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BUILDING CONDITION ASSESSMENT REPORT
CITY OF STRATFORD COOPER SITE
350 DOWNIE STREET, STRATFORD, ONTARIO

Issued Date: June 25, 2012

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RJC Project # TOR103282.0003

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Executive Summary

Introduction

Read Jones Christoffersen Ltd was retained by Aird & Berlis LLP on behalf of the City of Stratford to undertake a physical condition assessment of the building structure located at 350 Downie Street in the City of Stratford, Ontario, commonly referred to as the "Cooper Site".

Purpose

The primary purpose of this investigation was to determine the as-built condition of the building steel framing structure as it relates to the overall structural integrity of the building at the above noted site. In conjunction with the building structure, a review of the slab-on-grade, roof deck structure, roofing system, and exterior cladding elements was also undertaken as part of the investigation.

The findings of the condition assessment were used to evaluate the capability of typical structural elements within the existing building structure, given their present condition, to withstand current building design loads for the purpose of establishing the probable cost to remediate the structure in comparison with the cost to completely dispose of the structure.

Summary of Findings and Conclusions

The findings of our field survey has concluded that original construction deficiencies (e.g. missing rivet bolts), in service use (e.g. impact damage), fire related member failure and warping, and corrosion related deterioration of the steel superstructure exposed to rain and snow has locally reduced the structural capacity of the affected roof framing and column members. If the building is to be brought back in to a serviceable condition, rehabilitation of the observed deficiencies and deterioration is required. Furthermore, in order to avoid future growth of corrosion related deterioration, measures are required to protect the structure against rain and snow.

In order to analyze the structure's ability to support loads based on the Ontario Building Code (OBC), a computer modeling program called SAP 2000 (version 15) was used to model typical truss systems for the roof framing. Our theoretical structural analysis has found that in general, the typical roof trusses are capable of supporting the intended vertical loading, assuming that the members have not undergone any section loss due to corrosion; however, the roof beams spanning between trusses are not adequate to support the loads imposed. Reinforcing of the roof beams are required to meet the minimum load requirements of the OBC

Possible Courses of Action and Construction Cost Estimates

The following rehabilitation strategies were presented based on the findings of this report:

Option No. 1: "Do Nothing" Approach

As a minimum, if nothing is done to improve the functional performance of the roofing and exterior wall systems and thus limit the structure from ongoing deterioration, we recommend that the City further restrict access to the property and building by erecting a permanent fenced enclosure around the property. **Estimated Construction Cost: $375,000.00 plus HST**

Option No. 2: Rehabilitation of the Superstructure

The purpose of this strategy is to essentially repair, reinforce, restore, and protect the structure of the building to reinstate the structural integrity of the building and allow for its future re-use. Furthermore, with the future use of the building and projected timing of construction unknown, protection of the structure would also be required to maintain its integrity for that period of vacancy.

The following scope of work is the minimum recommended work required to restore the structural integrity of the building and protect it during its period of vacancy:

1. Wholesale removal and disposal of the existing roofing systems, including decking, strapping, vertical cladding at each apex, etc.;
2. Sandblast all steel to bright, clean steel;
3. Replace warped roof purlins;
4. Reinforce roof purlins;
5. Reinforce damaged and deteriorated truss members;
6. Chip concrete around bases of deteriorated columns;
7. Reinforce deteriorated column webs and flanges;
8. Repair delaminated and deteriorated mezzanine concrete;
9. Repair exterior concrete walls by removing and repairing delaminated concrete and injecting cracks in concrete;
10. Repairs to brick veneer, masonry infill, and cladding;
11. Coat all structural steel with Galvafroid or other protective coating;
12. Install new cladding and glazing in existing openings;
13. Install new roofing assembly including strapping and decking;
14. Replace all roof drains and rain water leaders;

Estimated Construction Cost: $4,640,000.00 plus HST

The following scope of work is the minimum amount of additional work required to allow a basic occupancy/use of the building:

1. Chip delaminated and elevated portions of slab-on-grade to relatively level and clean surface and place thin topping over entire area to provide a flat floor surface. Fill in pits and holes and repair delaminated concrete prior to placing topping;
2. Provide the minimum levels of heating and ventilation required for basic occupancy.
3. Provide the minimum levels of insulation and thermal protection required for basic occupancy.
4. Provide the minimum levels of fire and life safety systems required for basic occupancy.
5. Provide the minimum levels of security required for basic occupancy.
6. Provide the minimum levels of exterior cladding required for basic occupancy

Estimated Construction Cost: $9,720,000.00 plus HST

<u>Option No. 3: Building Demolition</u>

This strategy involves the complete demolition of the building structure, including sub-structure elements, down to grade. The purpose of this strategy is to end up with a brownfield site graded to the approximate current ground elevation for future development purposes as deemed appropriate by the City of Stratford.

Estimated Construction Cost: $1,200,000.00 plus HST

Recommendations

The existing structure at the Cooper Site in Stratford has suffered varying levels of deterioration, which is significant in some areas, as a result of its exposure to exterior elements and environment as well as fire damage. Salvaging the structure for future use and occupancy would require rehabilitation, reinforcement, and protection of the structure. Such work would entail repairs, replacements and reinforcement of the steel framing, concrete rehabilitation and repair work, complete removal and replacement of the roofing assembly including drainage, and significant repairs to exterior concrete, masonry, and metal cladding elements. However, complete demolition of the building would result in a substantial reduction in cost versus rehabilitation of the structure. Furthermore, the cost to construct a completely new building of similar magnitude may prove to be cheaper than any proposed retrofit to the existing building.

1.0 Introduction

Read Jones Christoffersen Ltd was retained by Aird & Berlis LLP on behalf of the City of Stratford to undertake a physical condition assessment of the building structure located at 350 Downie Street in the City of Stratford, Ontario, commonly referred to as the "Cooper Site".

This report is exclusively for the use and benefit of the City of Stratford and is not for the use or benefit of, nor may it be relied upon by, any other person or entity. The contents of this report may not be quoted in whole or in part, or distributed to any person or entity other than the City of Stratford.

1.1 Purpose

The primary purpose of this investigation was to determine the as-built condition of the building steel framing structure as it relates to the overall structural integrity of the building at the above noted site. In conjunction with the building structure, a review of the slab-on-grade, roof deck structure, roofing system, and exterior cladding elements was also undertaken as part of the investigation.

The findings of the condition assessment were used to evaluate the capability of typical structural elements within the existing building structure, given their present condition, to withstand current building design loads for the purpose of establishing the probable cost to remediate the structure in comparison with the cost to completely dispose of the structure.

(Refer to Section 9.0 of this report for Report Limits of Liability).

1.2 Scope of Work

As part of our physical condition assessment, the following work, briefly described below, was carried out:

.1 Review of all available original drawings, documents, and past reports describing the structure and its condition.

.2 Comprehensive visual review of the building structure to determine its as-built construction and physical condition, including obtaining the physical size and shape of each of the typical structural elements (i.e. roof beam, trusses, columns, etc.), as well as the configuration of the various structural connections.

.3 Full photographic survey of the structure including a video overview of the entire structure, with the observation fully logged on field note drawings created by RJC.

.4 Review of the slab on grade floor and foundation by means of ground penetrating radar and test pits at typical locations to determine the construction profile of the slab (i.e. thickness, use of beams, variations in thickness, etc.) and its structural framing orientation.

.5 Theoretical structural analysis of the various typical structural elements comprising the roof, floors, and supporting columns to confirm their current capacity and the structural limitations of the individual elements.

.6 Determination of the probable construction cost to remediate/rehabilitate the structural components of the building (i.e. roof frames, columns, slabs), as well as the probable cost to remove and dispose the entire building, returning it back to a brownfield site.

1.3 Site Visit and Project Personnel

Representatives from Read Jones Christoffersen Ltd were on site between October 3 and November 18, 2011 to perform the visual review of the building. Personnel conducting the review were:

- Mr. Michael Pond, P.Eng. – Read Jones Christoffersen Ltd (RJC) – Footings and Foundations
- Mr. Carlos Alegre, P.Eng. – Read Jones Christoffersen Ltd (RJC) – Structural
- Mr. Michael Park, C.E.T. – Read Jones Christoffersen Ltd (RJC) – Building Roofing and Cladding Systems

In addition, test-pits were made at four (4) column locations to establish typical footing conditions between November 10, 2011 and November 11, 2011 and steel samples were extracted from various non-critical components of the building structure on November 9, 2011 to determine the strength of the materials used on site.

1.4 Cost Basis

The costs presented in this report are broken down into the cost of remediating the structure versus the cost of complete demolition of the structure. All costs shown here are in 2012 Canadian dollars.

2.0 Building Description & History

2.1 Building and Site Description

The building located at 350 Downie Street is an abandoned industrial building constructed circa 1871 generally consisting of riveted steel construction and currently covering a footprint of approximately 160,000 square feet *(refer to Photo #1 in Appendix A)*. The building has undergone various iterations of additions and demolition over its history prior to and following abandonment in 1989.

The building located at 350 Downie Street was originally constructed in 1871 as a locomotive repair shop with expansions in 1889 and 1907, and an addition in 1949. Currently, only the 1907 expansion and 1949 addition exist on site, with the original building and 1889 expansion having been demolished in 2004 and 2010 respectively. The property is bounded by a community centre on Downie Street to the east, a municipal parking lot and new construction on St. Patrick Street to the north, residences on Wellington Street to the west, and St. David Street to the south *(refer to Photo #2 in Appendix A)*.

The remaining building is generally arranged with four (4) bays, all of which are open from the ground to the roof structure with the exception of the north-most bay, which includes a mezzanine level *(refer to Figure #1 below and attached Building Section in Appendix B)*. From north to south, the north-most bay (herein referred to as the "mezzanine bay") is approximately 615-ft long by 40-ft wide and 50-ft high to its peak. The next bay south (herein referred to as the "low bay") is approximately 770-ft long by 65-ft wide at a similar height of 50-ft to its peak. The 3rd bay south (herein referred to as the "high bay") is approximately 780-ft long by 70-ft wide and 67-ft high to its peak. Finally, the south-most bay (herein referred to as the "addition bay") is approximately 580-ft long by 50-ft wide and 38-ft high to the roof surface.

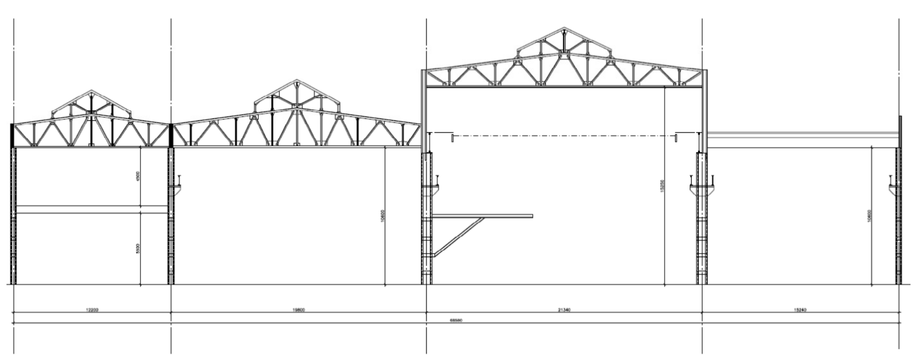

FIGURE #1: TYPICAL BUILDING SECTION

In plan, the main bays are denoted by lettered gridlines As, Cs, Ds, Es, and Fs, spaced in the north-south direction as per the bay width noted above. The transverse gridlines are numbered and identify the column spacing in the east-west direction, generally at 22' centres *(refer to attached Site Plan in Appendix B)*.

The footprint area of the remaining building under review is approximately 160,000 square feet. Access to the building is achieved from a municipal surface parking lot at the north side of the building, where the main entrance can be accessed near the centre of the north side of the building.

2.2 Structure Description

The building structure reviewed ranges from approximately 38-ft to 67-ft tall with the main portion of the building constructed of riveted built-up steel construction and the addition constructed of rolled structural steel sections. The building, in general, is constructed above grade with several pits of unknown depths present throughout the footprint of the building. In particular, the following observations were made based on our site investigation:

.1 Lateral Systems

In general, the steel structure utilizes cross-bracing in the vertical plane along gridlines As, Cs, and Ds and horizontal plane at the roof levels to provide lateral stability. Exterior cast-in-place concrete walls also contribute to the lateral stability of the structure, particularly for the mezzanine bay, which also utilizes the mezzanine floor and three (3) concrete "shafts" for stability. The exterior walls also appear to provide lateral stability in the north-south direction to the entire main building. *(refer to Photos #3 and #4 in Appendix A).*

.2 Foundations/Footings

Limited information regarding the type and size of footings and substructure was available at the time of our review. However, four (4) test pits were completed as part of our engineering services, which enabled us to obtain a general understanding of the below grade substructure for the interior columns.

In general, the substructure for the columns appears to consist of independent concrete piers supported by either spread footings or deep foundations (eg. caissons) *(refer to Photos #5 and #6 in Appendix A)*. This was observed at 3 of 4 test pit locations and further supported by the GPR data (refer to Section 4.2.2.1). Maximum excavation depths due to the size limitations of the excavator did not allow us to verify whether the pier was supported on a spread footing or a caisson cap.

.3 Ground Floor Slab

The ground floor surface is generally slab-on-grade construction interrupted by elevated pads, several pits of varying depths, some filled with sand, train rails and other embedded items *(refer to Photo #7 in Appendix A).* GPR testing indicated that the thickness of the slab-on-grade was not consistent throughout the facility and, in some case, was comprised of multiple layers of slab, likely as a result of changes in specific use of the facility. The GPR sampling conducted on site further indicated that the slab on grade consists of both plain concrete (unreinforced) as well as reinforced concrete with a varied thickness between 200mm to 500mm (8″ to 20″).

.4 Main Building Area

The main building area constructed in 1907 consists of riveted steel with main roof trusses spanning in the north-south direction across each bay (varying from 40′ to 70′) supported by built-up steel column sections. The columns are spaced approximately 22′ apart along the length of the facility. Rolled steel 'C' and 'I' section purlins span between trusses to support the roof deck. Large plate girders are also present within the structure, formerly utilized to support mobile crane loads carrying locomotives.

.5 Mezzanine

The mezzanine floor is typically constructed of a 6″ thick lightly reinforced suspended slab spanning over rolled steel 'I' beams spaced approximately 6′ apart. The steel beams span in the east-west direction between riveted built-up plate girders. The plate girders, in turn, span in the north-south direction between the columns. The mezzanine cantilevers approximately 6′ beyond gridline Es. Three cast-in-place concrete "shafts" exist below the mezzanine floor between gridlines 8-9, 18-19, and 28-29, each with an intermediate floor level.

.6 Addition

The 1949 addition is separated structurally from the main building, supported by its own columns with its own lateral bracing. An expansion joint is also present near the mid-point of the length of the addition. The addition is constructed of rolled structural steel sections with riveted connections. The roof deck is supported by 'C' section beams spanning in the east-west direction with 'WF' section girders spanning in the north-south direction between columns.

2.3 Exterior Cladding and Roof

Having undergone various phases of expansions, additions, modifications, repairs, and demolition, the building utilizes several forms of cladding and roofing systems. During our site investigation, we made the following general observations related to the composition of the cladding and roofing systems:

.1 Exterior Cladding

The main building appears to have been originally constructed with thick cast-in-place exterior concrete walls tied to the building columns with large window openings *(refer to Photo #8 in Appendix A)*. In some areas, the building is clad with corrugated metal siding, which was installed to enclose the building in areas where other sections of the building were demolished such as the east wall and a portion of the north wall near the west end of the building *(refer to Photo #9 in Appendix A).* At the west end of the main building, the window openings were infilled with concrete blocks with a brick veneer installed on the north face only from grade to half of the height of the building in that area *(refer to Photo #10 in Appendix A).* In general, many of the window openings have had the windows removed and have been boarded with plywood, particularly on the ground level windows.

The cladding around the 1949 addition was observed to be constructed of multi-wythe brick bands at the top (including the parapet) and middle of the wall with metal cladding between the bands and above the cast-in-place concrete foundation wall *(refer to Photo #11 in Appendix A).* The metal cladding generally runs the full length of the building and appears to have been placed to protect the windows that originally existed between the bands of brick. The sills of the windows not in contact with the foundation wall were made of precast concrete *(refer to Photo #12 in Appendix A).*

.2 Roofing System

The main building is constructed similarly for each of its three bays, with a higher, sloped roof with a central peak at the central half of each bay (herein referred to as the "apex") elevated by short walls from the low sloped roof on either side. The apex roof areas are generally constructed of sheet metal supported by wood strapping and metal U-channel grid. The walls at the edges of the apexes were generally constructed with wood studs sheathed with plywood, and in some cases cement board, and coated with asphalt felt, similar to the low-slope roof areas below. The assembly of the low-slope roof areas at the outer bands of each bay was typically constructed with mopped multi-ply asphalt roof membrane on solid 2" thick tongue-and-groove wooden roof deck spanning over the steel purlins *(refer to Photo #13 in Appendix A)*.

The roof of the 1949 addition is a flat roof (with mild slope towards the exterior south parapet wall) constructed with a multi-ply roofing system with pea gravel and copper flashing. The membrane was applied to the underlying solid 2" thick tongue-and-groove wooden roof deck.

2.4 East Building Description

For reference purposes, the east building (demolished in 2010) was of traditional wood frame and masonry construction. The roof structure consisted of wood decking, which appeared to have been planks laid flat parallel to the length of the building. The wood decking was supported by roof joists, which were in the direction of the pitch of the roof. The joists were supported by wooden beams, which in turn were supported by peaked wooden trusses spaced at approximately 15 foot centres. The wooden trusses were typically of timber construction and span the full width of the building (i.e. north/south direction) with no intermediate columns. All the timber elements comprising the roof structure were rough sawn elements and the exterior walls of the east building were of solid multi-wythe masonry construction with steel siding as exterior cladding. The roof trusses were supported by a combination of steel columns and masonry piers built integrally with the exterior building walls. The foundations of the exterior walls were likely of masonry construction and the ground floor slab was a concrete slab-on-grade.

Refer to the RJC Report in Appendix D of this report for further information regarding the east building.

2.5 History & Background

The building was constructed by Grand Trunk Railway (GTR) as a locomotive shop to accommodate their growing steam locomotive market, with the site in Stratford being selected as it was located at the crossroads of the main line from Quebec to Chicago and the east-west line from Buffalo to Goderich on Lake Huron. The original shops were completed in 1871. After acquiring Great Western Railway (Hamilton to Detroit), GTR expanded the Stratford facility in 1889 to accommodate the influx of staff and equipment relocated from Hamilton. Another major expansion was constructed in 1907 to provide more space to the increasing size of the locomotive, and a final addition was constructed in 1949 to accommodate even larger locomotives. During that time, GTR was absorbed by Canadian National Railway (CNR) in 1923.

Due to the takeover by diesel engines, CNR no longer required the locomotive repair shops and sought offers for the fully equipped facility in 1953. In 1959, the U.S.-based Cooper-Bessemer Corporation (later named Cooper Energy Services) leased the facility from CNR for its manufacturing purposes. By 1989, due to the turnaround in fortunes for Cooper Energy Services, the building became, and remains, vacant.

Since becoming vacant, the property has seen a few changes in ownership with several proposals and plans put forth for redevelopment of the facility, none of which ever came to fruition. In 2002, a major fire occurred in the west end of the building causing extensive damage. Another smaller fire occurred in 2008, with only minor damages noted. In 2004 and 2010, respective demolition of the 1871 and 1889 portions of the building were completed, leaving the 1907 expansion and 1949 addition as the building currently existing on the site.

2.6 Document Review

The following drawings were provided for our review:

1. *Site Drainage Drawing* with unknown Title, Drawing Number or Author; date stamped in by CNR on October 18, 1935; note on drawings indicates "Source: Perth County Archives)
2. **Arrangement of Drainage and Sewer Systems, Stratford Shops, Grand Trunk Railway System** by The Arnold Company (Engineers – Constructors: Electrical – Civil – Mechanical); Drawing number not indicated/visible; dated August 21, 1907
3. *Railing Plan*; Drawing Number 4CS-5821; Title, Date, and Author not legible
4. **Ground Floor Plan, Mezz. Floor Plan, Building Sections**; Drawing Number MEAS-1 by Thor David Dingman designer Job No. 192 dated February 1992; Issued to Alan Gough for Use and Review on February 14, 1992; Project Name: The "Old Cooper Site" Building Evaluation
5. **Locomotive Shop East End Framing, Stratford Shops, Grand Trunk Railway System** by The Arnold Company (Engineers – Constructors: Electrical – Civil – Mechanical); Drawing number 90100-F5; Dated May 5, 1908
6. **Section Showing Arrangement for Jib Cranes for Erecting Pits, Stratford Shops, Grand Trunk Railway System** by The Arnold Company (Engineers – Constructors: Electrical – Civil – Mechanical); Drawing number 90804-F1; Dated June 6, 1907
7. **Diagram: Sewers and Drains used on Stratford Shops** by Canadian National Railway Mechanical Department, Stratford Shop; Drawing number 55-717 Rev. E dated February 5, 1946

8. *Roof Truss Details* by unknown author; Title, Date, and Drawing Number not visible

3.0 Description of Field Work and Testing

3.1 Visual Review and Documentation

Representatives of Read Jones Christoffersen Ltd performed a visual examination of the building structure, roof structure and roofing, and exterior cladding to identify the extents of deterioration and damage to the building. In general, given the exposed and abandoned condition of the building, the structure and building envelope components have all experienced varying degrees of deterioration.

The roof trusses and beams were reviewed from a boom lift to allow close-up visual inspection of the structural members. Each member was photographed for documentation purposes and observations were recorded on detail sheets identifying general condition of the reviewed structural element as a whole with areas of concern specifically noted in the location observed. In particular, heavy corrosion, pitting and section loss, where observed, were noted on the detail sheets as well as missing rivets, dents, warping etc.

3.2 Test Pits

With the assistance of Mass Contracting, RJC excavated four (4) test pits to expose discreet sections of the interior column substructure. The intent of the test pit excavations was to document the in situ condition of the column foundation and establish typical footing sizes, thicknesses, and depths below grade.

The locations of the test pits have been graphically shown on the Ground Floor Plan in Appendix B.

3.3 Ground Penetrating Radar

The services of multiVIEW Locates Incorporated were retained to perform a Ground Penetrating Radar (GPR) survey of the site to establish general details of the slab-on-grade such as its thickness, reinforcement details, and irregularities. A typical column line, namely gridline ‘20’, was selected and scanned from the south exterior wall to the north exterior in several passes to establish typical slab-on-grade conditions. Next, the length of the building was scanned in the east-west direction in several passes between gridlines ‘Ds’ and ‘Es’ to identify any variations in the slab-on-grade over the length of the site. General imagery from the GPR scan was generated as part of the survey.

The locations of the GPR test locations have been graphically shown on the Ground Floor Plan in Appendix B.

3.4 Material Testing

The services of a Stratford local steel fabricator were retained by RJC to obtain four (4) steel samples from the existing structure. The location of the samples taken from the structure were identified by RJC and chosen such that the structural integrity of the member sampled was not reduced or affected. The samples were delivered to Exova Limited material testing laboratories where nine (9) coupons were extracted by Exova from the samples delivered. The testing was required to gain an understanding of the in situ physical and chemical attributes of the steel with respect to tensile strength and chemical make-up. Both of these analyses were required to accurately determine the grade of steel the buildings were constructed of for purposes of conceptual structural analysis of the steel framed structure.

4.0 Summary of Findings

Based on our experience in the repair and rehabilitation of deteriorated building structures the actual quantity of repair during construction as compared to those deterioration quantities detected during the condition survey will be larger. The information provided in this section with respect to deterioration/delamination detected is actual deterioration and has not been increased to accommodate the final repair of this deterioration.

4.1 Building Superstructure

.1 Main Roof Structure (Original Building)

.1 Visual Survey

With the west end of the original building being exposed due a fire, varying degrees of corrosion and damage was observed in the roof structure. In the fire damaged areas (between gridlines 1 and 12), more extensive surface corrosion was observed on the purlins and trusses, particularly in the apex zone of the roof *(refer to Photo #14 in Appendix A)*. Also observed in that area were warped purlins *(refer to Photo #15 in Appendix A)*, warped members of the centre tie-trusses (secondary trusses perpendicular to main roof trusses) *(refer to Photo #16 in Appendix A)*, as well as typical warping of the truss members on the vertical faces of the apex in the low bay *(refer to Photo #17 in Appendix A)*.

Throughout the remainder of the original building, heavier corrosion and section loss is evident in many areas along the bottom chord of the roof trusses. This occurrence was generally found in the mezzanine and low bays in areas of roof leakage, near pipe leakages, towards the exterior of the building and adjacent to open skylights *(refer to Photo #18 in Appendix A)*. Often, water would appear to funnel down opposing diagonal members of the truss, causing a more concentrated flow of water onto the bottom chord, which is generally where more extensive corrosion and section loss was identified.

In isolated locations, dented members were identified, likely the result of some form of impact during original construction or operation of the facility *(refer to Photo #19 in Appendix A)*. Several locations were identified with more significant rivet corrosion, generally in areas of heavier corrosion of the truss members *(refer to Photo #20 in Appendix A)*. There were also only a couple of locations observed where truss members were bowed out of plane.

.2 Material Analysis

Exova Limited material testing company was provided with steel samples which were used to extract coupons for analysis.

Table 4.1 below summarizes the results of physical and chemical analyses for steel samples taken from the original building:

Table 4.1: Steel Sample Material Analysis

Coupon No.	Ultimate Tensile Strength ksi (MPa)	Carbon Content (% of total mass)
1. Angle Leg (Original)	56.8 (392)	0.13
2. Channel Web (Original)	57.9 (399)	0.17
3. Plate (Original)	58.5 (403)	0.21
4. Rivet (Original)	Avg. Hardness = 75	0.13

A Canadian standard for steel had not yet been established at the time of construction of the original building. However, the ASTM A9 designation for steel was established in 1900, and later published in an ASTM Specification in 1914, which offers an appropriate frame of reference for the grade of steel used for the original building. As per the material analysis, the ultimate tensile strength of the steel in the original building ranges from 56.8 ksi to 58.5 ksi with an average of 57.7 ksi. The ASTM A9 Specification published in 1914 lists the ultimate tensile strength of steel in the range of 55 ksi to 65 ksi, with the yield strength to be taken as one-half of the ultimate tensile strength. The analysis results fall within the specified range for A9 steel and, therefore, the yield strength of the steel used for the structural analysis was half of 57.7 ksi, or 28.9 ksi.

Details of the material analyses can be found in Appendix C.

.3 Structural Analysis

There are three (3) typical roof trusses, one in each bay, within the original building that were considered for analysis. A typical truss from each bay was measured and section properties established where accessible in order to accurately model and analyze each of the typical trusses for vertical loading. The loads considered on the trusses included self-weight, superimposed dead loads from the roof assembly and miscellaneous mechanical and electrical services, and snow loading as per the 2006 Ontario Building Code. The trusses were modeled using CSI's SAP2000 (Version 15) program. In general, the analysis results indicate that the typical roof trusses are capable of supporting the intended vertical loading, assuming that the members have not undergone any section loss. Areas with section loss are to be reviewed on a case specific basis to determine the necessity for reinforcing.

The purlins spanning between the roof trusses were typical in all three bays of the original building. The purlins are simple-span beams with uniform loading and, therefore, were analyzed using simple hand calculations. The results of the analysis suggest that the beams are not adequate to support the loads imposed based on the current building code.

.2 Main Roof Structure (Addition)

.1 Visual Survey

Small areas of flaking paint and minor surface corrosion were observed throughout the roof structure. Areas of heavier corrosion were observed on the top flange of beams and girders in areas where signs of roof leaks and rotting roof deck were evident *(refer to Photo #21 in Appendix A)*. Heavier corrosion and section loss was also observed in some isolated areas, often near the interface of the original building and the addition at gridline Cs. In general, the steel in the addition did not appear to have experienced as much deterioration as the original building.

.2 Material Analysis

As noted for the original building, steel samples were also taken from the addition and delivered to Exova Ltd where coupons were extracted for physical and chemical analyses of the steel material. The results of the material analyses are summarized in Table 4.2 below:

Table 4.2: Steel Sample Material Analysis

Coupon No.	Ultimate Tensile Strength ksi (MPa)	Carbon Content (% of total mass)
1. Angle Leg (Addition)	66.8 (461)	0.21
2. W-Section Flange (Addition)	65.7 (453)	0.18
3. W-Section Web (Addition)	68.8 (474)	0.16
4. Plate (Addition)	65.3 (450)	0.22
5. Rivet (Addition)	Avg. Hardness = 88	0.18

The ultimate tensile strength of the tested samples ranged from 65.3 ksi and 68.8 ksi with an average of 66.7 ksi. Those values are consistent with the Canadian designated S40 steel published in 1935 where the ultimate tensile strength was to range from 60 ksi to 72 ksi and the yield strength was considered to be half of the ultimate tensile strength, but not less than 33 ksi. To be conservative, a yield strength of 33 ksi was used for the structural analysis purposes for the addition.

Details of the material analyses can be found in Appendix C.

.3 Structural Analysis

The beams and girders in the addition are typical throughout that bay, simply-supported with uniform loading and, therefore, were analyzed using simple hand calculations. The analysis results found the beams and girders to be adequate to support the intended loading.

.3 Columns

.1 Visual Survey

The columns in the original building were generally observed to have flaking paint and minor surface rust at various locations throughout the building, in particular in exposed areas such as the fire damaged area, similar to all the structural elements present on site. More significant areas of deterioration were observed at the bases of the columns along gridlines ‘Es’ and ‘Fs’ both at the ground and mezzanine levels. Several roof penetrations for rain water leaders were observed along ‘Es’ with the roofs of the low bay and mezzanine bay both sloped towards those leaders. Failure of the roofing, deteriorated piping, and oversized roof openings allowed the collected water to leak through the roof *(refer to Photo #22 in Appendix A)* along that gridline onto the mezzanine floor, often ponding around the columns. Along gridline Fs, large openings for windows were completely open to allow moisture into the building, causing varying degrees of ponding on the surface of the mezzanine at the bases of the columns along the exterior wall. Much of the water collected on the mezzanine floor would spill over the edges of the mezzanine floor and collect on the ground floor slab, often collecting around the bases of the columns at the ground level. In addition, the open design of the built up columns allowed moisture to pour down the columns and through the mezzanine slab to the ground level. This exposure to moisture near the bases of the columns contributed to more rapid deterioration and corrosion of the columns at their bases, where heavier corrosion and section loss was observed *(refer to Photo #23 in Appendix A)*.

Aside from the typical corrosion at the bases of the columns in the mezzanine bay, some areas were observed with holes burned through the webs of the columns along gridline Fs *(refer to Photo #24 in Appendix A)* and areas with missing rivets along gridline Ds *(refer to Photo #25 in Appendix A)*.

.2 Structural Analysis

An important factor to consider when addressing the columns is that they would have been designed to carry the loads of several large steam locomotives as well as any hoisting equipment and support structure required for the operation. The future use of the site is not known, but in its current unoccupied state, the building will not experience any locomotive type loads and much of the hoisting support structure was removed from the site, further alleviating the columns of significant loads. The columns were analyzed for self-weight of the structure, superimposed dead loads due to roofing, cladding, and miscellaneous services remaining on site, and snow live loads. The columns along 'Es' and 'Fs' were also analyzed for the loads imposed by the mezzanine floor with an assumed occupancy live load of 50 pounds per square foot (psf), as required by the Ontario Building Code for offices on levels above the first floor.

The slenderness of the columns was also taken into account for the analysis. Bracing members along the lettered gridlines offered reduced slenderness in the weak axis of the columns along As, Cs and Ds, whereas the mezzanine slab provided lateral support in both the weak and strong axes for the columns along gridlines Es and Fs. Furthermore, columns along Cs were tied into the concrete walls for lateral stability. However, the strong axis for the columns along Ds were not laterally supported between the ground level and the roof trusses, making slenderness more of a factor when considering the capacity of those columns.

Typical columns on each gridline were analyzed using the truss reactions obtained from results of the SAP2000 models as well as hand calculations to determine the column capacities in comparison with actual loading. All of the analysis results conclusively found the columns to be adequate to support the intended loads from the unoccupied building with reserve capacity to accommodate future use if necessary.

.4 Mezzanine Floor Framing

.1 Visual Survey

Visual review of the mezzanine floor included review of the structural steel as well as 100% visual review of both the top surface and soffit of the mezzanine floor slab. Areas of visible concrete deterioration were identified on a plan with approximate dimensions. Control joints were cut into the slab between columns above all girders. Cracking was observed throughout the top surface of the slab typically running east-west above steel beams *(refer to Photo #26 in Appendix A)*. Few areas of delamination and spalling were visually evident on the top surface, although a greater degree of deterioration was found at the west end of the mezzanine in the fire-damaged area.

The soffit of the slab had more extensive and obvious signs of deterioration, with several areas of exposed wire-mesh reinforcing either due to minimal concrete cover or spalled concrete *(refer to Photo #27 in Appendix A)*. Flaking paint was observed throughout the soffit and rust stains were generally more prevalent near the exterior wall, the cantilevered edge, and around openings and penetrations through the slab *(refer to Photo #28 in Appendix A)*. It should be noted that the type of reinforcement used for the slab could not be confirmed by visual inspection but appeared to be simply a wire-mesh laid in near the middle-bottom of the slab.

The mezzanine floor framing was comprised of steel girders and beams and was generally found to have flaking paint and minor surface corrosion throughout. Areas of heavier corrosion and some section loss were also typically observed near the exterior wall at Fs and near the cantilever beyond Es. Steel members framing openings were observed to experience heavier corrosion and the corrosion was most prevalent on the top and bottom flanges of the beams and girders as well as at the beam connections to the girders *(refer to Photo #29 in Appendix A)*. The cantilevered slab edge channel was observed to be heavily corroded for the length of the mezzanine due to water essentially cascading over the edge of the mezzanine slab during rainfalls, with higher degrees of corrosion observed near the column gridlines *(refer to Photo #30 in Appendix A)*.

.2 Acoustical Survey

As part of the review of the mezzanine slab, a delamination survey using the chain drag method was performed on approximately 100% of the top surface. Locations of delamination were identified on a plan with approximate dimensions.

Only few areas of top surface delamination were identified by the chain drag, mainly in the fire-damaged area. Although a concrete structure exposed to those kinds of extreme conditions would typically experience extensive deterioration and delamination, the presence of only very fine reinforcing near the bottom of the slab may not result in significant enough internal stresses to cause delamination on the top surface, which is most likely the reason for the limited amount of delamination found on the top surface.

.3 Structural Analysis

A typical girder and beam were analyzed for typical loading including superimposed dead loads from the 6" thick slab plus 5 psf allowance for any supported services and an assumed occupancy live load of 50 psf, as per the Ontario Building Code for offices on levels above the first floor. Hand calculations were performed for the simply-supported structural elements with uniformly distributed loading and both the typical girder and beam were found to be adequate to support the loading described above.

With some excess capacity available in the girders and beams, the typical girder and beam were both re-analyzed for a live load of 100 psf and found to be capable of supporting the increased loading.

It should be noted that the steel grade used for the analysis of the mezzanine floor framing was 29 ksi, as per the material analysis results for the original building summarized in Table 4.1.

.5 Lateral Framing

.1 Visual Survey

Visual review of the lateral framing included tying trusses, cross bracing, and tying beams between columns in the north-south direction, cross-bracing in the horizontal plane just below the roof structure, and centre trusses running in the north-south direction between the main roof trusses.

In the addition, no major issues were observed with the horizontal bracing with the exception of 2 bays along gridline Cs near the east end of the building where members of the vertical cross-bracing were torched out *(refer to Photo #31 in Appendix A)*.

In the original building, the tying trusses framing into the main roof trusses were typically warped out of plane in the fire-damaged area, likely the result of heat induced stress-relief due to the lack of an expansion joint in the original building *(refer to Photo #16 in Appendix A)*. The lateral systems in the fire-damaged area also experienced surface corrosion as a result of their exposure, with some heavier corrosion and section loss observed in the upper tying trusses along gridline Cs *(refer to Photo #32 in Appendix A)*.

.6 Exterior Walls

.1 Visual Survey

The exterior walls of the original building were constructed of cast-in-place concrete and although they do not provide any bearing or vertical support to the structure, the walls do offer lateral stability to the structural system and, therefore, must be considered as part of the structure for the purposes of this report.

Large window openings were constructed in the exterior walls between columns and intermediate tying members with concrete bands essentially following the columns vertically and the tying members horizontally with the foundation wall extending approximately 3′ above grade at exterior walls. At the west end of the building, the openings were infilled with concrete masonry units at the exterior walls. Along the south side of the original building (gridline Cs) at the interface with the addition, the openings remain open to the addition. There are no concrete walls present on the east end of the original building and the north side still has original windows in some areas, plywood covering in other areas, and completely open in other areas.

Visual inspection of the exterior concrete walls was performed from the interior and exterior of the building with varying degrees of damage and deterioration noted. In general, cracking and delaminated concrete was observed throughout the walls. Severe deterioration, cracking, and spalling concrete was observed at the west end of the building *(refer to Photo #33 in Appendix A)*. General exterior wall deterioration observed throughout the building included the following:

- Horizontal cracking and sill delamination of horizontal bands for full length;
- Vertical cracking through vertical bands for full length;
- Cracking (vertical, horizontal, and/or diagonal) propagating from corners of window openings;
- Spalling and delamination of concrete corners at top of foundation wall band.

4.2 Building Substructure

.1 Column Footings and Foundations

.1 Test Pits

The four (4) test pits were excavated to an approximate depth of 10′ to identify the layout and dimensions of typical column footings since no foundation details were identified on the existing drawings provided.

.2 Visual Survey

The following information was observed and recorded based on the visual observations made at each of the test pit locations:

Test Pit #1

Test Pit #1 was created on the south and west side of the steel column located at gridline ‘16’ & ‘Es’. The column supports the roof spanning the “high bay” and “low bay”. The excavation was approximately 10′ deep.

The foundation supporting the 32″ by 32″ column pier was observed to be a spread footing or caisson cap with plan dimensions of approximately 72″ by 72″ located approximately 17″ below finished grade (from top of footing elevation). The depth of the footing/caisson cap was measured to be approximately 27″. *(refer to Photo #34 in Appendix A)*.

What appeared to be a concrete grade beam framed into the east face of the footing/pier cap. Its cross sectional dimensions were 33" wide by 35" deep *(refer to Photo #35 in Appendix A)*.

The slab on grade was measured at 7" (175mm) and was reinforced with welded wire mesh.

<u>Test Pit #2</u>

Test Pit #2 was created on the north and east side of the steel column located at gridline '20' & 'Ds'. The column supports the roof spanning the "low bay" and "mezzanine bay". The excavation was approximately 10' deep.

The foundation supporting the 32" by 16" column pier was observed to be a spread footing or caisson cap with plan dimensions of approximately 92" by 76" located approximately 71" below finished grade (from top of footing elevation). The depth of the footing/caisson cap could not be confirmed *(refer to Photo #36 in Appendix A)*.

The slab on grade was measured at 5" (125mm) and was reinforced with welded wire mesh.

<u>Test Pit #3</u>

Test Pit #3 was created on the south and west side of the steel column located at gridline '15' & 'Cs'. The column supports the roof spanning the "addition bay", but not the "high bay" since the structure is separated by an isolation joint due to the different times in which the structure was built (i.e. 1907 original structure and 1949 addition). Only the foundation on the south side supporting the addition roof was reviewed at this test pit. The excavation was approximately 10' deep.

The foundation supporting the column for the addition at test pit #3 was observed to be a 23" wide by 36" deep by 17'-0" long grade beam supported on either end by 24" by 23" concrete piers *(refer to Photo #37 in Appendix A)*. The grade beam- pier system appears to bridge the older foundation system.

The slab on grade was measured at 7" (175mm) and was reinforced with welded wire mesh.

Test Pit #4

Test Pit #4 was created on the north and west side of the steel column located at gridline '17' & 'Cs'. The column supports the roof spanning the "high bay" but not the "addition bay" since the structure is separated by an isolation joint due to the different times in which the structure was built (i.e. 1907 original structure and 1949 addition). The excavation was approximately 10' deep.

The foundation supporting the column pier could not be reviewed at the test pit location due to the presence of a below grade concrete pit. The pit was likely built to allow access to the underside of equipment in a similar fashion to a mechanics pit in an auto service garage *(refer to Photo #38 in Appendix A)*.

.2 Ground Floor Slab

.1 Ground Penetrating Radar (GPR)

The data collected by the GPR scan was evaluated and summarized by multiVIEW. In general, the scan identified the following details related to the slab-on-grade:

- In some areas, there appears to be 2 floor slabs, the upper layer between 200-250 mm thick and the lower one from 150-250mm thick, with total thickness ranging from 200mm (in areas with one slab) to 500mm (total thickness with 2 slabs);
- The depth of engineered fill below the slab-on-grade varies from 1000mm to 1500mm below the slab surface;
- Various areas of the slab-on-grade were unreinforced;
- Some areas of the slab-on-grade may have voids below due to washing out of soil/fill materials;
- Reinforcing steel in the east-west direction was generally spaced at 150mm centres.

.2 Visual Survey

In conjunction with the visual review, members of the consulting team also performed a topographic survey to establish approximate elevations and the surface profile of the slab-on-grade. The visual review also included a complete chain drag of all accessible/uncovered areas of the slab-on-grade to determine the extent of surface deterioration of the slab-on-grade. The following observations were made during the visual review of the slab-on-grade:

- Several dirt infilled pits throughout the footprint of the building;
- Several elevated concrete pads;
- Locations were observed with old anchors and fastenings to the slab;
- Train rails cast into the concrete slab;
- Several areas of varying sizes of debonded concrete topping;
- Several small areas of delaminated concrete scattered throughout the site;
- Few isolated areas of pitted concrete surface;
- Areas of differential settlement;
  (refer to Photos #39 to #44 in Appendix A)

4.3 Building Envelope

.1 Exterior Cladding

.1 Visual Survey

Varying states of damage and deterioration were observed during visual review of the exterior cladding. Observations made in relation to the cast-in-place exterior concrete walls are summarized in sub-section *4.1.6 Exterior Walls*, above. Metal cladding elements observed throughout the building were in relatively good condition with some areas of minor corrosion noted near exposed edges. The brick veneer at the northwest corner of the main building has experienced some weathering and moisture related deterioration as well as an area of significant impact damage *(refer to Photo #45 in Appendix A)*.

The exterior brick walls of the addition have experienced some significant deterioration around the entire addition. Severe deterioration of the brick was observed at most window sills, including spalling of the precast sills as well *(refer to Photo #46 in Appendix A)*. Some less severe deterioration was also observed above the windows at lintels. Brick replacement work was evident in several isolated areas, particularly below window sills *(refer to Photo #47 in Appendix A)*. The newer bricks were also observed to have some levels of deterioration. Spalling of the brick was also observed for the full height of the building near the corners of the addition, particularly at the interface between the main building and the addition *(refer to Photo #48 in Appendix A)*.

.2 Roof Assembly

.1 Visual Survey

In general, the roofing was observed to be in poor condition, which was particularly evident upon observing the varying degrees of deterioration at the underside of the roof deck in all areas of the building *(refer to Photo #49 in Appendix A)*. Flaking paint, damp and rotting wood, and corroded sheet metal were observed throughout the building in both the main building and the addition *(refer to Photo #50 in Appendix A)*. Since the condition and stability of the roof deck material was questionable, only a cursory review of the roof surfaces could be performed through the skylights of the building from a boom lift. From the cursory review of the roof surface performed, it was obvious that the roofing materials had well exceeded their useful service life, were in a state of complete disrepair, and were no longer functioning as intended.

5.0 Conclusions/Discussions

5.1 Superstructure

.1 Visual Investigation

Varying extents of deterioration of the steel structure were observed during the site investigation. Having been abandoned for over 20 years and exposed to the outdoor environment in some areas, the structure has experienced deterioration resulting from exposure to moisture and annual temperature fluctuations. Extensive deterioration is evident throughout the concrete exterior walls where some significant delamination and spalling of concrete was observed. The structural steel was coated with lead paint, acting as a sacrificial coating similar to galvanizing commonly used today. Due to the temperature fluctuations and moisture issues, much of the paint has flaked off of the structural members. Damage to the structure resulting from a fire that occurred in 2002 was also observed.

In general, the visual review of the site identified the following signs of damage and deterioration that would require repair, reinforcement, or replacement:

- Fire damage observed from gridlines ‘1’ to ‘13’ and ‘Cs’ to ‘Ds’, resulting in the following: warped truss, beam, and cross-bracing members that require replacement; roofing and roof deck completely burned away, particularly at apex; more extensive surface corrosion as a result of greater exposure to moisture. Some of the warping in cross-bracing and tying trusses can be attributed to stress relief of the structural system of the facility as there are no expansion joints present in the older portion of the building.
- Areas of significant steel section loss (10% to 20%) on main truss members require replacement or reinforcing. This is particularly evident on the bottom chord of members near open skylights, where windows were removed and some other areas with localized roof leaks.
- Significant corrosion at bases of columns along gridline Es requires reinforcing. Several rain water leaders are present along this line where the pipes are leaking and/or the roof is leaking around the pipe. The dripping/flowing water accumulates on the surface of the mezzanine floor and the ground floor around the columns.
- The steel within the addition was in relatively good condition with little deterioration observed. One area, however, was observed where cross-bracing and a steel beam were torched out, likely to accommodate delivery or maneuvering of a large piece of equipment. Reinstatement of the removed

section is required to reinstate the structural integrity of the building at that location.

- Isolated and randomly located dents in steel members were noted throughout the facility and documented wherever observed. Most of the dents observed were likely a result of an impact either during construction or operation of the facility. At this time, the impact damage does not appear to be affecting the structural integrity of the steel framed superstructure; however, its original load carrying capacity has been locally reduced and the damaged areas should be repaired.
- Missing rivets were identified in various areas throughout the facility, particularly on plate girders and columns. Some instances were regular while others appeared random. No areas were observed where a missing rivet was a result of excessive deterioration.
- Cracking was observed throughout the mezzanine floor, with delaminations observed mostly on the soffit of the slab. No significant reinforcing steel was observed within the concrete, with the exception of wire mesh. The concrete floor experienced several areas of ponding and moisture accumulation, particularly near the columns (both exterior and interior).

The following remedial work would be recommended based on the above summarized observations:

- Sandblast all structural steel to clean, bright steel;
- Replace or reinforce all warped truss members;
- Replace all warped purlin members with new steel members to match existing section dimensions;
- Reinforce truss members with section loss by adding steel bar or steel angle stitch welded to the deteriorated member with appropriate development length beyond the deterioration as required;
- Reinforce bases of deteriorated steel columns by welding steel plates to the web and/or flanges to make up lost section area. Note that chipping of concrete encasing the column would be required to expose any buried deterioration;
- Replace torched-out members with new steel members to match existing section properties in the addition;
- Where deemed critical, reinforce any dented truss members;
- Place new, fully torque, bolts where rivets are identified as missing. Bolt size to match original rivet diameter and to include washers;
- Remove any deteriorated concrete on the top-surface and soffit of the mezzanine slab. Clean any exposed reinforcing to bright steel. Patch areas of

delamination with polymer-modified cementitious repair material. Rout and seal all top-surface cracking with rigid concrete crack repair material;

.2 Structural Analysis

Typical structural elements were analyzed for current building code design loading to determine the general structural adequacy of the building. Based on the results of the analyses, the following conclusions were made:

- Roof trusses were found to be capable of supporting assumed loading;
- Roof purlins in the original building were not adequate to support assumed loading and reinforcing is required;
- Roof girders and beams in the addition were found to be capable to support assumed loading;
- Columns were generally established to be capable of supporting current assumed loading;
- Mezzanine girders and beams were found to be capable of supporting an assumed occupancy load of 50 psf;
- Reinforcing within the mezzanine slab could not be verified, therefore, reinforcement of the mezzanine slab, may be required.

Results of the analyses concluded that the structure is generally capable of resisting the imposed loads, with the exception of the roof purlins in the original building. Reinforcement of the roof purlins would be required, which could be achieved by stitch welding an additional section to the bottom of each member to increase their capacity. Given the limited information available in relation to the mezzanine slab, reinforcement of the slab might also be required to accommodate occupancy loads.

5.2 Substructure

The substructure (foundation system) for the building appears to consist of independent spread footings and/or deep foundations at the individual column locations throughout the original building as well as the addition. No conclusive evidence was available to establish the type of foundation; however, based on the data collected from the test pit locations and GPR testing it does not appear as though a "raft slab" foundation was used in the construction of this building.

Furthermore, our review of the superstructure did not identify any visually obvious signs of differential settlement of the steel framing system and therefore it can be inferred that the foundations are able to support the loads applied to the superstructure (snow, occupancy/use).

Review of the slab-on-grade was also considered as part of the substructure investigation. Pits and train rails were scattered throughout the slab. The GPR scan of the slab identified that the slab was reinforced with some areas having more significant reinforcing, which is consistent with the fact the building was used to accommodate some very significant machinery and equipment. Also noted in the GPR scan was the potential of multiple levels of slab-on-grade in isolated areas possibly due as a result of infilling of a former pit. From our visual review, cracking was evident throughout the slab-on-grade and several areas of ponding water were observed following heavy rain falls; however, the noted types and extent of deterioration of the slab on grade does not appear to impact the overall structural integrity of the building.

5.3 Exterior Cladding and Roof

Although the main scope of the investigation was to assess the condition of the building structure, a cursory review of the cladding and roofing systems employed was undertaken to provide a further understanding of the levels of deterioration observed as a result of the environment to which the building was exposed. The cladding elements such as the brick and cast-in-place concrete have experienced significant moisture related deterioration throughout the building including several areas of spalling brick and concrete. The roofing was in an obvious state of disrepair and the moisture penetrating through the membrane has caused some significant levels of deterioration and failure of the decking materials. In general, the cladding and roofing systems were found to be in poor to very poor condition.

6.0 Possible Courses of Action

As per the mandate put forth by the City of Stratford, two courses of action were considered as part of this investigation. The first option is to salvage the existing structure, making necessary repairs to maintain its structural integrity. The second option is to completely demolish and dispose of the building, including the slab-on-grade and footings, providing a sound grade and turning over the site as a brownfield site to the City of Stratford. In both cases, the future use of the site was not considered.

6.1 Option #1: "Do Nothing" Approach

The "Do Nothing" approach is a hypothetical analysis that explores the option of allowing the building to continue to exist in its present state, assuming it remains unused/abandoned and that it continues to deteriorate as it is exposed to weathering.

It is generally accepted that the deterioration of a building and it's individual systems and elements that are exposed to rain, snow, and wind will typically deteriorate at accelerated rates if left unprotected. The rate of deterioration typically increases at an even faster rate for derelict or abandoned buildings that are no longer heated/cooled (conditioned) or do not have functioning roofing and exterior wall systems. As such, an increased risk of localized or total collapse can be expected as the building continues to deteriorate.

As a minimum, if nothing is done to improve the functional performance of the roofing and exterior wall systems and thus limit the structure from ongoing deterioration, we recommend that the City further restrict access to the property and building by erecting a permanent fenced enclosure around the property.

6.2 Option #2: Rehabilitation of Superstructure

The purpose of this strategy is to essentially repair, reinforce, restore, and protect the structure of the building to reinstate the structural integrity of the building and allow for its future re-use. The extent of reinforcing required would generally be limited to areas exhibiting section loss caused by deterioration or other forms of damage as well as those members found to be inadequate for current loading requirements, namely the roof purlins in the original building. Prior to performing any reinforcing work, preparation of the site and structure would be necessary; all of the roofing materials, including the decking and strapping, would need to be removed and all of the steel would need to be sandblasted to clean, bright steel. Upon completion of any reinforcing, repair, or replacement work to the steel structure, the roof assembly would need to be replaced with new. Exterior walls would also require repairs to ensure their stability and restore their structural integrity, particularly where severe cracking and delamination of the exterior concrete walls was observed west end of the original building.

With the future use of the building and projected timing of construction unknown, protection of the structure would also be required to maintain its integrity for that period of vacancy. Placement of a new roofing system, complete with decking and strapping as required would offer a great deal of protection to the structure. Use of a protective coating, such as Galvafroid, on the structural steel could also prove useful if the space is not expected to be conditioned. Cladding any wall openings would offer additional protection against the ingress of moisture and strategically placed glazing panels could be introduced to allow the heat from the sun to warm the interior space during winter months. These sorts of measures would be required to compensate for the lack of HVAC services within the space.

Mandatory Scope of Work

The following scope of work is the minimum recommended work required to restore the structural integrity of the building and protect it during its period of vacancy:

7. Wholesale removal and disposal of the existing roofing systems, including decking, strapping, vertical cladding at each apex, etc.;
8. Sandblast all steel to bright, clean steel;
9. Replace warped roof purlins;
10. Reinforce roof purlins;
11. Reinforce damaged and deteriorated truss members;
12. Chip concrete around bases of deteriorated columns;
13. Reinforce deteriorated column webs and flanges;
14. Repair delaminated and deteriorated mezzanine concrete;
15. Repair exterior concrete walls by removing and repairing delaminated concrete and injecting cracks in concrete;
16. Repairs to brick veneer, masonry infill, and cladding;
17. Coat all structural steel with Galvafroid or other protective coating;
18. Install new cladding and glazing in existing openings;
19. Install new roofing assembly including strapping and decking;
20. Replace all roof drains and rain water leaders;

The above scope of work is the minimum level of work required to address the present levels of deterioration and restore the integrity of the structure and protect it during its period of vacancy. The advantages and disadvantages to salvaging the existing structure are discussed below:

Advantages

- The main shell of the structure is already constructed;
- The capacity of the existing columns and footings are conducive to the addition of one, or more, floor levels within the building;
- Large bay widths allow flexibility for future use of the space;
- Soil remediation is not mandatory.

Disadvantages

- Limited to existing building footprint and structural arrangement;
- Cost to restore structure much higher than demolition of the existing structure;
- Structure likely to remain vacant and exposed for a prolonged period of time, potentially leading to recurrence of deterioration requiring subsequent rehabilitation;
- Greater potential for unforeseen site conditions to present difficulties during any potential retrofit project.

Required Scope of Work for Occupancy

The following scope of work is the minimum amount of work required to allow a basic occupancy/use of the building:

21. Chip delaminated and elevated portions of slab-on-grade to relatively level and clean surface and place thin topping over entire area to provide a flat floor surface. Fill in pits and holes and repair delaminated concrete prior to placing topping;
22. Provide the minimum levels of heating and ventilation required for basic occupancy.
23. Provide the minimum levels of insulation and thermal protection required for basic occupancy.
24. Provide the minimum levels of fire and life safety systems required for basic occupancy.
25. Provide the minimum levels of security required for basic occupancy.
26. Provide the minimum levels of exterior cladding required for basic occupancy

The above scope of work is the minimum that is required to achieve basic occupancy/use and does not allow for any level of substantial architectural finishes for flooring, walls (interior and exterior), or ceiling systems.

6.3 Option #3: Building Demolition

This strategy is relatively self-explanatory, essentially involving the complete demolition of the building structure, including sub-structure elements, down to grade. The purpose of this strategy is to end up with a brownfield site graded to the approximate current ground elevation for future development purposes as deemed appropriate by the City of Stratford.

The following scope of work is the minimum recommended work required to demolish the existing building and deliver a brownfield site:

Mandatory Scope of Work

1. Protection of the site for the duration of the demolition work to restrict access only to contractor and consultants as well as maintain site safety;
2. Demolition of above grade structure down to slab-on-grade, including but not necessarily limited to:
    a. Wholesale removal of roofing assembly, including roofing materials, decking, flashings, etc.;
    b. Wholesale removal of all building cladding from all exterior walls of the building, including concrete walls, metal cladding, and masonry walls and veneer;
    c. Wholesale demolition of the steel structure including trusses, beams, purlins, columns, cross-bracing, etc.;
    d. Wholesale demolition of above grade concrete structures including walls and mezzanine floor slab;
    e. Salvage steel elements from the site and deliver off-site to steel recycling facility (salvage value retained by the contractor);
    Dispose of remaining building materials such as wood, roofing materials, and other waste at appropriate off-site disposal facilities, preferably at recycling facilities where appropriate;
3. Demolition of below grade structures, including, but not necessarily limited to:
    a. Wholesale demolition of sub-grade concrete structures including slab-on-grade, footings and foundations, etc.
    b. Recycle all demolished and removed concrete elements by crushing and reusing on the site as a granular fill material meeting the standards of a Granular 'B' material;
    c. Deliver a clean site graded to the approximate current ground elevation;
    d. Soil remediation is NOT included as part of the scope of work.

The above scope of work will allow for delivery of a brownfield site, providing an open lot that would be prepared for future development plans. The advantages and disadvantages to completely demolishing the existing building are discussed below:

Advantages

- With the exception of soil remediation requirements, this option provides a 'clean slate' for future development;
- Not limited to existing footprint, orientation and arrangement of the existing building and structure;
- New construction can often be delivered faster and cheaper than retrofit construction of comparable magnitude;
- Value of salvaged materials can help reduce the cost of this option;
- Initial capital cost much lower than Option #1

Disadvantages

- As opposed to Option #1, left with no structure remaining on-site;
- Brownfield site soil must be remediated prior to any new development;
- Environmental impact due to disposal of materials;

7.0 Opinion of Probable Construction Costs

The following cost estimates for rehabilitation of the super-structure versus complete building demolition of this facility represents our opinion of the probable construction costs and are based on the information obtained during this condition survey. The following cost estimates should be treated as "ball park" figures only and cannot be guaranteed accurate.

Based on the construction review experience we have in the repair and rehabilitation of existing structures and buildings, we advise that it is reasonable to assume that the repair quantities - as compared to those deteriorated quantities observed during the condition survey - will be larger. Different items for repair characteristically have exhibited different increases in size during the repair program. Our summary to follow, which outlines the estimated construction costs, has considered this increase from the observed deteriorated quantities.

7.1 Option #1: "Do Nothing" Approach

The construction cost estimate to build a permanent perimeter fencing system around the property (approximately 2,500 linear feet of 8" chain link security fencing), as described in Section 6.1 of this report, has been included as part of this report. The estimated cost of this option, assuming all work is performed in one year in 2012 dollars, is approximately $375,000.00 plus HST.

7.2 Option #2: Rehabilitation of Superstructure

The construction cost estimate for the rehabilitation of the superstructure, as described in Section 6.2 of this report, has been included as part of this report. The estimated cost of this option, assuming all work is performed in one year in 2012 dollars, is approximately $5,830,000.00 plus HST. For a complete breakdown of the cost for this option, refer to Table 7.2.

Table 7.2a – Option #1: Rehabilitation of Superstructure – Mandatory Structural Upgrades

Item	Description	Estimated Value
Scope of Work		
1	Removal and Disposal of Existing Roofing Assembly	$ 240,000.00
2	Sandblast Steel to Bright, Clean Steel	$ 500,000.00
3	Replace Warped Roof Purlins	$ 200,000.00
4	Reinforce all Roof Purlins	$ 330,000.00
5	Reinforce Damaged and Heavily Deteriorated Truss Members	$ 150,000.00
6	Chip out Concrete at Bases of Deteriorated Columns	$20,000.00
7	Reinforce Deteriorated Column Webs and Flanges	$ 50,000.00
8	Repair Exterior Concrete Walls	$ 50,000.00
9	Repair Brick Veneer, Masonry Infill, and Cladding	$ 100,000.00
10	Coat Structural Steel with Protective Coating	$ 200,000.00
11	Install new Cladding and Block Infill Existing Openings	$100,000.00
12	Install new Roofing Assembly including Strapping and Decking	$ 1,600,000.00
13	Replace all Roof Drains and Rain Water Leaders	$ 100,000.00
14	Bonding, Mobilization, General Accounts, Overheads	$650,000.00
15	10% Contingency Allowance	$ 350,000.00
	Total ("Class D" - Cost Estimate)	**$ 4,640,000.00**

Table 7.2b – Option #1: Rehabilitation of Superstructure – Minimum Occupancy Upgrades

Item	Description	Estimated Value
Scope of Work		
1	Repair Delaminated and Deteriorated Mezzanine Concrete	$ 70,000.00
2	Repair and Resurface Slab-on-grade	$ 800,000.00
3	Minimum Required Heating and Ventilation	$ 1,200,000.00
4	Minimum Required Insulation and Thermal Protection	$ 500,000.00
5	Minimum Required Fire and Life Safety Systems	$ 800,000.00
6	Minimum Required Security Systems	$ 250,000.00
7	Basic Exterior Cladding & Glazing	$ 3,000,000.00
8	Miscellaneous Provisions and Upgrades	$ 1,000,000.00
9	Bonding, Mobilization, General Accounts, Overheads	$ 1,350,000.00
10	10% Contingency Allowance	$ 750,000.00
	Total ("Class D" - Cost Estimate)	**$ 9,720,000.00**

7.3 Option #3: Building Demolition

The construction cost estimate for complete demolition of the structure, as described in Section 6.3 of this report, has been included as part of this report. The estimated cost of this option, assuming all work is performed in one year in 2012 dollars, is approximately $1,200,000.00 plus HST. For a complete breakdown of the cost for this option, refer to Table 7.2.

Table 7.3 – Option #3: Building Demolition

Item	Description	Estimated Value
Mandatory Scope of Work		
1	Site Protection	$ 50,000.00
2	Demolition and Disposal/Recycling of Superstructure	$ 300,000.00
3	Demolition and Disposal/Recycling of Substructure	$ 600,000.00
4	Bonding, Mobilization, General Accounts, Overheads	$ 150,000.00
5	Contingency Allowance	$ 100,000.00
	Total ("Class D" - Cost Estimate)	**$ 1,200,000.00**

8.0 Recommendations

The existing structure at the Cooper Site in Stratford has suffered varying levels of deterioration, which is significant in some areas, as a result of its exposure to exterior elements and environment as well as fire damage. Salvaging the structure for future use and occupancy would require rehabilitation, reinforcement, and protection of the structure. Such work would entail repairs, replacements and reinforcement of the steel framing, concrete rehabilitation and repair work, complete removal and replacement of the roofing assembly including drainage, and significant repairs to exterior concrete, masonry, and metal cladding elements. However, complete demolition of the building would result in a substantial reduction in cost versus rehabilitation of the structure. Furthermore, the cost to construct a completely new building of similar magnitude may prove to be cheaper than any proposed retrofit to the existing building.

In consideration of the estimated costs, advantages, and disadvantages of each of the options, **we recommend complete demolition of the building outlined in Option 2 as described in Section 6.0 of this report to provide a brownfield site for future development for an estimated construction cost of $1,200,000.00 plus HST.** At the discretion of the City of Stratford, the cost of this option could be reduced to approximately $470,000.00 by deleting the substructure demolition scope of work, allowing that part of the work to be completed as part of any future development of the site.

9.0 Limits of Liability

The review of this property was of a visual nature only. Destructive testing was performed to expose typical column footings and steel samples were extracted to ascertain the grade of the construction materials. The intent of the investigation was to determine areas of visually obvious deterioration and need for repair and to determine, in a general way, the overall integrity of the existing structure.

Our review of the systems did not include a review of the safety aspects of the installation as this falls under the Jurisdiction of the Governing Authorities. In addition, testing of the building materials for Occupational Health and Safety or substance of potential environmental concern was not conducted.

This report is intended to provide the client with a general description of the systems employed in the building and to comment on their general condition, which may be apparent at the time of our inspection. Our comments are not a guarantee or warranty of any aspect of the condition of the building, whatsoever.

Drawings made available were used solely for the purpose of obtaining design information on elements hidden from view which the Engineer or his sub-consultants may require, supplemental to their visual inspection, in order to more fully describe the building but no comments can be made as to the construction of those elements.

Any and all previous opinions expressed by Read Jones Christoffersen Ltd., either verbally or in writing, regarding the condition or cost estimates for repair of the above elements are superseded by this report. The above costs are budget figures only, are based on the current market and are in present dollars. The actual costs may vary depending on the time of tendering, the actual detailed scope of work and market conditions.

Whereas any cost estimates done by the Engineer or his sub-consultants are based on incomplete or preliminary information and on factors over which the Engineer or his sub-consultants has no control, the Engineer or his sub-consultants do not guarantee the accuracy of these cost. Unless otherwise noted, costing information does not include H.S.T. or engineering and testing fees. Costs are based on "Current Year" Canadian Dollars and assume the work in each discipline is completed in one phase.

This report has been prepared for the exclusive use of Client. Read Jones Christoffersen is, however, prepared to provide a reliance letter to future owners of this property. The contents of this report may not be quoted in whole or in part of distributed to any person or entity other than by the Client of those parties possessing a reliance letter. Read Jones Christoffersen Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

10.0 Closing Remarks

Thank you for selecting Read Jones Christoffersen Ltd. for this project. RJC would be pleased to assist you with the implementation of our recommendations. Should you have any questions or concerns, please do not hesitate to contact this office.

READ JONES CHRISTOFFERSEN LTD.

for Carlos Alegre, P.Eng.
Project Engineer
Building Science and Restoration

Reviewed by:

Michael Pond, P.Eng
Associate
Building Science and Restoration

Appendix A
Photographs

Photo #1: General view of existing building from exterior

Photo #2: Aerial photograph of the site

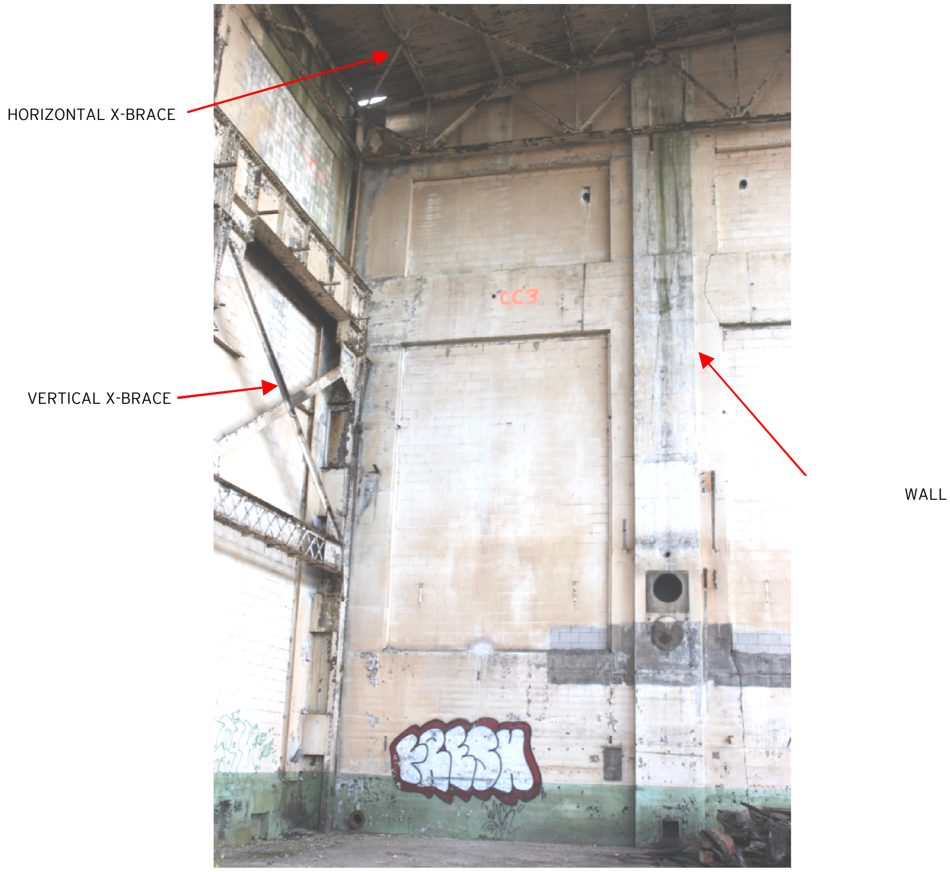

Photo #3: General view of typical lateral framing elements

Photo #4: Lateral stabilizing elements of mezzanine bay (slab and shaft)

Photo #5: Concrete pier in test pit location

Photo #6: Concrete pier in test pit location

Photo #7: Typical embedded items in slab-on-grade

Photo #8: Typical cast-in-place concrete walls with block infilled openings

Photo #9: Metal clad segment of the building (west end)

Photo #10: Brick cladding up to mid-height of building (northeast corner)

Photo #11: Metal cladding infill of window openings at Addition Bay

Photo #12: Typical pre-cast concrete window sill

Photo #13: General view of typical roof soffit

Photo #14: Typical steel corrosion in apex of roof in fire damaged region

Photo #15: Typical warped purlins in fire damaged region

Photo #16: Typical warped tie-truss in fire damaged region

Photo #17: Typical warped apex vertical member in fire damaged region

Photo #18: Typical corrosion along bottom chord of roof truss

Photo #19: Typical dent/kink in truss member

Photo #20: Typical rivet corrosion

Photo #21: Typical roof leak causing corrosion on top flange of girder in Addition

Photo #22: Typical storm water leader and corrosion at top of column

Photo #23: Typical corrosion at column base

Photo #24: Burned hole through column web

Photo #25: Typical missing rivets on column along gridline Ds

Photo #26: Typical mezzanine slab cracking

Photo #27: Mezzanine slab soffit delamination and exposed reinforcing steel

Photo #28: Typical corroded steel and slab soffit deterioration near slab opening

Photo #29: Heavily corroded bottom flange of steel girder below mezzanine slab

Photo #30: Typical corrosion of steel at mezzanine slab edge

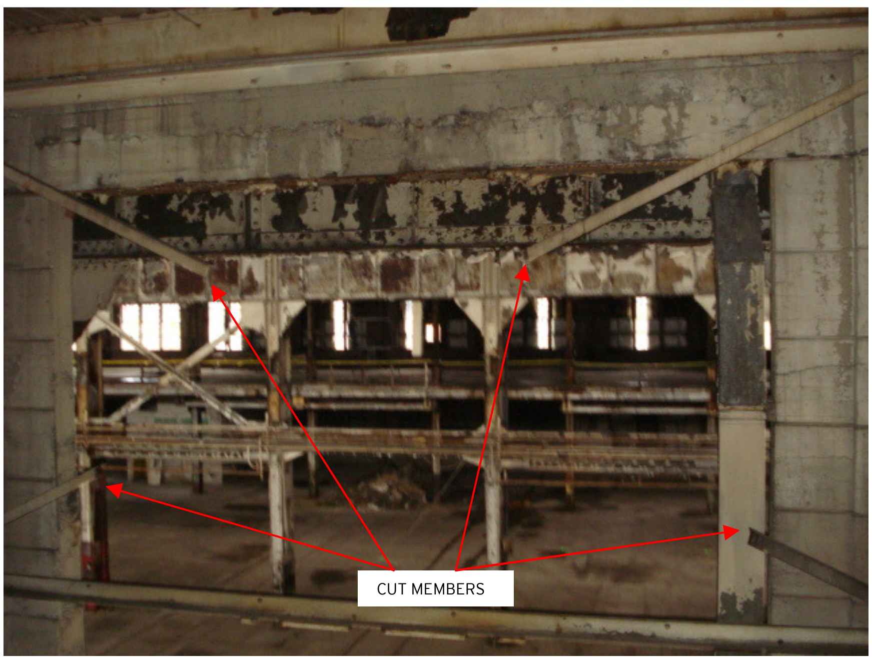

Photo #31: Vertical cross-bracing cut-out in addition

Photo #32: Corrosion at tying truss near grid 'Cs-1'

Photo #33: Exterior concrete wall deterioration

Photo #34: Test Pit #1 – Footing / Caisson Cap

Photo #35: Test Pit #1 – Face of potential Grade Beam

Photo #36: Test Pit #2 – Footing / Caisson Cap

Photo #37: Test Pit #3 – Grade Beam and Piers

Photo #38: Test Pit #4 – Concrete Pit interference

Photo #39: Typical slab-on-grade dirt infill area

Photo #40: Typical slab-on-grade raised concrete pad

Photo #41: Typical slab-on-grade embedded anchors

Photo #42: Typical slab-on-grade embedded train rails

Photo #43: Slab-on-grade area of deterioration/damage

Photo #44: Slab-on-grade area of differential settlement

Photo #45: Impact damage and deterioration of exterior brick veneer

Photo #46: Typical deterioration of concrete sill and masonry below

Photo #47: Typical repaired masonry below window sills

Photo #48: Deterioration of masonry wall at junction with existing wall

Photo #49: Typical roof assembly and condition

Photo #50: Typical roof deck rotting and deterioration