

It would be prudent to expose several of these columns and beams by removing masonry on the interior (where applicable) and exterior of the building to determine the general condition of these members.

4.1.4 Snow Loads at Gable Ends

The National Building Code of Canada (NBC) first formalized the concept of increased snow loads due to drifting and blowing snow caused by higher roofs and obstructions in 1965. Our previous work on the lower roofs of the Sudbury Community Arena (refer to our reports of November 2018 and March 2019, attached in Appendix C) generally confirms that only the base, ground snow load was considered in the original design – as was typical of the period.

Building Codes are typically not retroactive, meaning that buildings designed to previous editions need not be upgraded to meet newer requirements. However, 'Commentary L: Application of National Building Code of Canada (NBC) Part 4 of Division B for the Structural Evaluation and Upgrading of Existing Buildings' of the Structural Commentaries (User's Guide – NBC 2015: Part 4 of Division B) identifies the NBC 1965 changes to drifting snow as a *benchmark change* that should be considered in the evaluation of all existing structures.

The gables at the east and west ends of the High Roof are large enough to meet the definition of a roof obstruction, resulting in increased, localized snow loads due to drifting snow that were unlikely to have been considered in the original design. The roof structures immediately adjacent to these gables are anticipated to be structurally deficient and in need of compensating construction to meet minimum Code requirements.

Policies and procedures should be developed to implement a *snow watch* protocol on the east and west ends of the High Roof. This entails regular monitoring of the depth of snow on the roof(s) in question with plans to remove then snow when depths reach 575 mm. As discussed in 4.2.3 below, a similar program is already in force at other roof locations.

Snow accumulation calculations considered in the Ontario Building Code (OBC) are inherently generic to accommodate a wide variety of conditions and often yield conservative snow load design values and extents. A wind-tunnel study of the building that considers prevailing winds, building geometry, thermal properties of the roof, and the surrounding landscape is likely to result in a net reduction to the extent of roof reinforcement required and is recommended.

4.2 Low Roofs

Original, low roof structures generally consist of a concrete slab (assumed to be 64 mm thick) and a metal pan deck spanning approximately 600 mm to 750 mm between open-web steel joists and structural steel beams.

4.2.1 Structural Steel

Building Staff have previously identified concerns regarding ongoing leakage in part of the 1951 Low Roof at the northwest corner of the arena, which does not yet appear to have been addressed. We noted evidence of excessive water infiltration and corrosion at the open-web steel joists and underside of the steel pan that appears to have worsened since our review in 2018 (refer to Appendix C). The continued and progressive deterioration of the structure is concerning but appears to be limited to a relatively small area nearest the northwest stair. Further review is recommended to confirm if the extent of deterioration observed is structurally significant and if it affects previously proposed reinforcing schemes to address snow loads as described in 4.2.3 below.

It is imperative that roof leaks be addressed in a timely manner to mitigate progressive deterioration of the structure. If left unaddressed, corrosion of the structural members could accelerate over time, eventually resulting in a reduction in the performance level requiring extensive compensating construction.

Our review of the roof structure over the Main Entrance Lobby in March 2019 was limited by the plaster ceiling finishes. Where reviewed, we noted generally light surface corrosion on the open-web steel joists, beams, and underside of the steel pan deck. We were able to access a small area of the west side of this roof during our current review and noted similar conditions, although our access and review was significantly restricted by existing mechanical systems.

We did not observe any obvious signs of excessive water infiltration in the ceiling finishes below the Low Roofs, nor were any known areas of ongoing leakage identified by Building Staff.

4.2.2 Steel Framing Embedded in Perimeter Walls

Steel members supporting the Low Roofs that are located in the perimeter walls are subject to the same risks as steel members supporting the High Roof as identified in 4.1.3 above. Although we did not identify any obvious evidence that may be associated with of excessive corrosion or water infiltration at these conditions, exploratory openings are recommended at several locations to verify the condition of the steel structure.

4.2.3 Snow Loads

As discussed in 4.1.4 above, we have previously confirmed (refer to Appendix C) that the 1951 Low Roofs were not designed to accommodate increased snow loads due to drifting from the adjacent, higher roof. Extensive reinforcement is anticipated throughout all original Low Roofs to meet the minimum life-safety standards outlined in the OBC.

It is our understanding that these roofs currently remain under snow watch and that policies and procedures are in place to ensure that snow depths on the Low Roofs never exceed 400 mm. Active snow watch should continue until either the roof structures are reinforced, replaced, or the building is unoccupied but a slightly deeper snow limit is permissible as discussed in 7.1.1 below.

4.3 Concourse Level and Grandstand Framing Systems

The suspended Concourse and Grandstand Floor structure generally consists of cast-in-place concrete slabs (thickness unknown) and beams spanning between structural steel beams and columns.

4.3.1 Concrete

We did not observe any obvious signs of distress or deterioration in the Concourse and Grandstand Floor framing, where reviewed.

Cracking was observed on the surface of the concrete at several locations across the Concourse Level but did not appear to be consistent with that associated with structural distress, but rather due to initial shrinkage in the concrete during construction, in our opinion.

4.3.2 Structural Steel

Structural steel members below the Concourse Level and Grandstands were generally observed to be in fair to good condition with areas of light, surface corrosion observed throughout.

While beyond the scope of a typical Structural Condition Assessment, we noted that the structural steel members did not appear to have any fire protection applied (i.e., spray-applied fireproofing, rated enclosures, intumescent paint… etc.). Fire safety requirements should be reviewed and confirmed by a qualified Building Professional with the expectation that all steel members supporting floors be protected in accordance with the minimum requirements of the OBC.

4.3.3 Main Entrance

Concrete elements in buildings near entrances with high levels of traffic from outside are often at risk of accelerated deterioration due to elevated exposure to chloride-contaminated water from de-icing salts. Chloride ions will penetrate deeper into the concrete over time and with frequent exposure, eventually reaching a critical concentration at the depth of the embedded steel reinforcement, or supporting steel structure below, resulting in an accelerated cycle of deterioration. If left unaddressed, deterioration of this nature will eventually reduce the performance level of the structure, requiring remedial measures, compensating construction, and/or replacement. The concrete and steel structure at, and around, building entrances are at elevated risk of experiencing this type of deterioration. As most patrons enter the building from the Main Entrance on the south side, we would expect that the structure in this area is especially vulnerable.

We could not review the underside of the concrete slab at the Main Entrance Lobby during our walkthrough due to the ceilings below. However, we did not identify any obvious signs of excessive water infiltration in the finishes. Similarly, we did not identify (nor were we notified of) any obvious issues associated with debonding of the tile flooring on the top side, which could be an indicator of issues in the concrete below.

Chloride ion content in concrete can be determined by extracting concrete cores and testing for chlorides at various depths along the core to develop a profile of the concentration levels. Once the concentration reaches a critical level at the depth of the reinforcement, the structure is at risk of accelerated deterioration in the presence of sufficient moisture. Typical remediation strategies to mitigate the effects of chloride contaminated concrete include localized replacement of corroded reinforcing steel, regular application of concrete surface sealers to mitigate exposure to moisture, and/or the installation of sacrificial cathodic anodes. In severe cases, contaminated concrete is replaced. We recommend sampling the chloride ion content in the slab at the Main Entrance at a minimum of three (3) locations to determine the chloride ion profile in the concrete.

The Main Entrance Stair structure could not be reviewed during our walkthrough as it is above the finished ceiling within the team Dressing Rooms and associated showers. However, we anticipate that they are framed with structural steel.

Building Staff noted that the stairs require regular maintenance to address concerns associated with tripping due to the stair tread nosing plates separating from the terrazzo infill. We did not identify any locations where the nosing plate was not reasonably tight to the terrazzo during our walkthrough but did note some cracking in the terrazzo at several treads.

It is reasonable to expect that the stair structure is similarly exposed to elevated levels of moisture and chlorides, like the Main Entrance slab. Corrosion of steel elements in the stair treads could manifest in the deformation of the nosing plate and/or cause cracking in the terrazzo finish. Review of the stair structure from below is highly recommended. Openings in the ceilings below will be required to sufficiently expose the structure to facilitate further review.

4.3.4 East Entrance (Minto Street)

We noted evidence of excessive exposure to moisture and corrosion of the steel-framed stair members from below at this location. Exposed flanges of steel beams at the stair landing were showing signs of localized corrosion but we did not identify any obvious section loss that would constitute a reduction in the performance level of the structure.

The underside of the concrete landing slab showed signs of relatively minor water infiltration through small cracks but was not extensively cracked nor displaying surface delaminations that are often associated with accelerated deterioration of the reinforcing steel. Similar to the Main Entrance slab discussed above, we recommend a minimum of two (2) concrete samples for chloride ion testing at this location.

4.4 Foundations

The limited existing architectural drawings available indicate that the original building foundation generally consists of reinforced concrete basement walls supported on reinforced concrete footings spanning between a combination of timber and steel driven piles. A conventional, reinforced concrete slab-on-grade (of unknown thickness) is noted throughout.

4.4.1 Concrete

The slab-on-grade was generally found to be in fair condition with several uneven areas and/or cracks observed throughout unless noted. Cracking generally seemed consistent with relatively minor settlements in the building foundations and/or soils below the slab.

Deterioration of the slab-on-grade in the area adjacent to the snow melt pit and tunnel access to the ice surface (due to abrasion from the Zamboni tires) has been an issue since at least 2013 (refer also to our report of March 2016, attached in Appendix C). Building Staff confirmed that ongoing maintenance currently includes levelling the slab depressions with a pavement repair product (FastPatch DPR by WVCO), which is not an appropriate concrete repair material. Concrete deterioration will continue, and the base slab will progressively wear down, likely resulting in increased areas needing repair. Partial replacement is anticipated in this area to reinstate the concrete slab over the long term.

The slab that slopes from the Zamboni area up to the exterior of the building is not a slab-on-grade, but rather a reinforced concrete slab that spans over the Mechanical Room below. Topside deterioration of this slab is relatively minor in comparison to that observed at the adjacent floor at this time, but it is critical to note that all repairs to this slab must be completed under the supervision of a Professional Engineer. Inappropriate repair methods and/or materials may exacerbate damage to the slab, resulting in a reduction in the performance level of the structure and more invasive repairs or complete replacement.

Leakage into the Basement Level, likely through the concrete walls, was generally observed around the perimeter of the arena:

- Exposed walls in the Mechanical Room at the north end of the building were wet in locations and showing signs of corroded reinforcing steel (i.e., rust staining on the walls, apparent concrete delamination… etc.).
- Building Staff noted that leakage is an ongoing issue along the west wall.
- Parging in the washrooms on the south face of the building was cracked and delaminating.
- Staining in the exposed walls under the East Stair (Minto Street).

Cracks in the concrete walls were generally observed throughout where exposed and are likely the source of most leakage into the Basement. The cracking could be associated with shrinkage in the concrete shortly after construction and/or movement / flexure in the walls under load. Where reviewed, the cracks did not appear to be severe enough as to be associated with a failure in the foundation system and are reasonably typical of concrete work of this vintage.

If left unchecked, this deterioration will accelerate over time as cracks slowly widen and/or embedded reinforcing steel continues to corrode, eventually resulting in a substantial reduction in the performance level of the structure. We anticipate a comprehensive crack repair and waterproofing remedial program will be required throughout to mitigate continued water ingress and associated deterioration of the foundation elements.

The old Coal Storage room (north and west of the Mechanical Room) is showing signs of extensive water infiltration through both the concrete roof slab and the basement walls. Crack repairs and waterproofing should extend up the walls and over the roof slab to mitigate continued water infiltration and structural deterioration. Excavation and waterproofing above the roof slab will be complicated by the large air-handling units currently installed above the slab, which will have to be temporarily relocated to complete the work.

The rink slab was being prepared for the ice-surface installation and was therefore not available for review during our walkthrough. Building Staff did not identify any concerns with the rink slab.

4.4.2 Piles

Existing drawings indicate that the concrete foundations are supported on a combination of structural steel piles (below the steel trusses and Concourse Level at the Main Entrance) and timber piles (around the perimeter of the arena and below the Grandstands). Piles are typically used on sites where the soils near the surface are incapable of either safely supporting, or would settle excessively under, the weight of a building. Piles are driven through the weak soils until they achieve a set refusal criteria, either due to skin friction against deeper, competent soils or when they come into contact with bedrock or very stiff tills.

Specific pile details are not indicated on the available drawings (i.e., capacity, dimensions, refusal and cut-off elevations, materials… etc.) and therefore, we cannot confirm if the timber piles were coated with a preservative prior to installation. The code in force during construction (NBC 1941) did not require that timber piles be treated if they were cut-off below the permanent ground water level, relying on the assumption that no significant biodeterioration mechanism exists when piles are submerged. The challenge with this assumption is that groundwater elevations can change over time, potentially exposing parts of the piles to conditions that could sustain damaging fungal or bacterial growth and associated rot in the wood.

The slow deterioration of timber piles would eventually result in excessive deflections/settlements in the foundations causing cracks to form in concrete and masonry walls, or other similarly brittle elements. This damage would likely progress, possibly even accelerating, with time as deterioration continues.

Although it is often difficult to definitively verify the root cause of cracks in concrete elements, as there are many factors that can and will contribute, we did observe cracks in the foundations and perimeter masonry walls (refer to 4.5 below) that could potentially be associated with differential movement in the foundations. Some initial settlement is to be expected in the years immediately following construction but movement that continues or occurs decades aft er construction can be an indicator of distress in the foundations. It is not possible to differentiate between new, existing, or worsening cracks during a single review.

In addition to ongoing monitoring to identify active cracks in the building, the Owner may wish to pre-emptively expose a representative sample of the timber piles for inspection and testing. This work should be completed by an individual or firm with extensive experience in the investigation and remediation of timber piles. The excavations necessary to sufficiently expose the piles will be intrusive and likely have to be completed during the summer months when building use is limited.

4.5 Masonry

As previously mentioned, the perimeter masonry walls generally appear to be an uninsulated, composite assembly consisting of 90 mm clay brick and a combination of 140 mm and 190 mm concrete masonry units (refer to Figure 2, below). The nature of a composite masonry wall is that both the brick and the concrete masonry are relied upon to work together to resist applied loads.

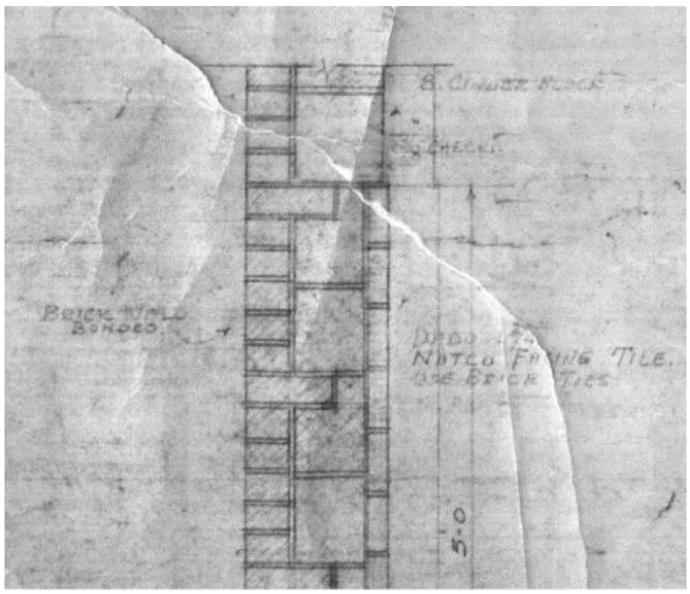

Figure 2: Typical composite masonry wall assembly c.1951.

Masonry walls on the interior of the building are, for the most part, partitions and not structural in nature. There will undoubtedly be walls throughout the building that were originally relied upon, or have since been modified, to carry some building loads.

4.5.1 Concrete Masonry Units

Exposed concrete masonry walls were observed to be in fair condition with predominantly vertical cracking noted throughout. Building Staff indicated that the walls are regularly repainted in all areas accessible to the public, which can make it more difficult to identify minor defects and/or evidence of recent movement.

Cracks can form in masonry walls in response to differential settlement in the foundations, a lack of control joints to control shrinkage shortly after construction, abrupt changes in wall geometry… etc. The severity and frequency of the cracking observed was not excessive or indicative of an immediate structural concern, in our opinion.

Peeling paint was observed at the tops of walls at several locations along the north and west walls, suggesting excessive water infiltration through the roof (most likely) and/or masonry cladding (less likely). Paint was peeling lower on the wall at the southwest corner, which was identified by Building Staff as being a known area of ongoing leakage.

We identified cracked and loose masonry over the opening to a Storage Room below the Grandstands at the northwest corner of the arena. Compensating construction is required to adequately support the remaining masonry over the opening and mitigate the risk of further damage and/or collapse.

4.5.2 Brick

Brick deterioration on the exterior of the building was generally minor, predominantly manifesting in spalled units near the tops of the walls due to water infiltration and subsequent freezing and thawing cycles. Cracks were observed on all sides of the building but did not appear to be excessive or indicative of an immediate structural concern. The most prominent cracks were observed on the north face of the arena, directly below the c.2006 Addition.

The original brick is a structural component of the composite wall assembly, which differs significantly from most modern masonry wall assemblies where the brick is a sacrificial veneer that does not contribute to the overall structural performance of the wall. Damaged and/or deteriorated brick masonry represents a reduction in the performance level of wall assembly and requires remedial intervention anticipated to consist of localized repairs or replacement as appropriate.

It is important to reiterate that the brick is a structural component on this building and cannot simply be removed and replaced to accommodate new insulation and/or waterproofing systems without compensating construction.

4.5.3 Precast Panels

Precast concrete fascia panels and accent trims adorn the south, east, and west elevations.

A2S attended the arena in February 2019 to complete a review of the precast panels at the Main Entrance in response to concerns raised by Building Staff (refer to our report of April 1, 2019, attached in Appendix C). We generally observed extensive panel cracking, concrete delamination, failed sealant between precast units and brick masonry, and damaged panel support elements. Immediate temporary supports were recommended and installed at two (2) locations, with further investigation and repairs recommended. It is our understanding that no further work has been done at this location. Precast panels at the Main Entrance continue to show signs of structural distress and generally poor performance. Complete replacement of the precast elements at the Main Entrance is recommended.

Precast elements on the east face of the building appear to be performing much better than those at the Main Entrance, with failed sealant between precast units and the brick masonry being the primary concern at this time. Failed sealant will promote water ingress and accelerate the deterioration of the backup and supporting elements. We identified two (2) locations where trim units were damaged on the east face and in need of repair.

4.6 Steel Canopy

A steel canopy structure addition has been installed over the Zamboni door on the south elevation of the arena. Existing documentation describing the canopy was not available for our review but is assumed to have been erected between 2009 - 2010.

The steel structure is supported on piled foundations at one end and bolted to the original concrete foundation wall at the other but is otherwise independent of the existing building.

All framing members are exhibiting light to moderate surface corrosion while the corrugated metal cladding was found to be in generally good condition. We did not identify any obvious deterioration consistent with a reduction in the performance level of the structure where reviewed. Deterioration will continue to progress, possibly at an accelerated rate, which will eventually compromise the structure if the steel is not cleaned and recoated.

4.7 Lateral Force Resisting System (LFRS)

Loads due to high winds and seismic events are resisted by a buildings lateral force resisting system (LFRS). The LFRS of the original arena generally consists of a combination of the steel trusses (in the north-south direction only) and unreinforced masonry shear walls between the structural steel columns around the building perimeter. We anticipate that the interior, masonry walls are generally non-loadbearing partitions that are not contributing significantly to the LFRS.

We note that there is distinct lack of masonry walls above the Concourse Level in the east-west direction. Numerous openings were made through the walls as part of the c.2000 additions and most of the original walls along the north side of the arena were removed as part of the c.2006 addition. We did not observe any obvious compensating construction that would reinstate the LFRS load path to accommodate the masonry wall removals.

LFRS design requirements in the NBC and OBC, specifically those associated with seismic events, have changed significantly since the design and construction of the original building. The code in force when the arena was originally constructed (NBC 1941) required that buildings be designed to resist forces due to earthquake only “in regions where destructive earthquakes are probable”, which Sudbury was not. Any plans to significantly extend the useful life of the arena should include for a seismic retrofit and upgrade of the existing structure to meet current standards.

A seismic retrofit would generally involve the construction of new braces and/or walls around and throughout the building to transfer lateral forces from the roofs and floors to the foundations. These new components would be rigidly connected to the existing structure, necessitating compensating construction at all tie-in points, and bearing on new foundation elements, including new piles.

5. OBSERVATIONS AND DISCUSSION – STAIR ADDITIONS C.2000

5.1 Low Roofs

The roof structures associated with the Stair Additions generally consist of a 38 mm deep steel deck spanning between a combination of structural steel beams and loadbearing concrete masonry units.

5.1.1 Structural Steel

The roof structure was observed to be in good condition with no obvious signs of excessive water infiltration, deterioration, or distress identified where reviewed.

5.1.2 Snow Loads

Existing structural drawings indicate that the roof structures were designed for loads significantly greater than the minimum design snow load (apparently to accommodate future, vertical expansion) but are still theoretically less than the maximum anticipated snow accumulation loads due to drifting, at some locations. It is highly unlikely that snow loads will exceed the total design load of these roofs and an active snow watch is not warranted at this time.

5.2 Concourse Level

The structural floor system in the stair additions generally consist of an 89 mm thick, reinforced concrete slab on 38 mm deep, composite steel deck spanning between structural steel beams and loadbearing concrete masonry walls.

5.2.1 Structural Steel

We attended the building in February 2016 to review the condition of the suspended floor structure at these entrances in response to concerns raised by Building Staff due to the level of corrosion observed on the steel framing (refer to our report of May 2016, attached in Appendix C). Following our review, repairs and compensating construction were completed at the northeast and northwest entrances. Where reviewed as part of the current scope of work, we did not identify any obvious signs of excessive water infiltration and/or continued corrosion of the steel structure on the interior of the building.

Similar structures at the southeast and southwest corners of the arena were deemed to be within acceptable limits in 2016, with a recommendation to remove all corrosion product, recoat the steel, and waterproof the foundation wall to mitigate continued deterioration. During our current review, we noted these recommendations have not been implemented and that the level of corrosion has progressed noticeably since our last visit to site. Compensating construction similar to that completed at the northeast entrance may now be required (i.e., replacement of existing header connections to the steel columns, localized column reinforcement, waterproofing on the exterior, elastomeric coating on topside of the slabs, and reinstatement of the fireproofing). Further review is recommended to determine the current extent of deterioration.

The steel columns on the perimeter of the additions continue to corrode at the base as they remain exposed to the exterior of the building. Significant modifications to the windows, doors, and structure would be required to properly protect the columns from excessive exposure to moisture over the long term. Alternatively, regular and potentially increasingly frequent maintenance will be required to ensure that column deterioration does not progress to the point where the capacity of the columns is compromised.

Installation of an elastomeric coating over the concrete slabs, as previously recommended, will require regular resurfacing to keep the structure protected from excessive exposure to moisture. Should maintenance lapse, we anticipate that corrosion of the steel deck and beams, similar to that observed at the northeast corner, will manifest and require significantly more compensating construction to accommodate.

5.3 Foundations

The stair addition foundations consist of reinforced concrete basement walls supported on a 400 mm thick, reinforced concrete raft slab beneath the footprint of each addition. Raft slabs are commonly used on weak soils as they decrease the risk of differential settlements between elements bearing directly on the slab and their relatively large size minimizes the net increase in pressure on the bearing stratum.

Aside from the leakage at grade (discussed in 5.2.1 above), we did not identify any obvious signs of distress or excessive deterioration in the concrete elements. The foundations at the southwest and southeast additions should be waterproofed as previously recommended (refer to our report of May 2016, attached in Appendix C) to mitigate continued water ingress and associated deterioration.

5.4 Masonry

5.4.1 Concrete Masonry Units

Loadbearing masonry walls generally consist of unreinforced, 190 mm concrete masonry units. Where reviewed, the masonry walls were generally found to be in good condition unless otherwise noted.

Significant cracking was observed in the newer masonry of the southwest addition where it abuts, or straddles, the construction joint to the original building. We anticipate that the cracks are associated with differential settlement between the c.2000 Addition and original building foundations. Damaged masonry units should be replaced with new, cracks repaired, and proper expansion joints provided between the construction vintages to allow them to move independently. Periodic review of the masonry in this area by Building Staff is recommended to identify signs of continued cracking.

New cracking or worsening conditions may be indicative of continued movement in the foundations, which would warrant further investigation.

Step-cracking was observed in the parged masonry at grade, on the exterior of the southwest addition, which is further indication of excessive differential settlement in the addition foundations. All cracks should be repaired to mitigate water infiltration and associated deterioration and reviewed on a regular basis.

At the northwest stair addition, we observed gaps between the steel deck flutes and the masonry in the stair shaft at the Roof Level. Stair walls are typically fire-rated assemblies in which gaps would not be permitted. Unless otherwise confirmed by a qualified Building Professional, the gaps should be sealed with an appropriate firestop material to prevent fire spread from the building to the building exit stair.

5.4.2 Brick

Where reviewed, the brick on the additions was observed to generally be in good condition with no obvious signs of distress, movement, or deterioration.

Unlike the original building, the brick on the c.2000 Additions is a sacrificial veneer that can be removed and/or replaced without compromising the structural capacity of the wall assembly.

5.5 Lateral Force Resisting System (LFRS)

Each addition is independent of the original building above grade and relies on unreinforced masonry walls to resist lateral loads due to high winds and seismic events.

Future modifications to existing loadbearing walls in these areas will result in a reduction in performance level of the LFRS, requiring analysis with compensating construction anticipated.

6. OBSERVATIONS AND DISCUSSION – CLUB SEATING AND LOUNGE ADDITION C.2006

6.1 Low Roof

The roof structure of this addition consists of a 76 mm deep steel deck spanning between structural steel beams.

6.1.1 Structural Steel

The roof structure was observed to be in good condition with no obvious signs of excessive water infiltration, deterioration, or distress, where reviewed.

6.1.2 Snow Loads

Loads considered in the design of this addition are unknown as existing drawings were not available and the structure has not been verified and analysed as part of the current study. It should have been designed for snow loads, including those associated with drifting from the adjacent high roof, in accordance with the OBC 1997.

6.2 Concourse Level

Review of the newer floor structure was limited to that observed from the basement Mechanical Room as existing structural drawings were not available for our review. From this vantage point, the structure appears to consist of a cast-in-place concrete slab spanning between new and existing steel beams.

We did not observe any obvious signs of distress or deterioration, where reviewed.

This addition is far enough from a building entrance that we do not anticipate corrosion associated with excessive exposure to moisture and/or chlorides to be a risk to the structure. Exposure to moisture is still a potential concern at the building perimeter (as it is in any building), but no evidence of excessive water infiltration was noted during our review.

6.2.1 Structural Steel

Newer steel beams have been installed below the concrete slab in the area above the Zamboni ramp that exits on the north side of the building. We anticipate that these beams were installed as compensating construction to accommodate the change in occupancy above from a low roof to part of the Concourse Level. A Building Professional should be retained to confirm if the beams meet the minimum required fire-rating with the expectation that they be coated with new spray-applied fireproofing material.

6.3 Foundations

The limited existing information available suggests that this vertical expansion was constructed on the existing arena foundations. We did not identify any obvious signs of compensating construction at the foundation level.

6.4 Lateral Force Resisting System (LFRS)

Steel braces were visible in the north wall of the addition and are assumed to transfer lateral loads in the newer roof framing to the original perimeter masonry walls below.

As noted previously, we did not identify any obvious compensating construction to account for the removal of several existing masonry wall panels between the existing steel columns. This addition may require compensating construction in any seismic retrofit that may be implemented in the original structure, as discussed in 4.7 above.

7. RECOMMENDATIONS

Where noted, recommended timeframes for further investigation/remediation are provided. Timeframes provided are not to be construed as the definitive remaining lifespan of a particular system, but rather to help identify the urgency of a particular recommendation.

All compensating construction is to be designed by a Professional Engineer, installed by a qualified Contractor, and with the approval of the Chief Building Official.

7.1 Immediate

Immediate recommendations are generally associated with obvious deficiencies in the building or structural elements that are likely to affect the safety of building occupants and should be addressed by the Owner as soon as possible. Patently obvious structural deficiencies identified during our review that, in our opinion, pose an immediate threat to public safety, are noted and will be reported to the Chief Building Official or Authority Having Jurisdiction.

Deferral is not recommended for any of these recommendations.

7.1.1 Maintain Snow Watch

In addition to the low roofs that are currently under snow watch (refer our reports of November 2018 and March 2019 in Appendix C), the gable ends of the High Roof should be similarly monitored during the winter and snow removed once it reaches a depth of **575 mm**. Refer to Figure 3 below for roof areas requiring snow watch policies and procedures.

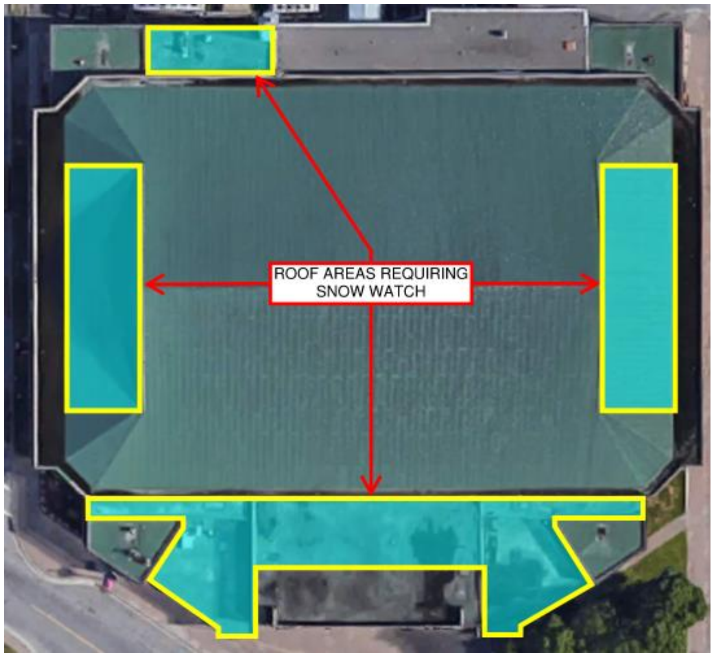

Figure 3: Sudbury Community Arena roof areas requiring snow watch.

Recommendations included in our previous reports included removing snow once it reached a depth of 400 mm, which included an allowance for the weight of workers on the roof during snow clearing operations. The current, revised, recommendation does not include such an allowance but is within safe limits and will result in fewer snow removal events and can be used at all roofs where snow watch has been recommended.

Refer to sections 4.1.4 and 4.2.3 for discussion.

7.1.2 Confirm, Add, and/or Replace Fire Protection

Although some specific conditions have been identified, this report does not include an exhaustive list of all potential concerns associated with fire protection of the building structure. A qualified Building Professional should be retained to confirm the minimum fire protection requirements outlined in the OBC as they apply to the arena structure and to develop repair details to address any deficiencies, as appropriate.

Steel beams supporting the original concrete Grandstand structure and newer steel beams installed below the concrete slab over the Zamboni ramp do not appear to have any fire protection.

Spray-applied fireproofing material was removed from parts of the floor structure at grade in the southeast and southwest stair additions to facilitate review and repairs of the corroding steel in 2016. The fire-rating of this structure must be maintained to ensure that the building exits are serviceable in the event of a fire. Further review of these members is required as foundation leakage has not been addressed and the extent of corrosion appears to have worsened. As the addition of spray-applied fireproofing will hinder this review, the additional investigation should be carried out as soon as possible with the intent of replacing the fireproofing immediately following.

Gaps between the flutes in the steel deck and the masonry stair shaft were observed at the northwest stair addition. An appropriate firestop material should be installed between the flutes to prevent potential fire spread between the building and the exit stair.

Refer to sections 4.3.2, 5.2.1, 5.4.1, and 6.2.1 for discussion.

7.2 Short-Term

Short-term recommendations are generally associated with structural elements that are displaying some degree of deterioration or structural distress that may continue to worsen, possibly at an accelerated rate, and possibly resulting in an unacceptable reduction in the performance level of the structure if not properly addressed. These may also include items that are anticipated to require compensating construction but should continue to perform at their current level if conditions do not change.

Deferral may be possible by implementing regular monitoring, occupancy limits, temporary measures… etc. but must be discussed and considered on a case-by-case basis.

7.2.1 Address Roof Leakage

We observed evidence of leakage through the high roof (i.e., apparent staining on the underside of the NLT deck, peeling paint on the inside face of some perimeter masonry walls on the Concourse Level) and at a section of the low roof at the northwest corner.

A series of test cuts are recommended across the high roof, with a specific focus in the vicinity of masonry walls exhibiting peeling paint, to confirm areas of water leakage and to expose the top surface of the NLT deck in areas where staining was observed below. A thermal scan of the roofs will help to identify areas of leakage and to locate proposed test cuts.

Leaks should be addressed promptly to mitigate water infiltration and the associated risk of deterioration of the structural members. Continued exposure to moisture will eventually result in a reduction in performance level of the structure necessitating replacement and/or compensating construction.

Typical roofing systems have a useful life expectancy of approximately 20 years. We could not confirm the age of that currently installed but anticipate that reroofing is warranted throughout to ensure adequate protection of the building structure.

Refer to sections 4.1.1, 4.2.1, and 4.5.1 for discussion.

7.2.2 Review Corroded Low Roof Structure at Northwest Corner

Corrosion of the existing metal pan deck and open-web steel joists appears to have worsened since our review in 2018 (refer to Appendix C). Further review is recommended to determine the severity of the deterioration and develop compensating construction, which is anticipated.

Structural reinforcing details previously prepared to account for snow accumulation loads on this roof may no longer be appropriate.

Refer to section 4.2.1 for discussion.

7.2.3 Review Underside of NLT Deck

Areas of the NLT deck exhibiting evidence of exposure to moisture should be reviewed in more detail via lift from the Concourse Level. This review is anticipated to consist of a series of readings with a moisture meter and probing from below to identify areas experiencing rot.

This review will be limited in scope due to the access available from the Concourse. As noted in section 7.2.1 above, a thermal scan of the roofs will help to identify areas of leakage and to locate areas for up-close review. An expanded investigation may be warranted if the wood is found to be in distress beyond areas that are visibly water stained.

Refer to section 4.1.1 for discussion.

7.2.4 Reinforce Openings in NLT Deck

Compensating structure around each of the openings through the NLT deck at the High Roof is anticipated to consist of new steel beams connected to the existing steel frame.

Refer to section 4.1.1 for discussion.

7.2.5 Address Foundation Leakage

Evidence of water leakage through the perimeter foundation walls was observed during our review, and has been reported by Building Staff, throughout the arena (including at the southeast and southwest stair additions).

We anticipate that an extensive concrete repair and waterproofing program will be required to mitigate continued leakage and associated risk of deterioration to the building structure. This work will likely involve excavation around the building, crack repairs in the concrete elements, and the application of a waterproofing membrane.

Concrete repairs and waterproofing are similarly anticipated to the roof structure over the old Coal Storage room at the west end of the Mechanical Room. Excavation to expose the concrete slab will require the temporary removal of several, large air-handling units currently resting on grade, above the slab.

Refer to sections 4.4.1 and 5.3 for discussion.

7.2.6 Maintain Exposed Steel Columns at Stair Additions.

The bases of the steel columns at the c.2000 Additions will require frequent cleaning and reapplication of epoxy paint or zinc-rich primer to mitigate continued corrosion due to exposure to water and chlorides from de-icing salts. Columns that were repaired and recoated at the northeast addition in 2016 are already showing signs of continued corrosion at the base.

Refer to section 5.2.1 for discussion.

7.2.7 Install Lintel Over Storage Room Opening Below Grandstands

A steel lintel is required over the Storage Room door frame to support the masonry above. The remaining masonry will continue to crack and eventually shake loose with continued use of the door in its current condition.

Refer to section 4.5.1 for discussion.

7.2.8 Crack Monitoring

The cracks identified in the masonry walls of the southwest stair addition should be repaired and monitored for continued movement, which may be indicative of ongoing and excessive settlements in either the addition or original building foundations. Crack monitoring should be performed on a regular basis to help identify cracks that are worsening over time from those that cycle with temperature.

It may be worthwhile to expand the crack monitoring program to include some of the more pronounced cracks observed in the c.1951 structure in hopes of identifying those associated with initial building shrinkage and/or settlement as compared to those that are active and may be attributable to continued movement in the foundations.

Refer to sections 4.4.2, 4.5.1, and 5.4.1 for discussion.

7.2.9 Brick Repairs

Cracked and spalling brick masonry should be replaced, and joints repointed as necessary throughout to reinstate the structural capacity of the wall assembly and mitigate water infiltration and associated deterioration of the structure.

Refer to section 4.5.2 for discussion.

7.2.10 Expose and Review Structure Below Entrance Lobby

As evidenced by the condition of the structure visible below the East Entrance (Minto Street), and the ongoing maintenance required at the adjacent stairs, the Main Entrance floor structure is anticipated to have been exposed to high levels of chlorides and water brought into the arena by users during the winter months. The risk of accelerated deterioration of the structure below the Main Entrance is elevated due to these potentially severe exposure conditions.

The structure below this area was not readily available for review during our walkthrough due to the finished ceilings above the bathrooms and Wolves changeroom. Selective removals of the finishes will be required to facilitate a proper review.

Refer to section 4.3.3 for discussion.

7.2.11 Repair and/or Replace Precast Elements

The original precast column enclosures and soffit panels at the Main Entrance are exhibiting signs of excessive cracking and differential movement relative to the supporting structure and should be replaced.

Removal of the panels will allow for a preliminary condition assessment of the steel structure back-up, which is otherwise not readily accessible.

Remaining precast elements on the east and west elevations should be re-caulked to mitigate water ingress and associated deterioration. Regular maintenance will be required to ensure caulked joints remain watertight. Damaged trim pieces should be similarly repaired to help mitigate water infiltration through the cladding.

Refer to section 4.5.3 for discussion.

7.2.12 Clean and Repaint Exterior Steel Canopy

The steel structure should be cleaned of all corrosion product and the surface prepared to receive an epoxy paint system or zinc-rich primer to mitigate continued deterioration. If not protected from the elements, the steel structure will eventually experience a reduction in performance level due to excessive corrosion, requiring compensating construction and/or replacement. Regular maintenance will be required to properly maintain the steel structure.

Refer to section 4.6 for discussion.

7.3 Long-Term

Long-term recommendations are generally associated with structural elements that are displaying some signs of deterioration, possibly minor structural distress, which is not anticipated to worsen significantly in the near future or at an accelerated rate. These are typically minor deficiencies that have developed slowly over the life of the structure or are associated with elements in the building with generally known life expectancies (e.g., roofing systems).

7.3.1 Repair Concrete Slab-on-Grade at the Snow Melt Pit and Tunnel Access to the Ice Surface

Ongoing deterioration of the slab-on-grade in the area regularly accessed by the Zamboni continues to deteriorate, primarily due to abrasion from the studded tires, and is being frequently patched with an inappropriate material by Building Staff. While relatively cost-effective in the short term, we anticipate that the lack of a proper repair will result in continued and expanding damage to the slab surface, eventually requiring a much larger area of repair and/or replacement.

Refer to section 4.4.1 for discussion.

7.3.2 Clean and Paint High Roof Steel

Generally light surface corrosion was observed on the structural steel framing at the High Roof. Cleaning the steel and recoating with new paint will help to prolong the service life of the structure.

Refer to section 4.1.2 for discussion.

7.4 Optional

Optional recommendations generally include additional investigation, testing, and/or analyses that the Owner and/or Stakeholders may wish to undertake to further explore the building structure to better understand potential limitations or verify the condition of structural elements that were not exposed or manifesting in obvious signs of distress to building finishes.

7.4.1 Investigate Timber Pile Foundations

It may be prudent to investigate and confirm the condition of the existing timber piles if considering expansion and extending the useful life of the arena. We strongly recommend retaining the services of an individual or firm with extensive experience specific to the investigation and remediation of timber piles to develop a scope of work and lead the investigation.

While we did observe some cracking in the foundation walls, some of which may be associated with movement in the foundations, we did not observe any obvious signs of excessive or global distress that we would anticipate in the event of widespread deterioration of the piles.

Refer to section 4.4.2 for discussion.

7.4.2 Investigate Steel Elements in Perimeter Masonry Walls

Steel members in uninsulated walls are at increased risk of deterioration due to condensation on the steel surfaces in freezing temperatures. While no obvious signs of distress or excessive deterioration were noted during our review, it would be prudent to confirm at some locations. This investigation is anticipated to include a series of openings on the interior and exterior faces of the perimeter masonry walls.

Refer to section 4.1.3 for discussion.

7.4.3 Reinforce Roofs for Snow Accumulation Loads

In lieu of ongoing snow watch procedures during the winter months, the existing roof structures could be reinforced to accommodate increased snow accumulation loads due to adjacent high roofs and/or obstructions. Ensuring that the structure is capable of safely resisting the anticipated snow loads without intervention is preferred to a prolonged snow watch.

Refer to sections 4.1.4 and 4.2.3 for discussion.

7.4.4 Complete Wind Tunnel Study

A wind tunnel study is able to more accurately describe the anticipated snow accumulation loads on building roofs, typically resulting in lower peak accumulation loads over smaller areas. We anticipate that this would result in less compensating construction in the existing roofs, possibly eliminating the need in some areas.

While the study and subsequent modelling can accommodate some changes in the existing building (roof thermal properties, for example), significant changes to the building geometry would render the results irrelevant and require further study and/or analysis. As such, this approach may not be feasible at this time if significant additions and/or modifications are anticipated.

An individual or firm that specializes in the field of wind tunnel modelling and finite area analysis would have to be retained to complete this specific study.

Refer to section 4.1.4 for discussion.

We trust that the enclosed information is adequate for your current needs. Please do not hesitate to contact us with any further questions or comments.

LICENSED PROFESSIONAL ENGINEER
2023.10.16
S.W. CAIRNS
PROVINCE OF ONTARIO

Sincerely,

Steve Cairns, P.Eng.
A2S Consulting Engineers

Attachments: Appendix A – Limitations (2 pages)
Appendix B – Photos (25 pages)
Appendix C – Previous Reports (39 pages)

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APPENDIX A – LIMITATIONS

“Consultant” in the following document refers to A2S Consulting Engineers.

- The scope of our work and related responsibilities related to our work are defined in our proposal and Conditions of Assignment.
- Any user accepts that decisions made or actions taken based upon interpretation of our work are the responsibility of only the parties directly involved in the decisions or actions.
- No party other than the Client shall rely on the Consultant’s work without the express written consent of the Consultant, and then only to the extent of the specific terms in that consent. Any use which a third party makes of this work, or any reliance on or decisions made based on it, are the responsibility of such third parties. Any third-party user of this report specifically denies any right to any claims, whether in contract, tort and/or any other cause of action in law, against the Consultant (including sub-consultants, their officers, agents and employees). The work reflects the Consultant’s best judgement in light of the information reviewed by them at the time of preparation. It is not a certification of compliance with past or present regulations. Unless otherwise agreed in writing by the Consultant, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity; it is written to be read in its entirety.
- Only the specific information identified has been reviewed. No physical or destructive testing and no design calculations have been performed unless specifically recorded. Conditions existing but not recorded were not apparent given the level of study undertaken. Conditions may differ from those observed, which were relied upon to develop our recommendations. Only conditions actually seen during examination of representative samples can be said to have been appraised and comments on the balance of the conditions are assumptions based upon extrapolation. Therefore, this work does not eliminate uncertainty regarding the potential for existing or future costs, hazards or losses in connection with a property. We can perform further investigation on items of concern if so required.
- The Consultant is not responsible for, or obligated to identify, mistakes or insufficiencies in the information obtained from the various sources, or to verify the accuracy of the information.
- No statements by the Consultant are given as or shall be interpreted as opinions for legal, environmental or health findings. The Consultant is not investigating or providing advice about pollutants, contaminants or hazardous materials.
- The Client and other users of this report expressly deny any right to any claim against the Consultant, including claims arising from personal injury related to pollutants, contaminants or hazardous materials, including but not limited to asbestos, mould, mildew or other fungus.
- Applicable codes and design standards may have undergone revision since the subject property was designed and constructed. As an example, design loads (such as those for temperature, snow, wind, rain, seismic, etc.) and the specific methods of calculating the capacity of the systems to resist these loads may have changed significantly. Unless specifically included in our scope, no calculations or evaluations have been completed to verify compliance with current building codes and design standards.
- Timeframes given for undertaking work represent our opinion of when to budget for the work. Failure of the item, or the optimum repair/replacement process, may vary from our estimate. This opinion is therefore given as a reasonable average approximation rather than a specific prediction.

- Qualified design professionals are required to perform additional evaluation (as necessary), design and general review during construction when carrying out the recommendations included in this report. Ongoing monitoring is required to confirm that repair or renewal measures are successful and to identify for changing conditions that would require increased levels of intervention or different repair / renewal strategies.
- Qualified contractors are required to implement any recommendations included in this report.
- Failure to implement the recommendations included in this report and/or failure to maintain building components appropriately could lead to ongoing and accelerated deterioration that may lead to unsafe conditions developing.

APPENDIX B – PHOTOS

Photo 1: Sudbury Community Arena (c.1951).

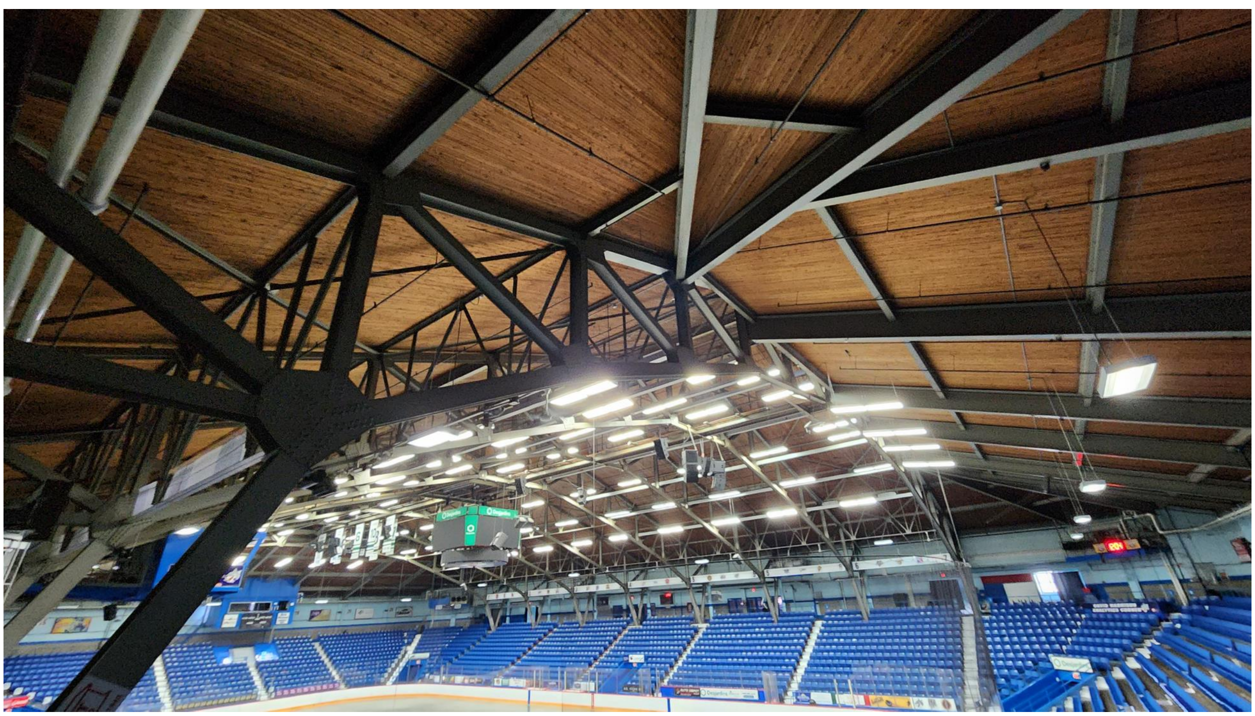

Photo 2: Typical High Roof structure (c.1951).

Photo 3: Typical High Roof structure exhibiting light surface corrosion (c.1951).

Photo 4: Water stains at underside of NLT deck (High Roof c.1951).

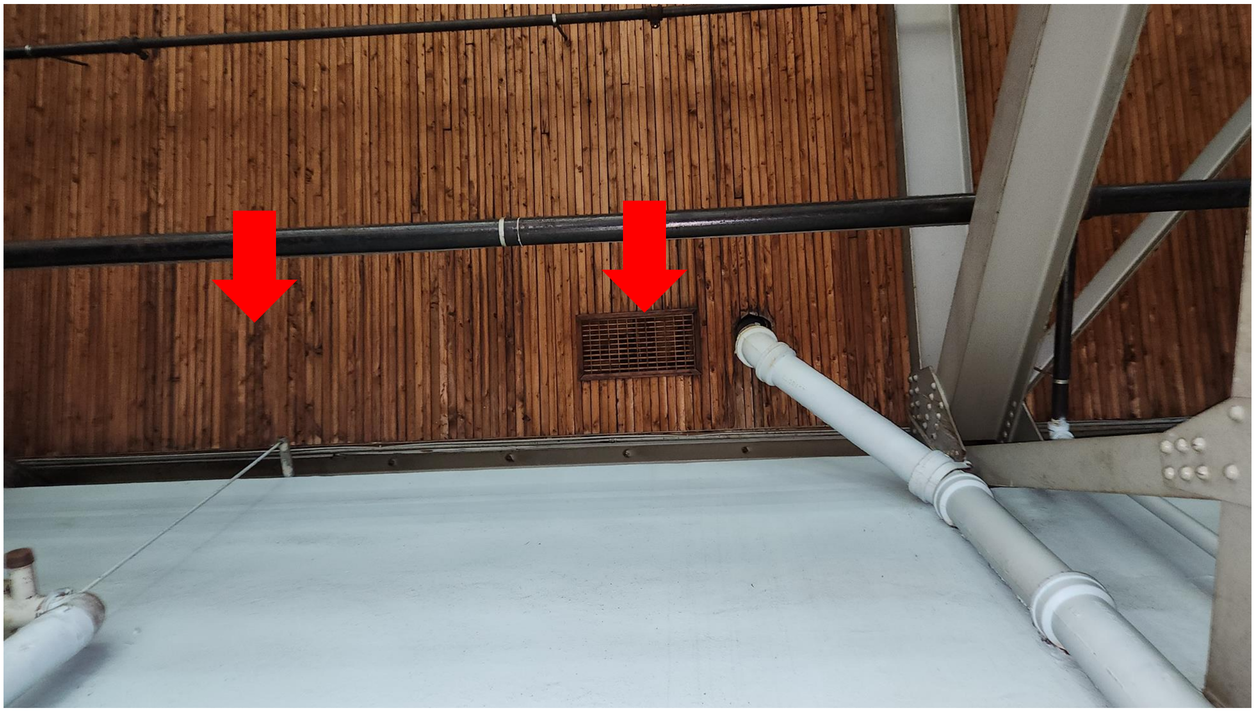

Photo 5: Water stains and typical opening through NLT deck (High Roof c.1951).

Photo 6: Typical High Roof structure exhibiting light surface corrosion (c.1951).

Photo 7: Deterioration of Low Roof structure at northwest corner (c.1951).

Photo 8: Typical Grandstand seating framing (c.1951).

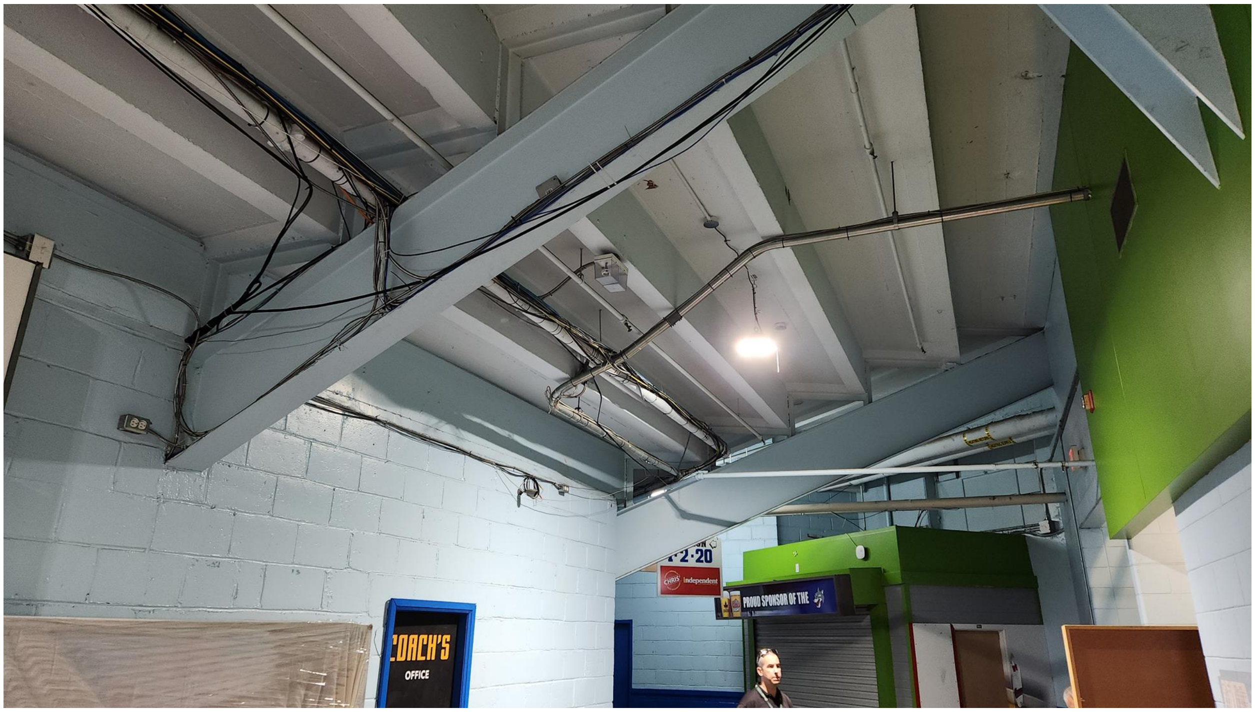

Photo 9: Typical Grandstand seating framing (c.1951).


Photo 10: Main Entrance stairs (c.1951).

Photo 11: East Entrance (Minto Street) stairs (c.1951).

Photo 12: East Entrance (Minto Street) stairs from below (c.1951).

Photo 13: Area of ongoing slab-on-grade repairs (c.1951).

Photo 14: Suspended concrete slab over mechanical room (c.1951).

Photo 15: Evidence of leakage through concrete foundation wall in the Mechanical Room (c.1951).

Photo 16: Evidence of leakage through concrete foundation wall in the Mechanical Room (c.1951).

Photo 17: Typical crack in concrete foundation wall (c.1951).

Photo 18: Typical crack in concrete foundation wall (c.1951).

Photo 19: Typical crack in concrete foundation wall (c.1951).

Photo 20: Cracked and spalling parging on the foundation walls at the south side of the building (c.1951).

Photo 21: Evidence of water infiltration through roof slab in the old Coal Storage room (c.1951).

Photo 22: Water on the floor in old Coal Storage room (c.1951).

Photo 23: Crack in concrete foundation and masonry walls (c.1951).

Photo 24: Cracks in concrete foundation and masonry walls (c.1951).

Photo 25: Crack in masonry wall (c.1951).

Photo 26: Crack in masonry wall (c.1951).

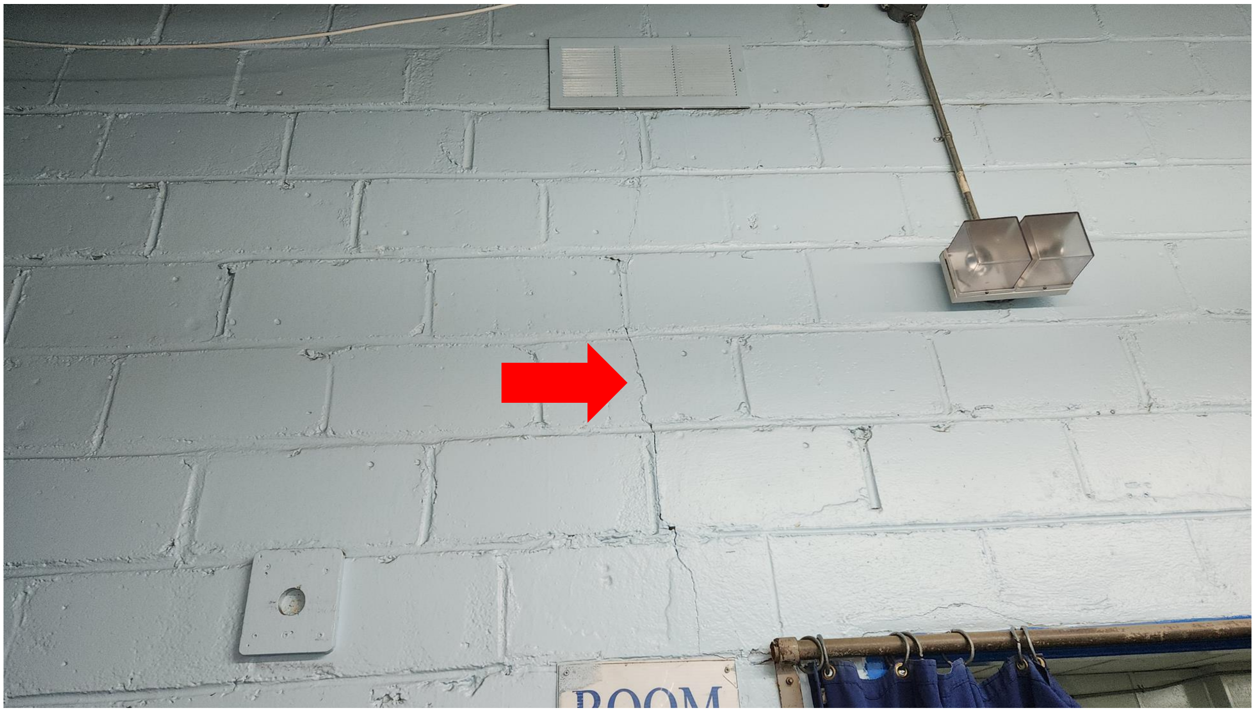

Photo 27: Crack in masonry wall (c.1951).

Photo 28: Peeling paint on masonry wall (c.1951).

Photo 29: Peeling paint on masonry wall (c.1951).

Photo 30: Missing lintel over Storage Room door (c.1951).

Photo 31: Damaged and cracked brick masonry (c.1951).

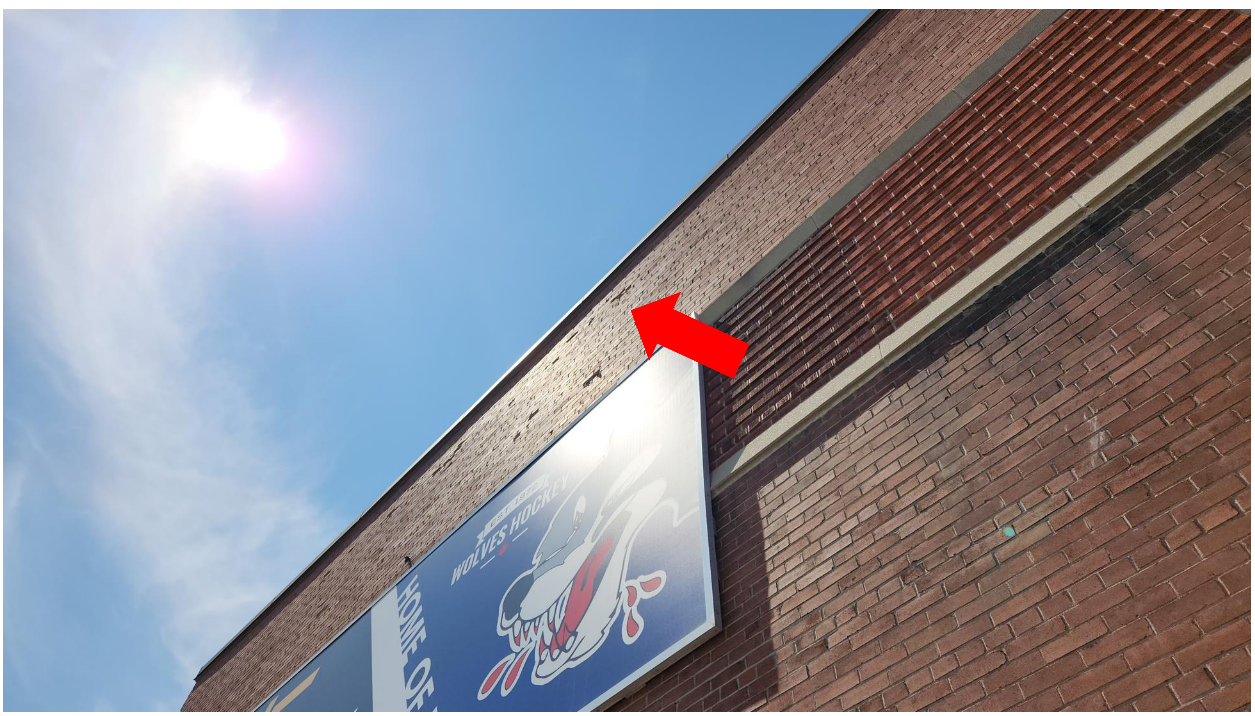

Photo 32: Damaged brick masonry (c.1951).

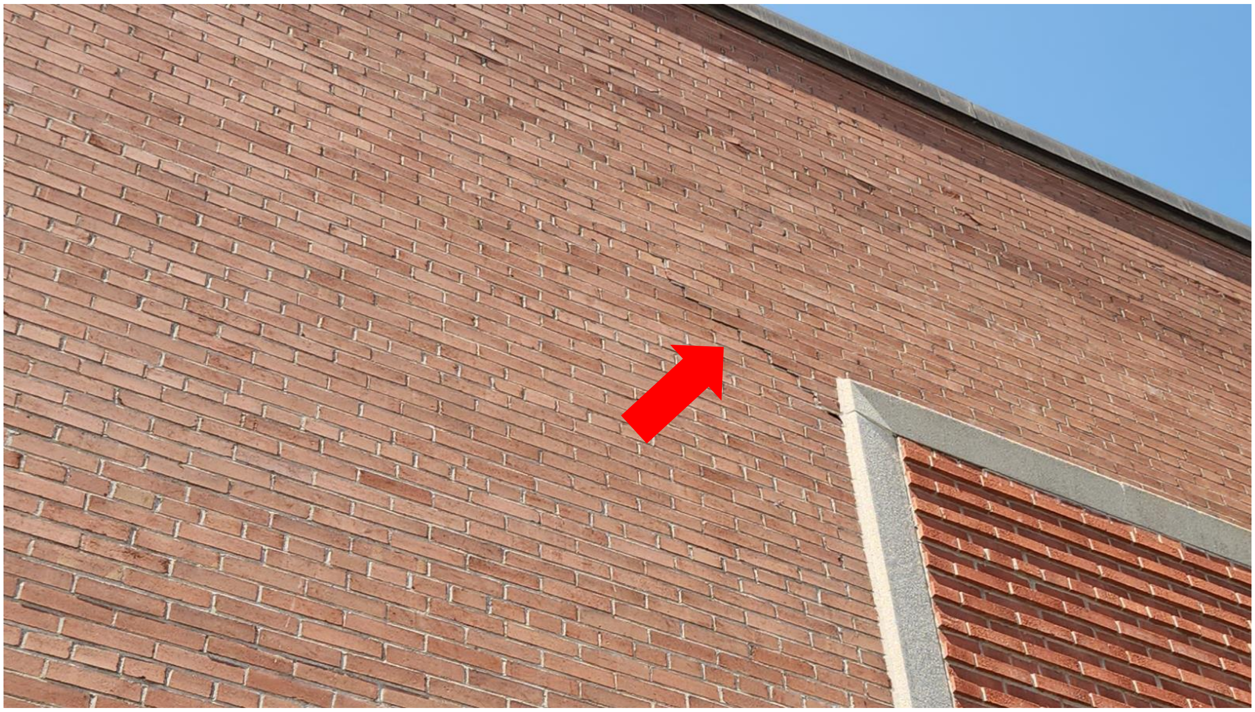

Photo 33: Cracked brick masonry (c.1951).

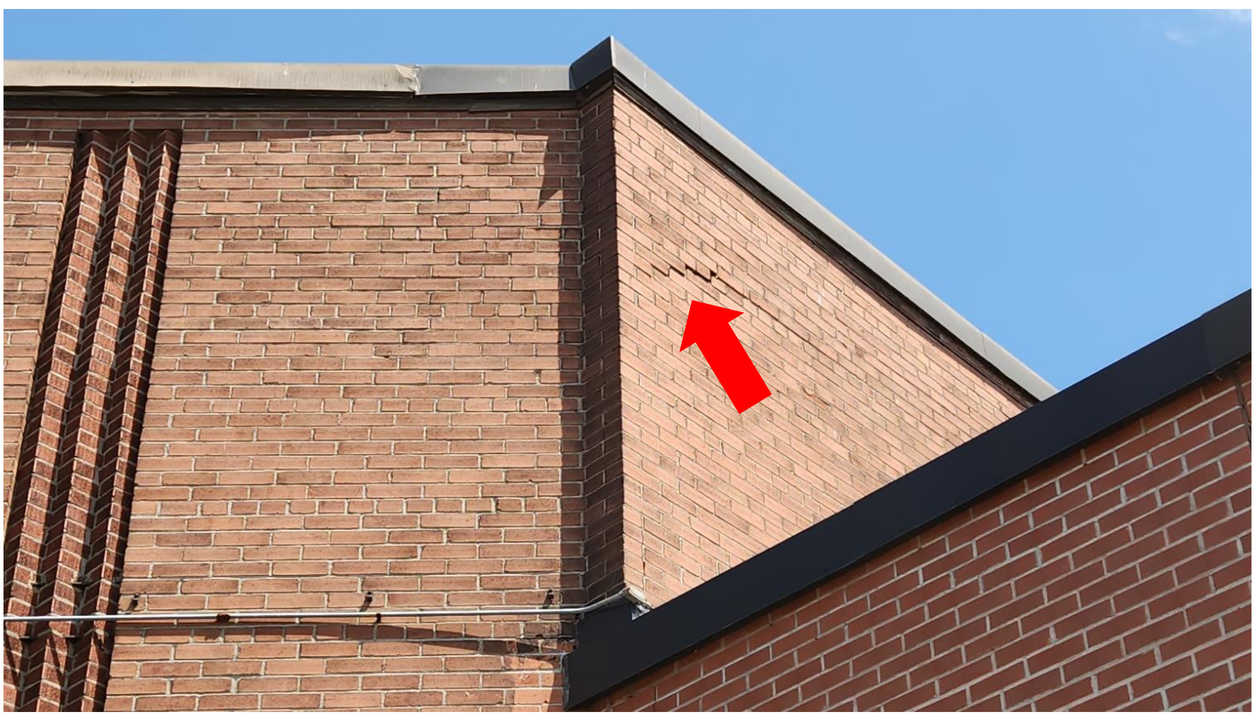

Photo 34: Cracked brick masonry (c.1951).

Photo 35: Cracked precast concrete at Main Entrance (c.1951).

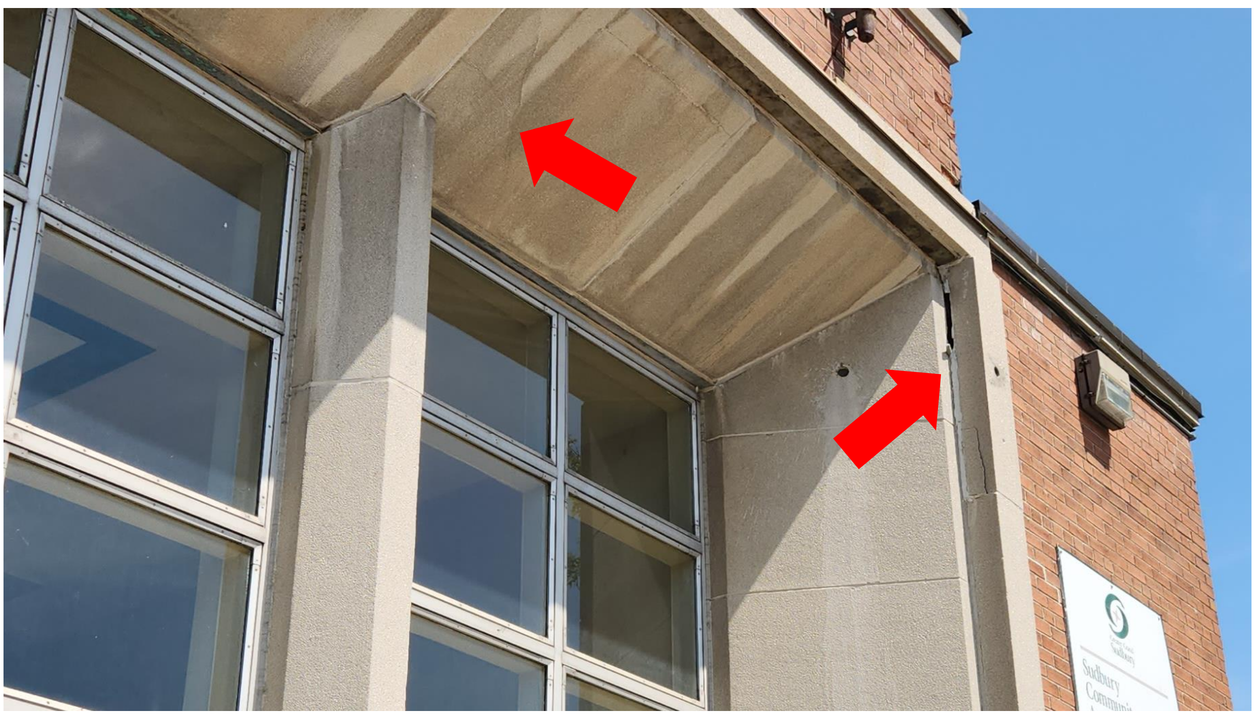

Photo 36: Cracked precast concrete at Main Entrance (c.1951).

Photo 37: Damaged precast concrete sill (c.1951).

Photo 38: Condition of steel canopy.

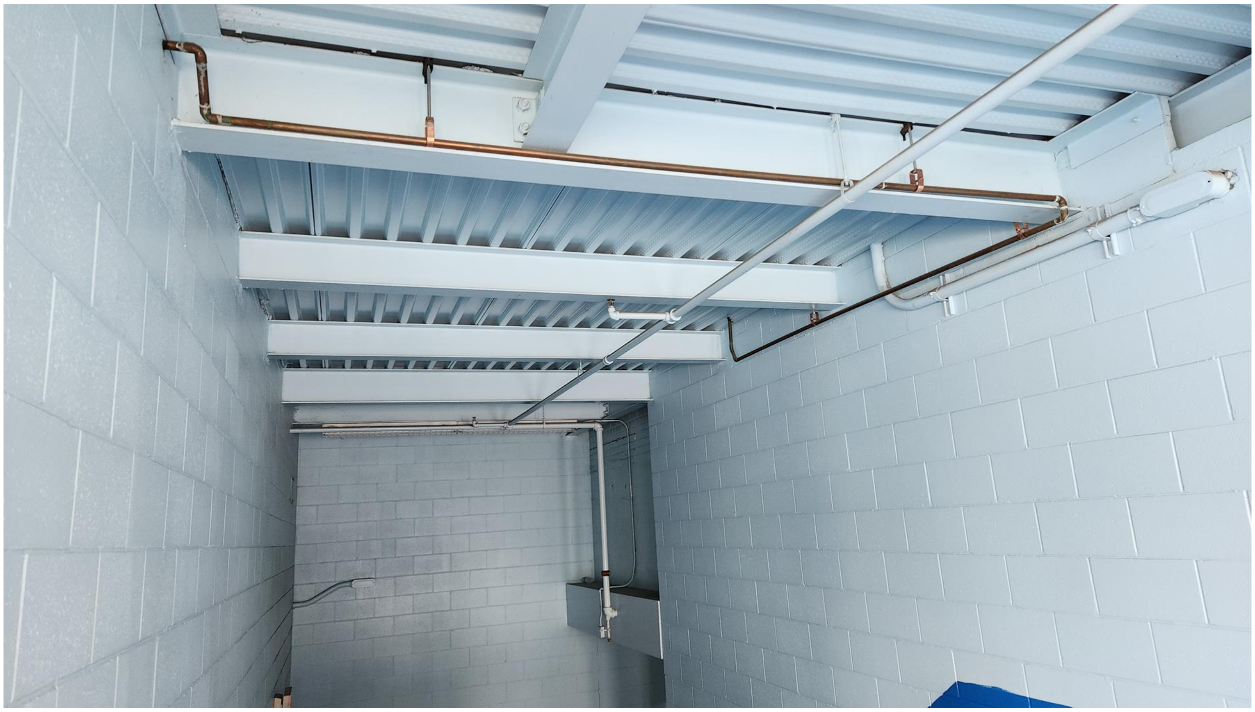

Photo 39: Typical floor framing (c.2000).

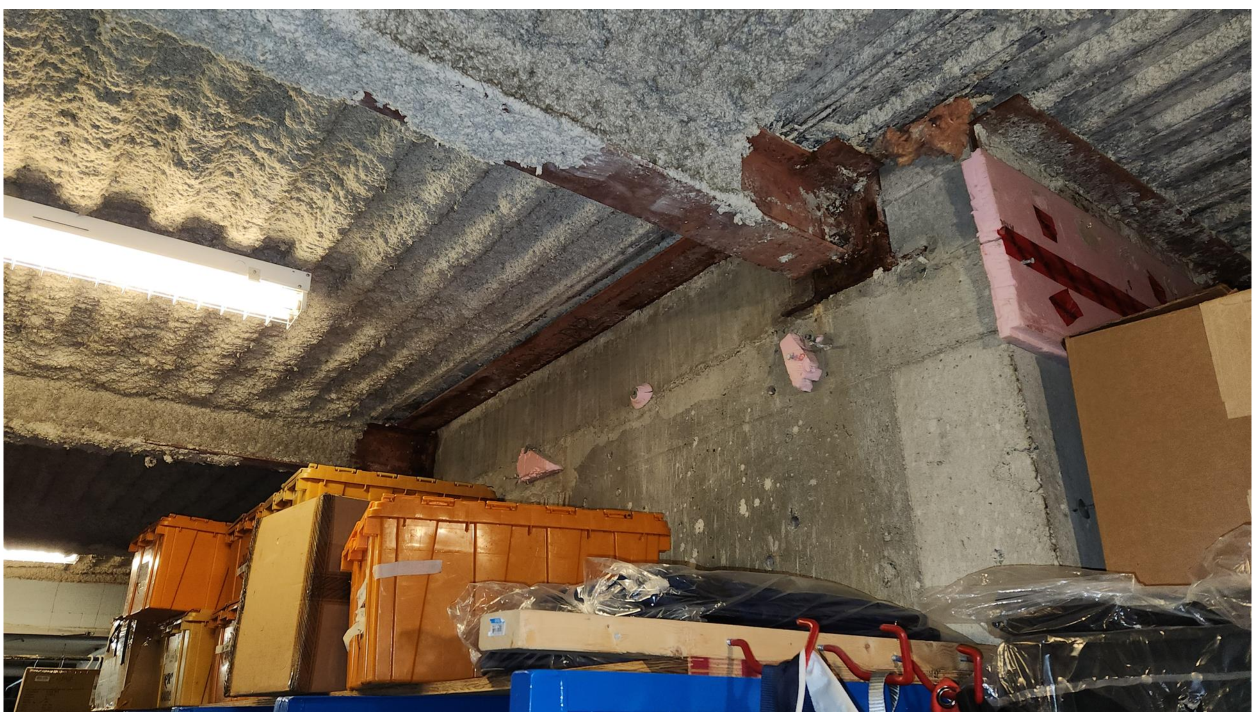

Photo 40: Ground Floor structure at the southwest stair addition (c.2000).

Photo 41: Corrosion of beam-to-column header connection at the southeast stair addition (c.2000).

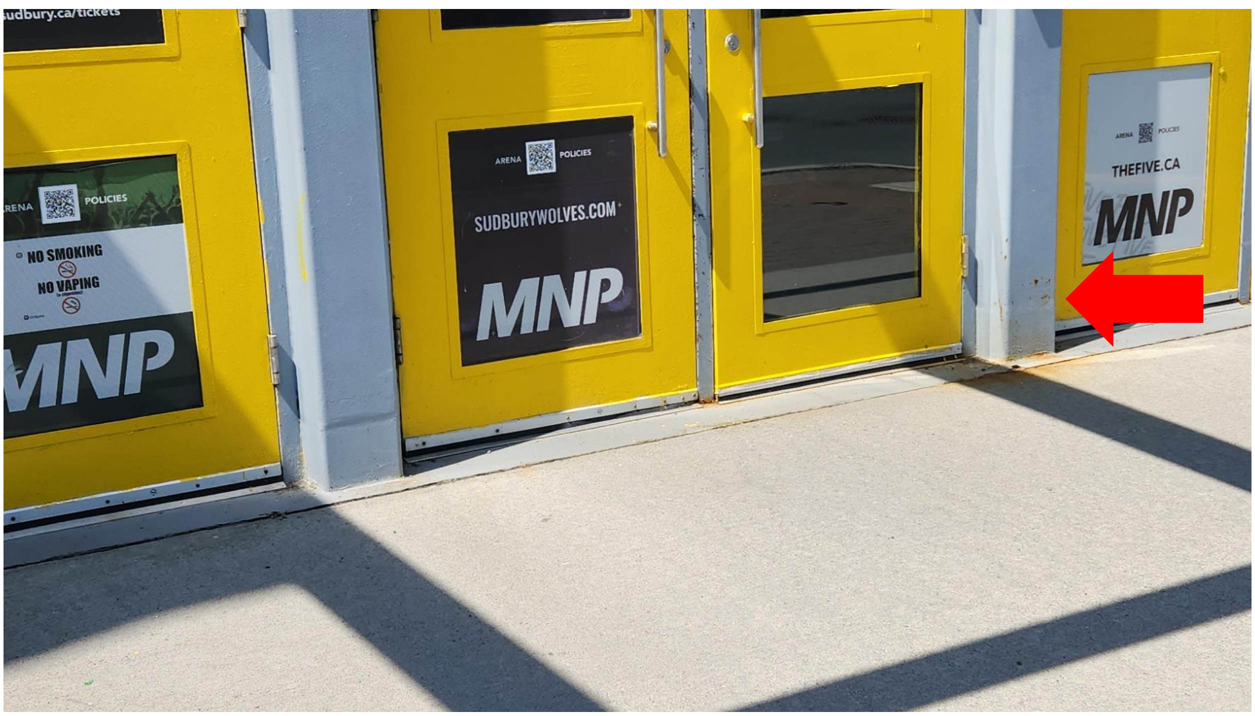

Photo 42: Evidence of continued corrosion to exposed steel columns at the northeast stair addition (c.2000).

Photo 43: Masonry wall cracks at the southwest stair addition (c.2000).

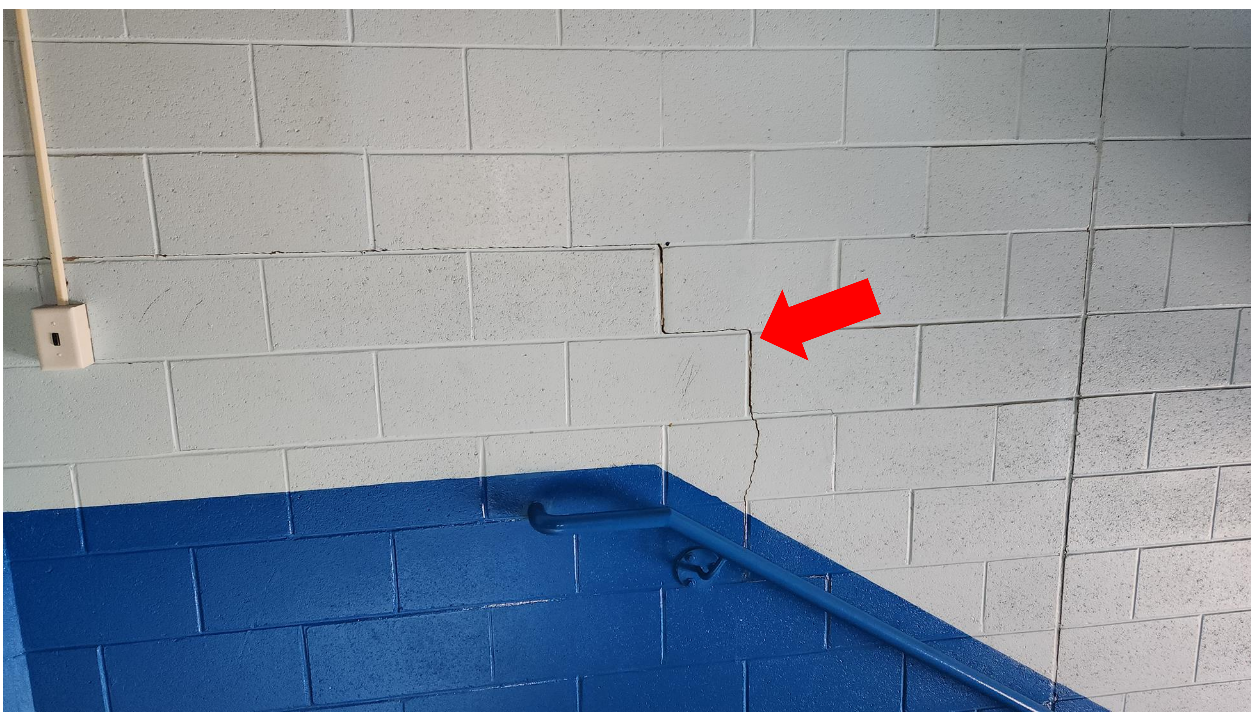

Photo 44: Masonry wall cracks at the southwest stair addition (c.2000).

Photo 45: Step-cracking in masonry wall at the southwest stair addition (c.2000).

Photo 46: Gaps between the steel deck flutes and top of wall at the northwest stair addition (c.2000).

Photo 47: Typical low roof framing (c.2006).


Photo 48: Typical floor framing (c.2006).

Photo 49: Steel beams below concrete slab believed to be installed as part of the c.2006 addition.

APPENDIX C – PREVIOUS REPORTS

March 4, 2016

CITY OF GREATER SUDBURY
200 Brady Street
Sudbury, Ontario
P3A 5P3
Attn: Nick Zinger

Dear Nick,

Re: **SUDBURY ARENA – 240 ELGIN STREET, SUDBURY, ON**
ZAMBONI SLAB SURFACE REPAIRS

As per your request and our proposal P15234, dated December 21, 2015, we have completed a visual review of the concrete Zamboni slab surface at Sudbury Arena to review the extent of concrete deterioration and provide recommendations for remedial measures.

1. OBSERVATIONS AND DISCUSSION

Although no original structural drawings of the building were available, the Structural Drawings associated with the 2013 slab repairs were made available to us. Based on these drawings, the existing slab surface consists of a 50mm bonded, reinforced concrete topping over the original concrete slab-on-grade (thickness and reinforcing unknown); however, it appears as though a thin, approximately 6 to 10mm thick, polymer modified mortar topping has been applied over, or in lieu of, the 2013 slab repair topping.

There is evidence of concrete wear/rutting and water ponding within the Zamboni wheel path depressions due to studded tire use. The mortar topping appears to be too thin and not adequately bonded to the substrate concrete to provide a durable driving surface for studded tires. Additionally, the substrate concrete does not have sufficient abrasion resistance to prevent ongoing degradation resulting in the observed wheel path depressions. In our opinion, the concrete deterioration is not currently structurally significant; however, we understand that the Owner would like to improve slab drainage, driveability and aesthetics.

In general, concrete abrasion and wear is mitigated by increasing concrete strength and nominal aggregate size, while decreasing water-to-cement ratio, permeability, and air content. It is also important to prevent overworking of the concrete during surface finishing to prevent fines and air voids from collecting at the concrete surface. Alternatively, ancillary protection can be applied at the surface to protect the underlying concrete.

2. REPAIR STRATEGIES

We present the following repair strategies with opinions of probable cost, which include a range of solutions to address the identified defects and promote adequate performance over the identified timeframe. All repair strategies presented require that the work be carried out by a qualified contractor under the review of a building professional. This process ensures that the building Owner will receive a high-quality repair, using high-quality, durable materials suited to the site-specific applications required.

Opinions of probable costs should only be considered preliminary, high-level budgets. Accurate budgeting can only be determined by a Cost Consultant and/or qualified Contractor based on a set of Contract Documents that clearly identify the scope of work for any further investigation and/or remedial repair details.

OPTION 1 – SACRIFICIAL WEARING SURFACE	$1,000

This strategy does not address existing deterioration but prevents further deterioration by applying a sacrificial wearing surface over the concrete slab surface. Heavy duty PVC or rubber mats could be laid under the Zamboni wheel paths to prevent ongoing concrete wear/rutting from the studded tires and conceal existing deterioration. The mats will likely move and be damaged by continual exposure to studded tires, particularly at the turning radius and will require ongoing adjustment, monitoring and maintenance/replacement.

Since this approach is a relatively low-cost option, requiring readily available material, it may be possible to attempt this repair strategy on a trial basis before proceeding with Option 2, 3 or 4.

Estimated Restoration Cycle: Annually

Advantages	**Disadvantages**
- Minimal disruption to building operations. - Lowest initial cost. - Some improvement to driveability.	- Requires ongoing mat adjustment, monitoring and maintenance/replacement. - Mats may be slippery when wet. - Does not address ponding water.

OPTION 2 – ABRASION RESISTANT COATING	$20,000

This strategy addresses existing deterioration by shotblasting to remove all loose and deteriorated concrete and applying a high-build, abrasion-resistant urethane or epoxy resin coating under the Zamboni wheel paths to prevent ongoing concrete wear/rutting from the studded tires. The wheel path depressions would be built-up with urethane/epoxy to help improve drainage.

Estimated Restoration Cycle: 5-7 Years

Advantages	**Disadvantages**
- Lower initial cost compared to Options 3 and 4. - Improves drainage. - Improves driveability.	- Requires ongoing monitoring and maintenance. - Not as durable as Options 3 and 4. - Disruption to building operations. - May be difficult to source local expertise in shotblasting and specialized coating application.

OPTION 3 – PARTIAL SLAB REPLACEMENT WITH HIGH STRENGTH CONCRETE $50,000

This strategy addresses existing deterioration by removing existing concrete (topping and original slab) and pouring a new concrete slab with abrasion-resistant surface hardener under the Zamboni wheel paths to prevent ongoing concrete wear/rutting from the studded tires. Concrete scanning would be required prior to concrete removal to identify embedded reinforcing and services.

Estimated Restoration Cycle: 10-15 Years

Advantages	**Disadvantages**
- Improves drainage.	- High capital cost.
- Requires minimal ongoing maintenance.	- Disruption to building operations.
- Improves driveability.	- Wheel path depressions will recur over time.

OPTION 4 – PARTIAL SLAB REPLACEMENT WITH STEEL GRATES/PLATES $70,000

This strategy addresses existing deterioration by removing existing concrete (topping and original slab) and installing steel grates or plates under the Zamboni wheel paths to prevent ongoing concrete wear/rutting from the studded tires. Drainage trenches could also be installed beneath the steel grates/plates for snow melt collection. Concrete scanning would be required prior to concrete removal to identify embedded reinforcing and services.

Estimated Restoration Cycle: 15-20 Years

Advantages	**Disadvantages**
- Best performance, longest service life.	- Highest capital cost.
- Significantly improves drainage.	- Disruption to building operations.
- Requires minimal ongoing maintenance.	
- Improves driveability.	

We trust that the enclosed information is adequate for your current needs. Please do not hesitate to contact us with any further questions or comments.

Sincerely,

Sam Colizza, P.Eng.
A2S Associates Limited

LICENSED PROFESSIONAL ENGINEER
2016.03.04
S. M. J. COLIZZA
100123333
PROVINCE OF ONTARIO

Attachments: Limitations

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LIMITATIONS

“Consultant” in the following document refers to A2S Associates Limited.

- The scope of our work and related responsibilities related to our work are defined in our proposal and Conditions of Assignment.
- Any user accepts that decisions made or actions taken based upon interpretation of our work are the responsibility of only the parties directly involved in the decisions or actions.
- No party other than the Client shall rely on the Consultant’s work without the express written consent of the Consultant, and then only to the extent of the specific terms in that consent. Any use which a third party makes of this work, or any reliance on or decisions made based on it, are the responsibility of such third parties. Any third party user of this report specifically denies any right to any claims, whether in contract, tort and/or any other cause of action in law, against the Consultant (including sub-consultants, their officers, agents and employees).The work reflects the Consultant’s best judgement in light of the information reviewed by them at the time of preparation. It is not a certification of compliance with past or present regulations. Unless otherwise agreed in writing by the Consultant, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity; it is written to be read in its entirety.
- Only the specific information identified has been reviewed. No physical or destructive testing and no design calculations have been performed unless specifically recorded. Conditions existing but not recorded were not apparent given the level of study undertaken. Conditions may differ from those observed, which were relied upon to develop our recommendations. Only conditions actually seen during examination of representative samples can be said to have been appraised and comments on the balance of the conditions are assumptions based upon extrapolation. Therefore, this work does not eliminate uncertainty regarding the potential for existing or future costs, hazards or losses in connection with a property. We can perform further investigation on items of concern if so required.
- The Consultant is not responsible for, or obligated to identify, mistakes or insufficiencies in the information obtained from the various sources, or to verify the accuracy of the information.
- No statements by the Consultant are given as or shall be interpreted as opinions for legal, environmental or health findings. The Consultant is not investigating or providing advice about pollutants, contaminants or hazardous materials.
- The Client and other users of this report expressly deny any right to any claim against the Consultant, including claims arising from personal injury related to pollutants, contaminants or hazardous materials, including but not limited to asbestos, mould, mildew or other fungus.
- Applicable codes and design standards may have undergone revision since the subject property was designed and constructed. As an example, design loads (such as those for temperature, snow, wind, rain, seismic, etc.) and the specific methods of calculating the capacity of the systems to resist these loads may have changed significantly. Unless specifically included in our scope, no calculations or evaluations have been completed to verify compliance with current building codes and design standards.
- Time frames given for undertaking work represent our opinion of when to budget for the work. Failure of the item, or the optimum repair/replacement process, may vary from our estimate. This opinion is therefore given as a reasonable average approximation rather than a specific prediction.
- Qualified design professionals are required to perform additional evaluation (as necessary), design and general review during construction when carrying out the recommendations included in this report. Ongoing

monitoring is required to confirm that repair or renewal measures are successful and to identify for changing conditions that would require increased levels of intervention or different repair / renewal strategies.

- Qualified contractors are required to implement any recommendations included in this report.
- Failure to implement the recommendations included in this report and/or failure to maintain building components appropriately could lead to ongoing and accelerated deterioration that may lead to unsafe conditions developing.
- Budget figures are our opinion of a probable current dollar value of the work and are provided for approximate budget purposes only. Accurate figures can only be obtained by establishing a scope of work and receiving quotes from appropriate contractors.

May 13, 2016

CITY OF GREATER SUDBURY
200 Brady Street
Sudbury, Ontario
P3A 5P3
Attn: Nick Zinger

Dear Nick,

Re: **SUDBURY ARENA – 240 ELGIN STREET, SUDBURY**
REVIEW OF STAIR ADDITIONS

As requested, we have completed our review of the structural steel framing at the Sudbury Arena Stair Additions to identify the extent of corrosion, analyse the remaining capacity of corroded structural members and provide recommendations for remedial measures.

1. BACKGROUND

The Sudbury Arena Stair Additions were constructed circa 2000 and consist of structural steel framing on reinforced concrete foundation walls and raft slabs. The 3-storey additions (including basement) include composite deck landings and a steel roof deck designed for future floor. The additions are enclosed with steel-framed curtainwall and doors between HSS columns. The HSS columns are partially exposed on the exterior.

At our site meeting on February 17, 2016, we observed evidence of leakage and steel corrosion at the basement level of the northeast stair addition. A Contractor was retained by the City to remove interior drywall and spray-applied fireproofing to expose the concealed structural steel members and the composite deck soffit. We returned to site on April 4, 2016 to review the exposed structure. Based on the extent of corrosion observed, temporary shoring was installed at the underside of the northeast stair ground floor landing immediately and steel inspection of all four stair additions was recommended to confirm the extent of cross-sectional area loss to corroding steel members, bolts and welds.

2. OBSERVATIONS

A summary of the items identified during the steel inspection on April 28, 2016 is provided below. Additional information is provided in the attached steel inspection report by Laabs Industries, dated May 4, 2016.

2.1 Northwest Stair (Gray & Brady)

Minor corrosion at the two (2) beam ends and their associated HSS columns and connections was observed. According to the steel inspection report, there was no appreciable section loss to the structural steel members and bolts and welds appeared to be intact with no sign of cracking. No significant corrosion to the composite deck soffit was observed. Steel should be cleaned with a wire brush and re-coated. Exterior remediation is required to improve waterproofing to prevent ongoing corrosion and deterioration.

2.2 Southwest Stair (Gray & Elgin)

Minor corrosion at the four (4) beam ends and their associated HSS columns and connections was observed. According to the steel inspection report, there was no appreciable section loss to the steel beams and bolts and welds appeared to be intact with no sign of cracking; however, the columns are corroded with evidence of section loss (i.e. approx. 20% section loss at HSS column flange). No significant corrosion to the composite deck soffit was observed. Steel should be cleaned with a wire brush and re-coated. Exterior remediation is required to improve waterproofing to prevent ongoing corrosion and deterioration.

We have confirmed the remaining column capacity to be adequate since they has been designed for future floor loading of 100psf; however, repairs are recommended prior to the next snow fall to mitigate further deterioration before the columns are subjected to roof snow loading.

2.3 Southeast Stair (Minto & Elgin)

Minor corrosion at the four (4) beam ends and their associated HSS columns and connections was observed. According to the steel inspection report, there was no appreciable section loss to the structural steel members and bolts and welds appeared to be intact with no sign of cracking. No significant corrosion to the composite deck soffit was observed. Steel should be cleaned with a wire brush and re-coated. Exterior remediation is required to improve waterproofing to prevent ongoing corrosion and deterioration.

2.4 Northeast Stair (Minto & Brady)

According to the steel inspection report, the cross-sectional area loss at the two (2) beam ends warrants partial beam replacement. The welds generally appeared to be intact with no sign of cracking; however, the bolts and columns are also extensively corroded with significant section loss (i.e. approx. 50% section loss at HSS column flange). The bottom of the column also appeared to show stress markings and bulging relating to the accumulation and subsequent freezing and expansion of trapped moisture. The source of moisture is likely leakage from rain, snow melt or groundwater through a crack or pinhole in the HSS column. Although the columns are uninsulated, condensation is unlikely since we did not observe significant evidence of moisture or corrosion of the column above-grade.

We expect that the base of the column will have to be removed and replaced with matching sections spliced to the existing. The beam ends could also be removed and replaced by introducing new steel columns supported by the raft slab. The composite deck span will need to be reduced by introducing new steel members running perpendicular to the deck, allowing the concrete slab to span between supports without relying on composite action from the corroded deck. Shoring has been installed to temporarily support the compromised structure; however, repairs are recommended prior to the next snow fall to mitigate further deterioration before the columns are subjected to roof snow loading.

2.5 Miscellaneous Items

The welds connecting the 'afterhours' sign to the steel framing at the northeast stair addition are too low to adequately support the sign, which is beginning to lean forward. The top of the sign should be welded to the frame to prevent further rotation.

3. RECOMMENDATIONS

We present the following repair strategies with opinions of probable costs, which include a range of solutions to address the identified defects and promote adequate performance over the identified timeframe. All repair strategies presented require that the work be carried out by a qualified contractor under the review of a building professional. This process ensures that the building Owner will receive a high-quality repair, using high-quality, durable materials suited to the site-specific applications required.

Opinions of probable costs should only be considered preliminary, high-level budgets. Accurate budgeting can only be determined by a Cost Consultant and/or qualified Contractor based on a set of Contract Documents that clearly identify the scope of work for any further investigation and/or remedial repair details.

OPTION 1 – WATERPROOFING AND STRUCTURAL STEEL REPAIRS	$250,000
Option 1 includes waterproofing of the below-grade structure, repair of compromised steel members and protection of remaining steel components. It is the minimum scope of work necessary to reinstate the structural integrity of the building and prevent ongoing deterioration. Repairs are recommended prior to the next snow fall to mitigate further deterioration before the columns are subjected to roof snow loading; however, we realize that due to budget and timing limitations, completing this strategy immediately at all 4 stair additions may not be feasible, therefore, we have also presented a 'phased approach' below. ***<u>All four (4) stair additions</u>:*** This option addresses moisture ingress by excavating the exterior foundation perimeter, including all landscaping and sidewalks and installing an elastomeric waterproofing membrane to the below grade structure. Exterior doors will require temporary removal and re-installation to facilitate proper termination of the membrane, approximately 12" above grade. Door frames will require replacement where deteriorated. Perimeter drainage would be improved by installing new sub-surface weeping tile and granular backfill with new sidewalks sloping away from the building. This strategy assumes the building has an existing sub-surface storm drainage system to tie-in to. Additional funds would be necessary if extensive civil work is required. As part of the repairs we would install sacrificial anodes at the basement level of the southwest and northeast stair and drill small holes through the HSS column flanges of each stair. This will allow ongoing monitoring of water ingress and rates of corrosion and prevent accumulation of trapped moisture. We recommend the anodes be checked on a semi-annual basis, before and after winter. All remaining above-grade structural steel on the exterior wall of the building would be wire brushed and re-coated with zinc-rich primer and epoxy paint. Following repairs, all exterior landscaping and sidewalks and interior finishes and fireproofing would be reinstated. Going forward, the use of de-icing salt at stair entrances should be limited by using non-corrosive salt alternatives and/or sand when possible. ***<u>Northeast stair addition only</u>:*** Replacement of deteriorated structural steel members is recommended as identified above. The base of HSS columns will be removed and replaced with matching sections spliced to existing; beam ends will be removed and replaced with new sections, and a new column will be installed on the existing raft slab. The existing beams will be re-connected to the new column. The existing composite deck span will need to be reduced by introducing new steel members running perpendicular to the deck, allowing the concrete slab to span between supports without relying on composite action from the corroded deck. *Continued on next page…*	

Additional temporary shoring will be required to facilitate these additional repairs. At this time, we expect this scope is only required at the northeast stair addition; however, excavation of the other stair additions may uncover additional locations of significant section loss.

Phased approach:
Structural steel corrosion at the northeast stair addition is the most advanced and is therefore the highest priority. We highly recommend completing repairs in this area prior to the next snow fall. If repairs cannot be completed before winter, the stair would have to be cordoned off from public access and fully shored from the underside of the roof structure down to the basement level.

Based on the rate of corrosion over the past 15 years at the 3 other stair additions, we expect that deferral is possible provided the following measures are taken:

a) Test pits are excavated on the exterior of the HSS columns to confirm there is no significant section loss on the exterior side of the columns, below grade;
b) Sacrificial anodes and drilled holes are installed on the interior side of HSS columns to allow for monitoring of corrosion rates and moisture ingress;
c) The use of de-icing salt at stair entrances is limited by using non-corrosive salt alternatives and/or sand; and
d) These areas are monitored on a semi-annual basis, before and after winter.

Approximately $100,000 should be made available to complete Phase 1 repairs only, at this time.

Estimated Restoration Cycle: Every 5-7 years

Advantages	Disadvantages
- Minimizes disruption to building operations. - Lowest initial cost. - Can be completed in a phased approach to work with budget and timeline constraints.	- Requires ongoing monitoring and maintenance. - Difficult to provide a robust waterproofing detail at the landing-to-wall interface which may result in ongoing moisture ingress and membrane patching repairs.

OPTION 2 – RE-CLADDING, WATERPROOFING AND STRUCTURAL STEEL REPAIRS $1,000,000

All four (4) stair additions:
This option is similar in scope to Option 1; however, it includes re-cladding of the entire façade above grade and construction of a new concrete curb below-grade to conceal the structural steel members from exposure to the elements and provide a durable and robust water-shedding surface in front of the main structure.

This option requires the assistance of an Architect in developing cladding options for consideration.

The waterproofing scope will be similar to Option 1; however, the new below-grade waterproofing will be applied to the new concrete curb. Sacrificial anodes and scheduled monitoring would not be required. We assume the exterior doors could be retained and re-used; however, damaged frames will require replacement.

The scope for the required structural steel repairs at the northeast stair is the same as Option 1.

Estimated Restoration Cycle: 25+ years

Advantages	Disadvantages
- Best performance, longest service life. - Requires minimal ongoing maintenance.	- High capital cost - Significant disruption to building operations.

We trust that the enclosed information is adequate for your current needs. Please do not hesitate to contact us with any further questions or comments.

Sincerely,

LICENSED PROFESSIONAL ENGINEER
2016.05.13
S. M. J. COLIZZA
100123333
PROVINCE OF ONTARIO

Sam Colizza, P.Eng.
A2S Associates Limited

Attachments: Steel Inspection Report by Laabs Industries, dated May 4, 2016 (2 pages)
Limitations (2 pages)

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RECEIVED: 05/04/2016

LAABS INDUSTRIES

Sudbury, Ontario
Ph#: {705} 586-2553
Fax: {705} 586-2553
Cell: {705} 918- 5227
Email: laabs@fibreop.ca

Contractor Survey Sheet – Site Report

Client: A2S Associates Ltd.

Site Report No.: 1

Address: 289 Cedar St., Suite 201
Sudbury, Ont.

Work Order No.: 16-0870

Project: Sudbury Arena Stair Supports

Project Location: Sudbury, Ont.

Inspector: Ken Laabs

Date of Inspection: April 28, 2016

1. **Governing Codes & Specifications:** CSA W47.1/W59 & Client's
2. **Survey of Welding Contractor(s):**
   a) Contractor(s) Certification to CSA W47.1/59: N/A
   b) Staffing includes qualified:
      i) Engineers N/A
      ii) Welding Supervisors N/A
      iii) Welders N/A
      iv) Welding Procedures N/A
   c) Remarks:
3. **Materials Verification:** N/A
4. **Consumables Verification:** N/A
5. **Consumables Storage:** N/A
6. **Additional Remarks:**

N/A = not applicable N/R = Not Requested N/V = Not verified

LAABS INDUSTRIES

Sudbury, Ontario
Ph#: {705} 586-2553
Fax: {705} 586-2553
Cell: {705} 918- 5227
Email: laabs@fibreop.ca

Contractor Survey Sheet – Site Report

1. Scope of Work

1.1 Inspection of structural steel welds and members at four stairway locations within the building.

2. Inspection

2.1 Visual inspection was carried out on the welds and structural members.

2.2 Thickness testing was carried out on five spots to determine corrosion effect on the inside of the columns. (One in each corner of the building on the base and one in the North-East corner at the doors as per the engineers request.)

3. Result

3.1 The welds inspected appear to be intact with no sign of cracking.

3.2 In all but one stairwell the structural steel has minor rust that has not yet resulted any appreciable loss of section on the outside.

3.3 In the North-East stairwell the welds are intact but the structural members have corroded extensively as have the bolts in the connections.

3.4 Thickness readings of the main structural column coming from the upper floor are as as follows..

North-East corner (bottom)	.128"
South-East corner (bottom)	.240"
South-West corner (bottom)	.185"
North-West corner (bottom)	.220"
North-East corner (top)	.237"

3.5 Thickness reading N-E (top) was taken approximately 1" above the floor corrosion. All other readings were taken from approximately 3" above bottom plate.

CANADIAN WELDING BUREAU
#9365
KEN LAABS
CSA W178.2
CERTIFIED WELDING INSPECTOR

Authorized Signature

Should any questions arise concerning this report, please do not hesitate to contact me.

LIMITATIONS

"Consultant" in the following document refers to A2S Associates Limited.

- The scope of our work and related responsibilities related to our work are defined in our proposal and Conditions of Assignment.
- Any user accepts that decisions made or actions taken based upon interpretation of our work are the responsibility of only the parties directly involved in the decisions or actions.
- No party other than the Client shall rely on the Consultant's work without the express written consent of the Consultant, and then only to the extent of the specific terms in that consent. Any use which a third party makes of this work, or any reliance on or decisions made based on it, are the responsibility of such third parties. Any third party user of this report specifically denies any right to any claims, whether in contract, tort and/or any other cause of action in law, against the Consultant (including sub-consultants, their officers, agents and employees).The work reflects the Consultant's best judgement in light of the information reviewed by them at the time of preparation. It is not a certification of compliance with past or present regulations. Unless otherwise agreed in writing by the Consultant, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity; it is written to be read in its entirety.
- Only the specific information identified has been reviewed. No physical or destructive testing and no design calculations have been performed unless specifically recorded. Conditions existing but not recorded were not apparent given the level of study undertaken. Conditions may differ from those observed, which were relied upon to develop our recommendations. Only conditions actually seen during examination of representative samples can be said to have been appraised and comments on the balance of the conditions are assumptions based upon extrapolation. Therefore, this work does not eliminate uncertainty regarding the potential for existing or future costs, hazards or losses in connection with a property. We can perform further investigation on items of concern if so required.
- The Consultant is not responsible for, or obligated to identify, mistakes or insufficiencies in the information obtained from the various sources, or to verify the accuracy of the information.
- No statements by the Consultant are given as or shall be interpreted as opinions for legal, environmental or health findings. The Consultant is not investigating or providing advice about pollutants, contaminants or hazardous materials.
- The Client and other users of this report expressly deny any right to any claim against the Consultant, including claims arising from personal injury related to pollutants, contaminants or hazardous materials, including but not limited to asbestos, mould, mildew or other fungus.
- Applicable codes and design standards may have undergone revision since the subject property was designed and constructed. As an example, design loads (such as those for temperature, snow, wind, rain, seismic, etc.) and the specific methods of calculating the capacity of the systems to resist these loads may have changed significantly. Unless specifically included in our scope, no calculations or evaluations have been completed to verify compliance with current building codes and design standards.
- Time frames given for undertaking work represent our opinion of when to budget for the work. Failure of the item, or the optimum repair/replacement process, may vary from our estimate. This opinion is therefore given as a reasonable average approximation rather than a specific prediction.
- Qualified design professionals are required to perform additional evaluation (as necessary), design and general review during construction when carrying out the recommendations included in this report. Ongoing

monitoring is required to confirm that repair or renewal measures are successful and to identify for changing conditions that would require increased levels of intervention or different repair / renewal strategies.

- Qualified contractors are required to implement any recommendations included in this report.
- Failure to implement the recommendations included in this report and/or failure to maintain building components appropriately could lead to ongoing and accelerated deterioration that may lead to unsafe conditions developing.
- Budget figures are our opinion of a probable current dollar value of the work and are provided for approximate budget purposes only. Accurate figures can only be obtained by establishing a scope of work and receiving quotes from appropriate contractors.

November 16, 2018

CITY OF GREATER SUDBURY
200 Brady Street
Sudbury, Ontario
P3A 5P3
Attn: Chad Kobylka

Dear Chad,

Re: **SUDBURY ARENA – 240 ELGIN STREET, SUDBURY, ON**
STRUCTURAL REVIEW FOR PARTIAL RE-ROOFING PROJECT

As per your request and our proposal P18197, dated October 31, 2018, we have carried out a general structural review of the existing roof framing in the area scheduled for re-roofing in 2018. The purpose of our review is to:

- Confirm the general condition of the existing roof structure;
- Identify deficiencies in the original design;
- Identify changes that may have resulted in load not considered in the original design;
- Confirm that the weight of the proposed roofing assembly does not constitute a reduction in performance level;
- Assess the potential impact of changes to the thermal performance of the roofing assembly; and
- Identify older roof structures that were designed prior to specific benchmark editions of the Ontario Building Code (OBC) and/or National Building Code of Canada (NBC).

As part of our review we completed the following:

- Reviewed the proposed re-roofing quotation by Damisona Roofing Ltd., dated September 5, 2018.
- Visited the site on October 30 and November 1, 2018 to complete a visual review of the accessible structure and to obtain the necessary measurements to facilitate a structural analysis; and
- Analyzed a rational sampling of structural members to confirm their adequacy for the imposed roof loading.

The current re-roofing project encompasses the low roof area above the "Wolves Den" at the northwest corner of the building (refer to Appendix A).

The existing roofing assembly is scheduled to be removed down to the existing structural deck and replaced with the following proposed roofing assembly (top-to-bottom);

Existing Roofing Assembly	Proposed Roofing Assembly
- Built-up roofing - Semi-rigid insulation (unknown type and thickness) - Vapour barrier	- Pea gravel in modified asphalt flood coat (5psf max) - 2-ply modified bitumen base sheet - 1/2” fiberboard sheathing - 3” polyiso insulation - Vapour barrier
Weight: approx. 5-7psf (assumed)	**Weight: approx. 8.5psf**
R-Value: 7 (assumed)	**R-Value: 20**

The OBC and NBC apply to the design of new buildings and are not retroactive. Our analysis considers guidelines for the review of existing structures as outlined in ‘Commentary L: Application of NBC Part 4 of Division B for the Structural Evaluation and Upgrading of Existing Buildings’ of the Structural Commentaries (User’s Guide – NBC 2015: Part 4 of Division B). The approach outlined in Commentary L has been used to evaluate the impact of the proposed re-roofing operations on the existing roof structure. The addition of new loads or changes to the thermal properties of a roof constitutes an *upgrade* to the structure, which is to be evaluated using current versions of the OBC or NBC considering load factors prescribed in Commentary L. Alternatively, existing structures that are not being upgraded can be evaluated using the code in force when built except where benchmark editions of the NBC have been identified that introduced significant changes to either the magnitude or extent of loads on roofs. All existing structures built prior to a benchmark edition have been evaluated using current versions of the OBC or NBC considering load factors prescribed in Commentary L. The following benchmark editions generally apply to the evaluation of roof structures:

- **NBC 1965 – Snow drifts due to high roofs and roof obstructions;**
- NBC 1970 – Retained rain loads on roofs due to blocked drains;
- NBC 1990 – Ground snow load changes resulting in significant increases in some municipalities; and
- NBC 1995 – Snow accumulation on large roofs.

No existing drawings were made available for the low roof framing. Based on our site observations and measurements, low roof framing generally consists of concrete on steel pan deck spanning between 14” deep open web steel joists (OWSJ) at 2’-0” centres supported by structural steel beams and columns. The concrete deck thickness was not confirmed during our review. We have assumed a conventional concrete thickness of 2½” for this vintage of construction.

We understand that there is active water leakage in several locations. Our visual review of the generally exposed roof structure did not identify any evidence of structurally significant deterioration resulting from excessive exposure to moisture. The topside of the roof deck should be reviewed during re-roofing operations to confirm the condition of the existing concrete.

As the arena was designed and constructed c.1951 and prior to the NBC benchmark year for snow drifts, we are of the opinion that the snow loads in force during original construction are un-conservative and not appropriate for our review. We have analyzed the low roof area for snow accumulation loading using the current version of the Building Code with the load factors recommended by the NBCC Structural Commentaries. The following reliability index was considered in our analyses:

Factor	Category	Index
System Behaviour	Failure likely to impact people	2
Risk Category	High	2

Past Performance	Record of satisfactory past performance	0
	Reliability Index	**4**

A specified snow loading of up to 205psf was considered in our analysis based on the size and height difference to the upper roof. We have considered a design dead load of 60psf based on the self-weight of the roofing assembly, structure and electrical/mechanical equipment.

Our analysis indicates that the existing OWSJ and steel beams associated with the low roof area are significantly under-designed to accommodate snow accumulation loading due to snow drift. We recommend reinforcing the existing structure as required. Roof reinforcement would most likely involve the installation of new channels between the existing OWSJ. Beams supporting the existing OWSJ and new channels would also require reinforcing with WT-sections welded to their bottom flange. Reinforcing details for the compensating structure must be prepared by a Professional Engineer licenced in Ontario, with repairs completed by a qualified Contractor.

The Commentaries recommend meeting with the Authority Having Jurisdiction (Chief Building Official) to discuss the findings of the evaluation, and to establish a timetable for any reinforcing work to be done. In the interim, we recommend implementing an immediate snow watch program for the low roof to ensure that snow heights do not exceed 16".

We trust that the above-mentioned information meets your current needs. Should any additional defects or areas of concern be uncovered during the re-roofing, the Contractor must contact a Building Professional for further investigation and review. We recommend clearly noting this requirement on the Contract Documents.

Sincerely,

LICENSED PROFESSIONAL ENGINEER
2018.11.16
S. M. J. COLIZZA
100123333
PROVINCE OF ONTARIO

Sam Colizza, P.Eng.
A2S Consulting Engineers

Attachments: Appendix A – Photographs (1 page)
Appendix B – Limitations (2 pages)

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APPENDIX A – PHOTOGRAPHS

Figure 1: Low roof area scheduled for re-roofing in 2018 (photo courtesy of Google Maps).

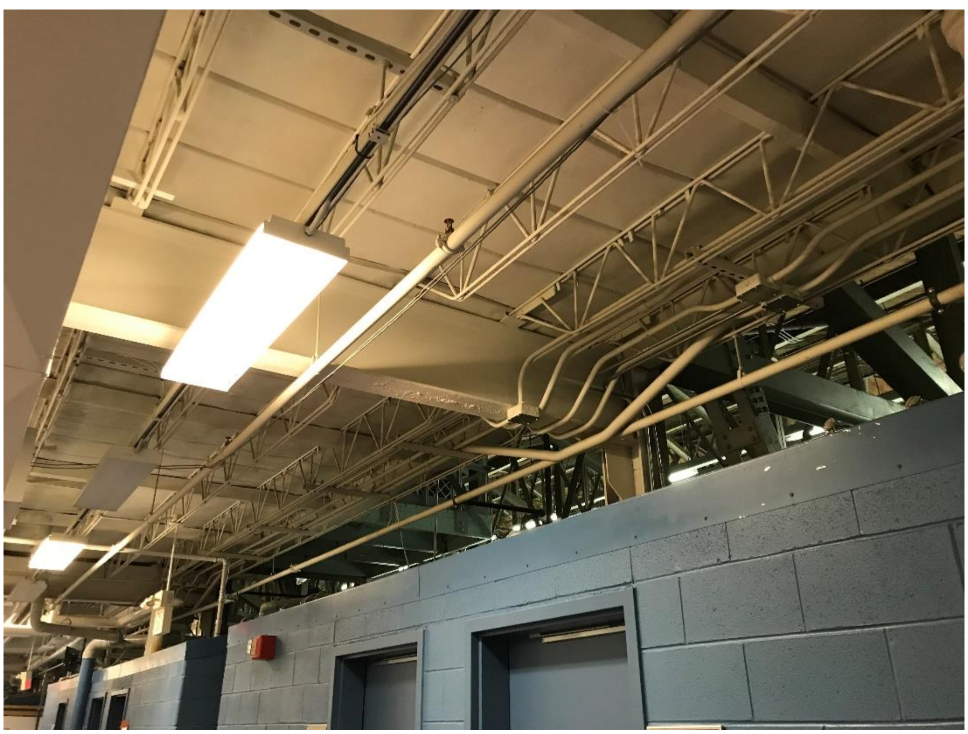

Figure 2: General view of low roof framing.

APPENDIX B – LIMITATIONS

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- No party other than the Client shall rely on the Consultant's work without the express written consent of the Consultant, and then only to the extent of the specific terms in that consent. Any use which a third party makes of this work, or any reliance on or decisions made based on it, are the responsibility of such third parties. Any third party user of this report specifically denies any right to any claims, whether in contract, tort and/or any other cause of action in law, against the Consultant (including sub-consultants, their officers, agents and employees). The work reflects the Consultant's best judgement in light of the information reviewed by them at the time of preparation. It is not a certification of compliance with past or present regulations. Unless otherwise agreed in writing by the Consultant, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity; it is written to be read in its entirety.
- Only the specific information identified has been reviewed. No physical or destructive testing and no design calculations have been performed unless specifically recorded. Conditions existing but not recorded were not apparent given the level of study undertaken. Conditions may differ from those observed, which were relied upon to develop our recommendations. Only conditions actually seen during examination of representative samples can be said to have been appraised and comments on the balance of the conditions are assumptions based upon extrapolation. Therefore, this work does not eliminate uncertainty regarding the potential for existing or future costs, hazards or losses in connection with a property. We can perform further investigation on items of concern if so required.
- The Consultant is not responsible for, or obligated to identify, mistakes or insufficiencies in the information obtained from the various sources, or to verify the accuracy of the information.
- No statements by the Consultant are given as or shall be interpreted as opinions for legal, environmental or health findings. The Consultant is not investigating or providing advice about pollutants, contaminants or hazardous materials.
- The Client and other users of this report expressly deny any right to any claim against the Consultant, including claims arising from personal injury related to pollutants, contaminants or hazardous materials, including but not limited to asbestos, mould, mildew or other fungus.
- Applicable codes and design standards may have undergone revision since the subject property was designed and constructed. As an example, design loads (such as those for temperature, snow, wind, rain, seismic, etc.) and the specific methods of calculating the capacity of the systems to resist these loads may have changed significantly. Unless specifically included in our scope, no calculations or evaluations have been completed to verify compliance with current building codes and design standards.
- Timeframes given for undertaking work represent our opinion of when to budget for the work. Failure of the item, or the optimum repair/replacement process, may vary from our estimate. This opinion is therefore given as a reasonable average approximation rather than a specific prediction.

- Qualified design professionals are required to perform additional evaluation (as necessary), design and general review during construction when carrying out the recommendations included in this report. Ongoing monitoring is required to confirm that repair or renewal measures are successful and to identify for changing conditions that would require increased levels of intervention or different repair / renewal strategies.
- Qualified contractors are required to implement any recommendations included in this report.
- Failure to implement the recommendations included in this report and/or failure to maintain building components appropriately could lead to ongoing and accelerated deterioration that may lead to unsafe conditions developing.

March 27, 2019

CITY OF GREATER SUDBURY
200 Brady Street
Sudbury, Ontario
P3A 5P3
Attn: Chad Kobylka

Dear Chad,

Re: **SUDBURY ARENA MAIN ENTRANCE – 240 ELGIN STREET, SUDBURY, ON**
STRUCTURAL REVIEW FOR PARTIAL RE-ROOFING PROJECT

As per your request and our proposal P19024, dated January 25, 2019, we have carried out a general structural review of the existing roof framing in the area scheduled for re-roofing in 2019. The purpose of our review is to:

- Confirm the general condition of the existing roof structure;
- Identify deficiencies in the original design;
- Identify changes that may have resulted in load not considered in the original design;
- Confirm that the weight of the proposed roofing assembly does not constitute a reduction in performance level;
- Assess the potential impact of changes to the thermal performance of the roofing assembly; and
- Identify older roof structures that were designed prior to specific benchmark editions of the Ontario Building Code (OBC) and/or National Building Code of Canada (NBC).

As part of our review we completed the following:

- Visited the site on February 22, 2019 to complete a visual review of the accessible structure and to obtain the necessary measurements to facilitate a structural analysis where exposed by a Contractor at four (4) locations; and
- Analyzed a rational sampling of structural members to confirm their adequacy for the imposed roof loading.

The current re-roofing project encompasses the low roof area above the main entrance at the south elevation of the building off Elgin Street (refer to Appendix B). This roof area is currently under snow watch protocol based on our previous findings relating to roof framing of similar vintage at the building.

The existing roofing assembly is scheduled to be removed down to the existing structural deck and replaced with the following proposed roofing assembly (top-to-bottom);

Existing Roofing Assembly	Proposed Roofing Assembly
- Built-up roofing - Semi-rigid insulation (unknown type and thickness) - Vapour barrier	- Pea gravel in modified asphalt flood coat (5 psf max) - 2-ply modified bitumen base sheet - 1/2" fiberboard sheathing - 3" polyiso insulation - Vapour barrier
Weight: approx. 5-7 psf (assumed)	**Weight: approx. 8.5 psf**
R-Value: 7 (assumed)	**R-Value: 20**

The OBC and NBC apply to the design of new buildings and are not retroactive. Our analysis considers guidelines for the review of existing structures as outlined in 'Commentary L: Application of NBC Part 4 of Division B for the Structural Evaluation and Upgrading of Existing Buildings' of the Structural Commentaries (User's Guide – NBC 2015: Part 4 of Division B). The approach outlined in Commentary L has been used to evaluate the impact of the proposed re-roofing operations on the existing roof structure. The addition of new loads or changes to the thermal properties of a roof constitutes an *upgrade* to the structure, which is to be evaluated using current versions of the OBC or NBC considering load factors prescribed in Commentary L. Alternatively, existing structures that are not being upgraded can be evaluated using the code in force when built except where benchmark editions of the NBC have been identified that introduced significant changes to either the magnitude or extent of loads on roofs. All existing structures built prior to a benchmark edition have been evaluated using current versions of the OBC or NBC considering load factors prescribed in Commentary L. The following benchmark editions generally apply to the evaluation of roof structures:

- **NBC 1965 – Snow drifts due to high roofs and roof obstructions;**
- NBC 1970 – Retained rain loads on roofs due to blocked drains;
- NBC 1990 – Ground snow load changes resulting in significant increases in some municipalities; and
- NBC 1995 – Snow accumulation on large roofs.

No existing drawings were made available for the low roof framing. Based on our site observations and measurements, low roof framing generally consists of concrete on steel pan deck spanning between 8" deep open web steel joists (OWSJ) at 2'-6" centres supported by structural steel beams and columns. The concrete deck thickness was not confirmed during our review. We have assumed a conventional concrete thickness of 2½" for this vintage of construction.

As the arena was designed and constructed c.1951 and prior to the NBC benchmark year for snow drifts, we are of the opinion that the snow loads in force during original construction are un-conservative and not appropriate for our review. We have analyzed the low roof area for snow accumulation loading using the current version of the Building Code with the load factors recommended by the NBC Structural Commentaries. The following reliability index was considered in our analyses:

Factor	Category	Index
System Behaviour	Failure likely to impact people	2
Risk Category	High	2
Past Performance	Record of satisfactory past performance	0
	Reliability Index	**4**

A specified snow loading of up to 205 psf was considered in our analysis based on the size and height difference to the upper roof. We have considered a design dead load of 50 psf based on the self-weight of the roofing assembly, structure, ceiling finishes and suspended electrical/mechanical equipment. Our analysis assumes a steel yield strength of 210 MPa based on the results of previous Leebs hardness testing performed at the building.

Our analysis indicates that the existing OWSJ and steel beams associated with the low roof area are significantly under-designed to accommodate snow accumulation loading within the snow drift area (approximately 50% of the main entrance roof area – refer to Appendix B). We recommend reinforcing the existing structure as required (similar to the low roof area over the Wolves Den). Roof reinforcement would most likely involve local OWSJ reinforcement and the installation of new support beams at OWSJ midspan. Beams and girders supporting the existing OWSJ would also require reinforcing with WT-sections welded to their bottom flange. Additional investigation is required to confirm the extent of reinforcing. Reinforcing details for the compensating structure must be prepared by a Professional Engineer licenced in Ontario, with repairs completed by a qualified Contractor. While no access was provided to review the additional c.1951 low roof areas adjacent the main entrance, we expect that they are similarly designed for a base snow load of 40 psf and require reinforcement (refer to Appendix B).

Our visual review identified light surface corrosion to steel beams and OWSJ where exposed at ceiling opening locations. The current extent of deterioration does not appear to be structurally significant at this time. Additional up-close visual review and cleaning and recoating will be required in conjunction with roof reinforcing. The topside of the roof deck should also be reviewed during re-roofing operations to confirm the condition of the existing concrete.

The Commentaries recommend meeting with the Authority Having Jurisdiction (Chief Building Official) to discuss the findings of the evaluation, and to establish a timetable for any reinforcing work to be done. In the interim, we recommend continuing the current snow watch program for the low roof areas to ensure that snow heights do not exceed 16".

We trust that the above-mentioned information meets your current needs. Should any additional defects or areas of concern be uncovered during the re-roofing, the Contractor must contact a Building Professional for further investigation and review.

Sincerely,

LICENSED PROFESSIONAL ENGINEER
2019.03.27
S. M. J. COLIZZA
100123333
PROVINCE OF ONTARIO

Sam Colizza, P.Eng.
A2S Consulting Engineers

Attachments: Appendix A – Photographs (2 pages)
Appendix B – Roof Plan (1 page)
Appendix C – Limitations (2 pages)

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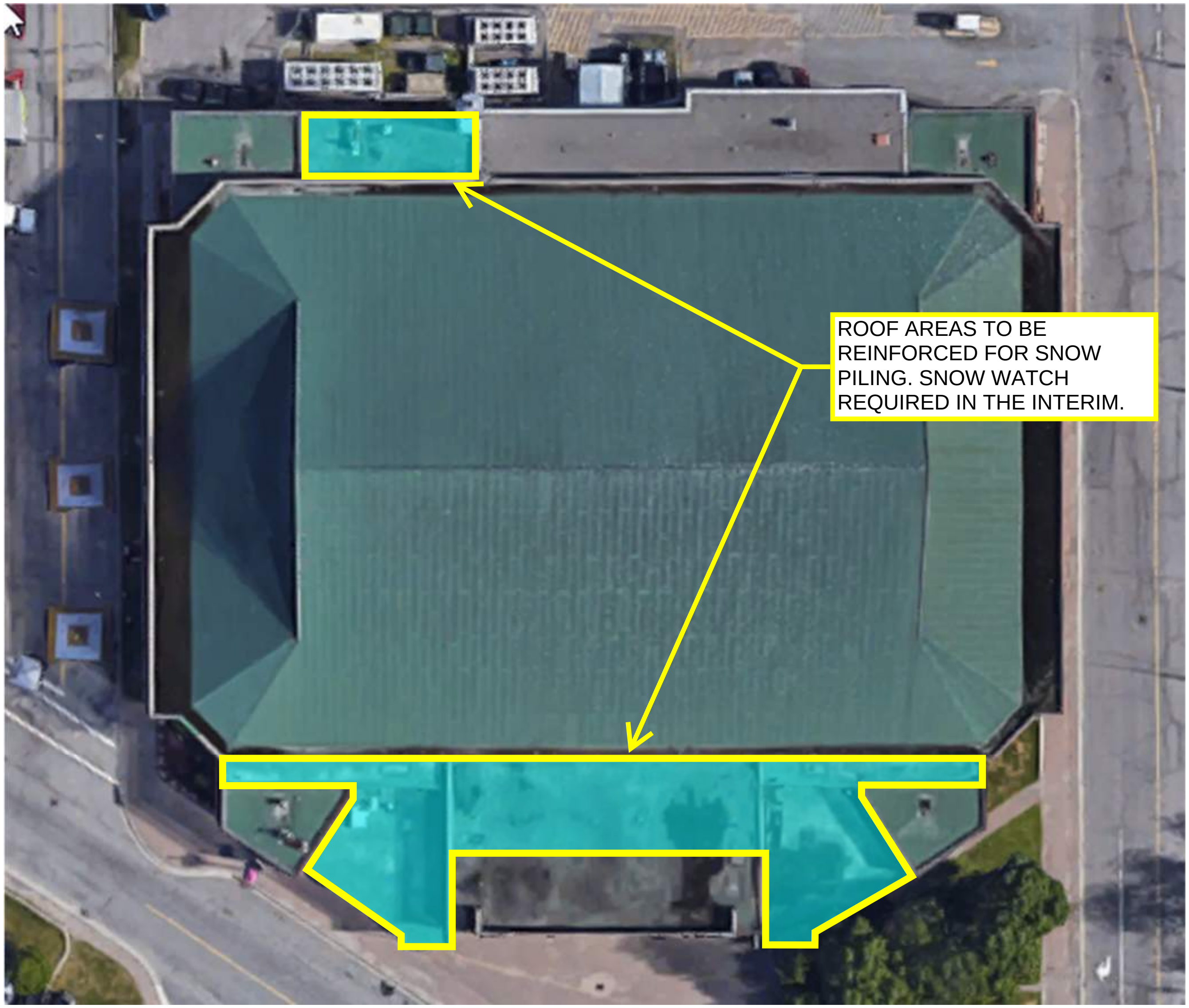
APPENDIX A – PHOTOGRAPHS

Figure 1: Low roof area scheduled for re-roofing in 2019 (photo courtesy of Google Maps).

Figure 2: The roof structure is generally concealed by plaster finishes.

Figure 3: Typical OWSJ framing spanning between steel beams.

APPENDIX B – ROOF PLAN



APPENDIX C – LIMITATIONS

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- Qualified contractors are required to implement any recommendations included in this report.
- Failure to implement the recommendations included in this report and/or failure to maintain building components appropriately could lead to ongoing and accelerated deterioration that may lead to unsafe conditions developing.

April 1, 2019

CITY OF GREATER SUDBURY
200 Brady Street
Sudbury, Ontario
P3A 5P3
Attn: Chad Kobylka

Dear Chad,

Re: **SUDBURY ARENA – 240 ELGIN STREET, SUDBURY, ON**
MAIN ENTRANCE PRECAST CONCRETE PANEL REVIEW

As per your request and our proposal P19024, dated January 25, 2019, we have carried out a general review of the existing main entrance precast concrete panels. The purpose of our review is to evaluate the current condition of the existing support anchors and backup structure to determine potential causes and develop appropriate repair strategies for the identified panel cracking and apparent displacement.

As part of our review we completed the following:

- Visited the site on February 22, 2019 to complete a cursory visual review of the panels from grade and an up-close review of the upper wall areas via lift access from the exterior;
- Returned to site on March 7, 2019 to complete an up-close review of the concealed wall assembly where partially exposed by a Contractor at one (1) location; and
- Returned to site on March 25, 2019 to complete an up-close review of the existing anchors and support structure where exposed by a Contractor at one (1) location.

No existing drawings were available for our review. Based on our site observations, the main entrance precast concrete panels appear to be connected to the backup structural steel framing with masonry ties at regular spacings. Vertical panels appear to be connected using light gauge wire twist ties, while soffit panels appear to be supported in bearing by structural steel lintels and tied back using double j-bolt anchors embedded in the panel and drilled through the steel lintel flanges.

Vertical panel cracking was generally widespread at panel edges and the topmost panel at each column was noticeably displaced, particularly at the two (2) outer column locations. We also identified two (2) locations of concrete delamination at panel edges. Concrete cracking and delamination are likely the result of embedded reinforcing bar corrosion.

Our visual review identified corroded and snapped wire ties. The small cross-section of these ties is inherently prone to damage as a result of section loss resulting from water infiltration and the associated corrosion. We expect that panel securement is compromised by the failed ties and the panels are currently relying on a combination of friction and sealant for support. Immediately following our up-close review, a Contractor was engaged to install temporary support to the topmost panels at both outer column locations.

We also identified cracked soffit panels. Soffit panel cracking appears to be isolated to those panels over window jambs. We expect that cracking is the result of restricted panel movement over the vertical panels. The vertical panels should be cutback to provide a larger soft joint to allow for differential movement.

The soffit panel j-bolt anchors generally appear corroded; however, deterioration appears to be limited to light surface corrosion to the nuts and bolts. Anchors were not installed at all pre-drilled holes in the supporting beam flanges and nuts were not fully threaded. The anchors have no appreciable resistance to seismic forces and could result in a falling hazard under relatively minor earthquake forces.

As a result of the observed defects, we recommend that precast panel cracks be epoxy-injected, concrete delamination be patched and that retrofit panel securement be installed by using a series of helical ties drilled through the panels into the backup structure. The backside of the panels should be exposed from the interior by removing plaster wall finishes to allow for review of tie installation.

We identified light surface corrosion to the structural steel backup members and lintels. The current extent of deterioration does not appear to be structurally significant at this time. Inspection ports should be cored into the sides of vertical panels to confirm the extent of deterioration at column bases where prone to de-icing salt use and the associated chloride-induced corrosion.

Sealant joints between adjacent panels are generally cracked and debonded. Sealants are beyond the end of their service life. There is also excessive staining on the soffit panels due to inadequate water-shedding at the leading edge. Sealant replacement and new drip edge flashings are required to mitigate progressive freeze-thaw deterioration to precast panels due to prolonged exposure to moisture and subsequent moisture absorption. The existing staining can also be cleaned (and recoated), if desired.

We recommend that all repairs be completed prior to next winter to address the observed defects prior to another progression of free-thaw cycling.

The recommendations provided herein identify the minimum scope of work required to address the identified defects and to mitigate the risk of progressive deterioration and a potentially unsafe condition from developing. While it is the most cost-effective strategy, it will impact aesthetics. Alternative management strategies are also available, including panel replacement with new precast concrete or Exterior Insulation and Finish System (EIFS) cladding to improve aesthetics and renew the entrance appearance, but at a higher cost.

We trust that the above-mentioned information meets your current needs. Please do not hesitate to contact us with any questions or concerns.

Sincerely,

LICENSED PROFESSIONAL ENGINEER
2019.04.01
S. M. J. COLIZZA
100123333
PROVINCE OF ONTARIO

Sam Colizza, P.Eng.
A2S Consulting Engineers

Attachments: Appendix A – Photographs (5 pages)
Appendix B – Limitations (2 pages)

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APPENDIX A – PHOTOGRAPHS

Figure 1: General view of main entrance precast concrete panels.

Figure 2: Typical vertical panel cracking.

Figure 3: Typical displacement of topmost panel on outer column.

Figure 4: Typical snapped wire tie.

Figure 5: Immediate temporary panel support.

Figure 6: Soffit panel cracks and staining.

Figure 7: General view of backside of soffit panels.

Figure 8: Surface corrosion and missing anchors at all pre-drilled holes.

Figure 9: Typical soffit anchor j-bolt. Surface corrosion and nut not fully threaded.

Figure 10: Typical sealant joint deterioration.

APPENDIX B – LIMITATIONS

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