REPORT ON
PARLIAMENT HILL – CENTRE BLOCK
NORTH ELEVATION
MASONRY-STEEL GIRDER ASSEMBLY
OTTAWA, ONTARIO

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EXECUTIVE SUMMARY

This report presents the results of a structural condition assessment of an existing steel girder that supports an exterior load-bearing masonry wall on the north elevation of Parliament Hill – Centre Block, which is a “Classified” Federal Heritage Building and is also a part of the Parliament Hill National Historic Site of Canada.

During a routine screening by HCD staff in August 2012, it was noted that a previously repaired exterior crack in the masonry at the steel girder top flange had reopened. Further investigation noted broken stones, stones moving outwards, and corrosion of the embedded steel girder.

JLR was retained by PPB to undertake a structural condition assessment of the area of concern. JLR’s scope included the review of background documentation, visiting the site to perform visual inspections on the exterior and interior faces of the wall, visiting the site to review portions of the girder exposed by selective dismantling of the masonry and take detailed measurements, perform structural calculations to assess the current risk to the structural integrity of the masonry-steel girder assembly, and prepare a report summarizing findings and make recommendations for repairs.

The investigation did not include destructive testing and selective dismantling of the masonry was restricted to the outer stone whythe. This report assumes that the portions of the existing steel girder reviewed are representative of the overall condition of the girder and that elements supporting the girder have sufficient capacity to resist the reactions imposed by the girder.

The existing steel girder is a built-up steel plate box girder that spans 12421 mm (40’-9”) between the west Ventilation Tower and the west Water Tower on the north elevation. The steel girder is located over the ceiling of historic room 216N (Speaker’s Dining Room). Refer to the figures in Appendix ‘C’ which show the framing elements supported on the existing masonry-steel girder assembly.

Documentation provided by PWGSC includes multiple condition assessment reports, selected original construction drawings, the original construction specification, and selected HCD North Dormer Screening Record Drawings from 2005 through 2012 (Appendix ‘D’).

The original construction drawings and specification were used to determine the material properties and framing for the structural analysis of the steel girder.

Review of previous condition assessment reports noted the following key points:

- During construction, material shortages were experienced and masonry work was performed during freezing conditions. Some materials were rejected from the site due to poor quality.

- Various masonry repair programs have been completed in 1933, 1952 and 1974; with other minor repairs with no reported records. Many different types of repair mortars have been used from soft lime to hard cement-based mortars.

- Structural steel elements embedded in the masonry are corroding, leading to degradation of the surrounding masonry.

- The rate of deterioration of unrepaired masonry increases exponentially.

- Roof drip edges are in need of repair and do not adequately shed water away from the walls.

- Windows, flashings and sealants are in poor condition and at the end of their service life.

JLR visited the site on December 9, 2012 and January 10, 2013 to observe the masonry-steel girder assembly and take detailed measurements of the portions of the existing steel girder top flange and outer web exposed by selective dismantling of the masonry surrounding the steel girder. Refer to the photographs in Appendix ‘A’ which present a representation of our observations.

The exterior face of the existing steel girder is concealed by the exterior Nepean Sandstone facing. Two areas of stone facing containing broken and cracked stones had been removed, exposing the edge of the steel girder top flange. These two areas were chosen for selective dismantling to increase the area of the exposed steel girder to include the web for detailed measurements.

The following observations were noted on the exterior face:

- The exposed areas of the steel girder are located immediately to the east and west sides of the North Gable respectively.
- Black soiling of the stone was noted on either side of the North Gable.
- The top flange consists of three plates riveted together and connected to the web plate with an angle section. Refer to Appendix 'B' for sketches identifying the girder construction and site measurements.
- Surface rust was observed on both the top and exterior side of the girder top flange. Some structural flaking was noted at the edge of the girder top flange.
- Hammer sounding indicated voids between the exterior edge of the girder top flange and outer whythe of stone.
- A number of mortar joints have been previously repointed, and a number of broken and cracked stones were noted.

The following observations were noted on the interior face:

- The steel girder is concealed by the interior brick masonry backing, except the bottom flange which is encased in concrete.
- The interior brick has damp proofing applied from the third course to the underside of the House of Commons Public Gallery Seating Area.
- Efflorescence was noted on the interior face of the brick near the west end of the steel girder.

A structural analysis of the steel girder was performed using both Working Stress Design (WSD) and Limit States Design (LSD) methodologies. Bending moment Demand/Capacity ratios of 0.85 and 0.61 were determined for WSD and LSD methodologies respectively. Shear Demand/Capacity ratios of 0.72 and 0.75 were determined for WSD and LSD methodologies respectively. A Demand/Capacity ratio of less that 1.0 indicates that the existing steel girder is not overstressed. Based on these findings the existing steel girder embedded in the masonry wall has sufficient capacity to support the applied loading.

In order for steel corrosion to occur, two elements are required – water and oxygen. The presence of chlorides such as salt will increase the rate of corrosion. Surface rusting of the girder top flange was noted; however, this rust appears to be limited to the top plates and did not appear to show signs of significant loss of section. Given the severe exposure of the masonry wall above the steel girder, it is very important that a properly functioning wall assembly be provided by means of sound stone and mortar joints to prevent water from entering the wall assembly and to allow moisture present within the wall assembly to egress through the mortar joints.

The broken and cracked stones noted at the top flange of the girder are likely caused by a combination of water penetration, freeze-thaw cycles, and localized steel expansion due to corrosion.

Previous repointing work has been met with varying degrees of success due to the varying nature of the repair mortars used. Hard cement-based mortars reduce breathability of the masonry wall assembly which may cause premature degradation due to accelerated freeze-thaw action. Lime-rich mortars have high workability and are more deformable, ensuring a higher degree of bond to the surrounding masonry which will reduce the ingress of water. Lime-rich mortars also have increased breathability.

The efflorescence noted on the interior face of the masonry wall assembly could be an indication that moisture is travelling through the wall from the exterior to the interior. This type of moisture travel is opposite to that of traditional masonry wall assemblies, in which moisture typically travels from the warm, moist interior towards the colder exterior. One possible reason for the atypical efflorescence is that the area above Room 216N is under negative pressure, possibly due to stack effects caused by the adjacent Ventilation Tower and Water Tower.

In order to ensure that the existing steel girder maintains its current condition and to extend its service life, it is recommended that PWGSC consider three options for the installation of a protective coating to the top flange of the girder:

- Do nothing and leave the steel girder "as is".

- Install coating on an "as needed" basis as any future damaged masonry is discovered.

- Install coating to the entire length of the steel girder.

It is estimated that the cost of the protective coating installation would be $10,000 per linear meter of girder, excluding the costs of mobilization and site access.

It is recommended that building envelop maintenance be performed to reduce the amount of water entering the masonry wall assembly, this maintenance includes repointing the exterior stone wythe (the selection of the mortar type is crucial to the effectiveness of the repointing), increasing drip edges at the sloped roofs, repairing or replacing windows and sealants, investigating the interior face of the masonry wall to determine if the area is subject to negative pressures and taking any corrective actions determined to be necessary, and maintaining the annual inspection program to identify areas of damaged mortar or stone to be replaced or repointed to ensure an optimally functioning wall assembly. Options involving building envelope maintenance have not been priced as this was outside the scope of this report.

REPORT ON
PARLIAMENT HILL – CENTRE BLOCK
NORTH ELEVATION
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1.0 INTRODUCTION

The original Centre Block on Parliament Hill was destroyed by fire in 1916 and was subsequently rebuilt between 1916 and 1927. The Centre Block houses the House of Commons and the Senate arms of the Canadian Government. The Centre Block is a "Classified" Federal Heritage Building and is also a part of the Parliament Hill National Historic Site of Canada.

During a routine screening by Heritage Conservation Directorate (HCD) staff in August 2012, it was noted that a previously repaired exterior crack in the masonry located at the existing steel girder had reopened. Further inspection was undertaken and the condition of the masonry wall assembly around the girder was found to include broken stones, stones moving outwards, corrosion of embedded steel elements and voids behind the outer wythe of stone. It was determined by the Parliamentary Precinct Branch (PPB) that a private sector Professional Engineer (P.Eng.) should be retained to undertake a structural condition assessment of the area of concern.

Public Works and Government Services Canada (PWGSC) retained J.L. Richards & Associates Limited (JLR) to perform a structural condition assessment of an existing steel girder which supports an exterior load-bearing masonry wall on the north elevation of Parliament Hill – Centre Block.

The scope of this investigation included:

- Review background information including existing drawings, specifications and reports provided by PWGSC.
- Visit the site to perform a preliminary visual inspection of the existing steel girder on both the exterior and interior faces.

- Visit the site to review portions of the existing steel girder exposed by selective dismantling of the masonry and take detailed measurements.

- Perform structural calculations to determine the applied loads and capacity of the existing steel girder and assess the current risk to the structural integrity of the masonry-steel girder assembly.

- Prepare a report summarizing the findings of the investigation and make recommendations for repairs.

Limitations of this Investigation Include:

- No destructive testing was performed as part of the investigation.

- Our analysis is limited to the existing steel girder and surrounding masonry assembly. It is assumed that the existing towers supporting the steel girder have sufficient capacity to resist the reactions imposed by the girder.

- Due to the existing interior brick back-up and outer stone wythe surrounding the existing steel girder, it is assumed that the portions of the steel girder exposed by the selective dismantling of the outer stone wythe are representative of the overall condition of the girder.

2.0 BASIS OF USE AND RELIANCE

This report has been prepared for the named client, for the stated purpose and for the named facility. Its discussions and conclusions are summary in nature and cannot properly be used, interpreted or extended to other purposes without a detailed understanding of discussions with the client as to its mandated purpose, scope and limitations.

This report has been prepared for the sole benefit and use of the named client and may not be used or relied on by any other party without the express written consent of J.L. Richards & Associates Limited. The report is copyright protected and may not be reproduced or used, other than by the named client for the stated purpose, without the express written consent of J.L. Richards & Associates Limited.

3.0 MASONRY-STEEL GIRDER ASSEMBLY DESCRIPTION

The existing steel girder is a built-up steel plate box girder that spans 12421 mm (40'-9") between the west Ventilation Tower and the west Water Tower on the north elevation. The existing steel girder is located over the ceiling of the historic Room 216N (Speaker's Dining Room).

The existing steel girder supports portions of the following elements:

- High roof framing above the House of Commons (Figure 1, Appendix 'C').
- The dining room floor located above the House of Commons Public Gallery (Figure 2, Appendix 'C').
- House of Commons Public Gallery Seating Area framing (Figures 3a and 3b, Appendix 'C').
- Existing masonry wall located directly above the existing steel girder (Figure 3b, Appendix 'C').
- Low roof beams above Room 216N framing between the existing steel girder and the north wall (Figures 3a and 3b, Appendix 'C').
- Ceiling finishes for Room 216N suspended from the low roof beams.

4.0 REVIEW OF BACKGROUND DOCUMENTATION

4.1 Documentation Provided by PWGSC

The following documents were provided by PWGSC:

- Excerpt from *Centre Block, Parliament Buildings, Ottawa*, pp 182-201, Source: Robert Hunter, Architectural History Branch.
- *Specification* – Parliament Buildings, Ottawa prepared by Darling & Pearson, Architects, 1916.
- *Condition Assessment of Centre Block Masonry – Parliament Hill, Ottawa*, prepared by Heritage Conservation Program, Real Property Services for Canadian Heritage and Environment Canada, March 1999.
- *100% Final Report – Centre Block Roof Masonry – North Towers, Parapets and Chimneys*, prepared by Heritage Conservation Directorate, Real Property Branch,

Architectural and Engineering Services, Public Works and Government Services Canada, April 20, 2004.

- *Centre Block Various Short Term Repairs, Ottawa, Ontario*, Watson MacEwen Teramura Architects, KIB Consultants Inc., Trevor Gillingwater, Conservation Services Inc., Craig Sims Heritage Consultant, Shirtliffe and Associates, April 27, 2012.
- *Selected Relevant HCD North Dormer Screening Record Drawings 2005-2012*
    - 2005-2006 – (3) Drawings
    - 2007-2008 – (2) Drawings
    - 2008-2009 – (1) Drawing
    - 2009-2010 – (1) Drawing
    - 2010-2011 – (1) Drawing
    - 2011 – (3) Drawings
    - 2011-2012 – (1) Drawing

The following original construction drawings were provided by PWGSC:

- FOUNDATION PLAN (WEST PART) - 1 OF 2 and 2 OF 2.
- GROUND FLOOR (WEST PART) – CONCRETE AND STEEL, 1 OF 2 and 2 OF 2.
- GROUND FLOOR PLAN WEST, 1 OF 2 and 2 OF 2.
- MAIN FLOOR PLAN, 1 OF 2 and 2 OF 2.
- FIRST FLOOR PLAN (WEST) - 1 OF 2 and 2 OF 2.
- SECOND FLOOR FRAMING PLAN - 1 OF 2 and 2 OF 2.
- THIRD FLOOR (WEST PART) – STEEL FRAMING PLAN, 1 OF 2 and 2 OF 2.
- FOURTH FLOOR (WEST PART) – STEEL FRAMING PLAN, 1 OF 2 and 2 OF 2.
- ROOF FRAMING PLAN - 1 OF 2 and 2 OF 2.

4.2 Excerpt from Centre Block, Parliament Buildings, Ottawa

This document is an historical account of the architecture and construction of Centre Block. There is no additional structural information included into the document; however, the heritage value and conservation considerations of Centre Block are reinforced.

4.3 Specification – Parliament Buildings, Ottawa (1916)

The following information related to material specifications is contained within the 1916 Specification:

- BRICK MASONRY: All brick masonry walls are noted to be constructed with either solid or hollow brick (location dependant) and are laid with cement mortar consisting of one (1) part Portland Cement to three (3) parts sand, tempered with a small amount of lime.

- EXTERIOR CUT STONE AND GRANITE: The exterior walls primary face stone is noted to be Nepean Sandstone rock-faced shoddies varying from three inches to twelve inches in height. The exterior cut stone are to be laid with “cement mortar” specified under “BRICK MASONRY”.

- STRUCTURAL STEEL:
  - All structural steel is noted to be open hearth medium steel with an ultimate strength not less that 60,000 and not more than 70,000 pounds per square inch (psi) (414 to 483 MPa). This steel is consistent with ASTM A9 Medium Steel for the era of construction, which has a minimum yield strength of 30,000 to 35,000 psi (207 to 241 MPa), or half of the specified ultimate strength.

  - All rivet steel is noted to be open hearth rivet steel with an ultimate strength not less than 45,000, nor more than 55,000 psi (310 to 379 MPa). This rivet steel is consistent with ASTM A9 Rivet Steel for the era of construction, which has a minimum yield strength of 23,000 to 28,000 psi (158 to 193 MPa), or half of the specified ultimate strength.

4.4 Condition Assessment of Centre Block Masonry (1999)

The following important points are noted from the 1999 Condition Assessment report:

- Shortages of materials, including brick, stone, and steel during wartime construction caused construction delays.

- Brick and stonework were laid during freezing conditions, which may affect the freeze-thaw resistance of the mortar.

- Some Nepean Sandstone was rejected from the site due to poor quality:

  Nepean sandstone was obtained locally and used for the rock-faced walling work (roughly square rubble Nepean), for corner stones for the ground floor to the third floor, and for the foundation wall trimmings as in the original building. Reports prepared during the work cite problems with the quality of the Nepean sandstone to the point where some stone was rejected on site. Eventually the situation was resolved by purchasing stone from a number of different quarries in the Nepean area. However, observations suggest that there is considerable variation in the quality of the stone in the building, varying from the most durable of quartzitic material to relatively vulnerable stone which may well have been quarried from beds where the cementing material was alternately partially calcareous, argillaceous or ferruginous.

- Various masonry repairs have been performed, specifically recorded is masonry repointing work in 1933, 1952, and 1974; however, other minor repairs were observed for which there are no records. Many different types of repair mortars have been used on the building façade from soft lime to hard cement-based:

  Water can not evaporate through hard cement-based mortars and it is forced to evaporate through the stone instead. This causes more salt crystallization within the stone surface and accelerates the effect of freeze-thaw action and spalling of the stone.

- Structural steel embedded in, or supporting the masonry is corroding, which is leading to degradation of the surrounding masonry:

  Steel structural elements buried within the stone are corroding and, when they do so, they expand and destroy the surrounding masonry. This problem has been clearly seen at many locations. For example, the window lintel locations in the courtyard parapet walls where the poorly maintained mortar joints have allowed water ingress to the shelf steel plates and caused them to corrode. The expanded corroded steel exerted significant pressure on the stone lintels and caused them to crack.

- The rate of deterioration of unrepaired masonry increases exponentially:

There is clear evidence that the rate of deterioration of unrepaired masonry increases exponentially. It was observed that by 1997 some of the areas of 'immediate concern' had deteriorated to the point where they were now in worse condition than the 'emergency' areas identified in 1994. This has raised a concern regarding the masonry identified in 1994 as requiring 'repair over the medium term' with which no repairs have yet been done.

4.5 Centre Block Roof Masonry – North Towers, Parapets and Chimneys (2004)

The 2004 report on the North Towers, Parapets, and Chimneys, cites many of the same deficiencies noted in the 1999 Condition Assessment report; however, the following additional important points were noted:

- Roof drip edges are in need of repair and do not adequately shed water away from the walls:

  Performance and weathering of the copper connections, flashing, drip details, solder and caulk materials have deteriorated and require extensive repairs or replacement.

4.6 Centre Block Various Short Term Repairs (2012)

The 2012 Various Short Term Repairs report cites many of the same deficiencies noted in the 1999 and 2004 reports; however, the following additional important points were noted:

- Some of the windows, flashing, and sealants located on the North Gable are noted to be in poor condition and at the end of their service life:

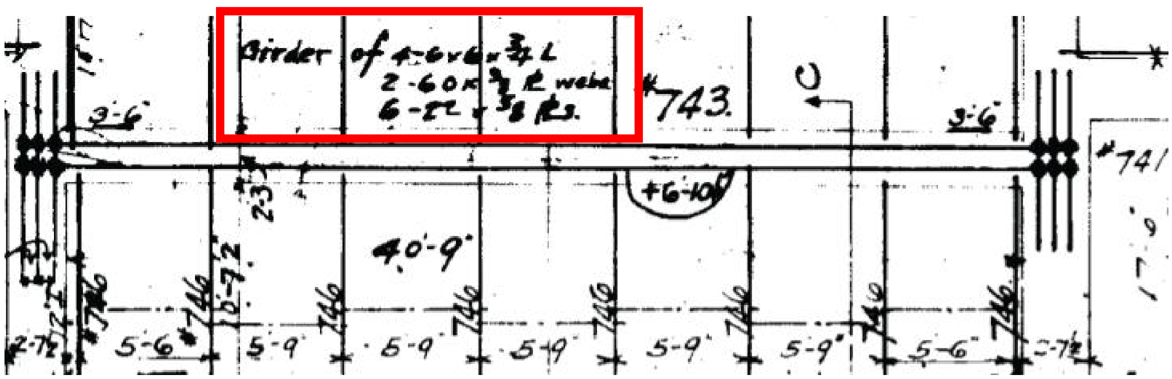
  Window W6-011 is at the end of its service life and should be replaced as soon as possible (see below) – there is next to nothing that can be done to prolong its service life. In the meantime it should be monitored once a year to ensure its structural integrity and also monitor the possibility of the outer wythe of stone is moving at Units C and D. If glass breaks continue, or if portions of the frames and vents continue to corrode through, it will become necessary to remove the units and to replace them with a weatherproof plug.

4.7 HCD North Dormer Screening Record Drawings 2005-2012

Refer to the Selected HCD Screening Record Drawings attached in Appendix 'D'. The drawings indicate a progression over the years of cracked joints, open joints and movement in quoins in the top section of wall around the Dining Room window. This progression appears to have moved down the wall from the top section towards the existing steel girder with evidence of additional cracking and open joints over the height of the wall.

4.8 Original Construction Drawings

The steel girder construction is noted on the first floor framing plan drawing; the drawing indicates the construction as follows:



The PDF scans provided of the drawings do not have sufficient resolution to determine the construction of the girder; however, the assumed construction is as follows:

- (4) – 152x152x19 (6"x6"x3/4") Angles
- (2) – 1524x9.5 (60"x3/8") Web Plates
- (6) – 559x16 (22"x5/8") Flange Plates

Refer to the Sketch SK1 in Appendix 'B' which shows the assumed construction of the steel girder.

5.0 INVESTIGATION

5.1 Preliminary Visual Inspection

Our Messrs. Matthew Burtt, P.Eng., and Rick Westwell, P.Eng., visited the site on December 9, 2012 to conduct a visual inspection of the existing steel girder and surrounding masonry. Both the exterior and interior face of the girder was reviewed.

Exterior Face

Access to the exterior face of the steel girder was provided via a temporary steel stair scaffolding to the low roof above Room 216N. The entire length of the exterior wall between the west Ventilation Tower and west Water Tower was enclosed in a temporary wood-framed heated enclosure to approximately 2 m (6'-6") above the low roof.

The steel girder is concealed by the exterior Nepean Sandstone stone facing, however, two areas of stone facing had been removed from the wall by masons engaged by PWGSC Plouffe Parks Heritage Mason (PPHM), exposing the built-up top flange of the steel girder. The first exposed area is approximately 750 mm (2'-6") long and is located at the east side of the North Gable (Photograph No. 1, Appendix 'A'). The second exposed area is approximately 1600 mm (5'-3") long and is located at the west side of the North Gable (Photograph No. 2, Appendix 'A'). Black soiling of the stone was noted on either side of the North Gable (Photograph No. 3, Appendix 'A').

The sections of the steel girder top flange visible due to removed stone facing consisted of four steel sections riveted together. The four sections consisted of three top plates of similar thickness and a bottom section which was thicker (Photograph No. 4, Appendix 'A'). It is assumed that the bottom steel section is the 152x152x19 (6"x6"x3/4") angle noted on the structural drawings that connects the top flange plates to the web plate. Surface rust was noted on all exposed steel, and some structural flaking was noted at the edge of the girder top flange exposed on the east side of the North Gable (Photograph No. 5, Appendix 'A').

One rivet head was visible in each of the exposed areas. A web stiffener was also visible at each of the exposed areas; it is assumed that the web stiffeners are located where the low roof beams located above Room 216N frame into the steel girder (Photograph No. 6, Appendix 'A').

Hammer sounding was preformed and hollow sounds were noted at the top flange of the steel girder, indicating a possible void between the face of the top flange and the back of the remaining stone facing.

A number of mortar joints had been repointed, including the joints running parallel to the top flange of the steel girder, of which most had nearly been entirely repointed (Photograph No. 7, Appendix ‘A’). A representative from HCD indicated that some of the repointing had been completed in August 2012 as part of work performed to temporarily close and prevent additional ingress of water into the cracks discovered during site review at the north dormer.

A number of stones had broken along the top flange of the steel girder; the cracks in the broken stones had been pointed with the same repair mortar used to repoint existing mortar joints.

Interior Face

Access to the interior face of the steel girder was provided via a crawlspace located below the House of Commons Public Gallery Seating Area.

The steel girder is concealed by the interior brick masonry backing, except for the bottom flange, which is encased in concrete (Photograph No. 8, Appendix ‘A’). No spalling of brick was noted. Hammer sounding was preformed and hollow sounds were noted at the apparent top flange of the steel girder, indicating a possible void between the face of the top flange and the back of the brick.

The interior brick masonry backing has damp proofing applied from the third course to the underside of the House of Commons Public Gallery Seating Area. Efflorescence is visible on the interior face of the brick masonry backing near the west end of the steel girder (Photograph No. 9, Appendix ‘A’).

5.2 Detailed Site Measurements

Our Messrs. Matthew Burtt, P.Eng., and Brent Whaley, P.Eng., visited the site on January 10, 2013 to conduct a review of the exterior face of the girder where selective masonry removals had been completed.

The two areas of exposed top flange noted in the preliminary visual inspection had been enlarged by selective dismantling of the exterior stone wall by masons engaged by PPHM. The two openings penetrated through the entire wall assembly and exposed the entire width of the top flange, a portion of the web plate and stiffener angles (Photograph Nos. 10 and 11, Appendix ‘A’).

Surface rust was observed on both the top and exterior side of the exposed portions of the top flange (Photograph No. 12, Appendix ‘A’). The surface rust was flakey and friable and was measured to be approximately 4 mm (5/32”) thick, except for one area of the side flange at the east opening which had localized flaking over 6 mm (1/4”) in thickness. Very little surface rust was noted on the underside of the top flange. The top flange width was measured to be 559 mm (22”) wide, confirming the width noted on the structural drawings.

Very little surface rust was noted on the web plate. An ultrasonic thickness gauge was used to determine the thickness of the web plate. A number of readings were taken on both the east and west openings and can be found on Sketches SK2 and SK3, Appendix ‘B’. The average web thickness was measured to be 10.3 mm (13/32”).

It was noted that the stone immediately to the west of the west opening had no backing brick or stone between the stone face and the steel girder web face (Photograph No. 13, Appendix ‘A’). Voids were also noted between the exterior face of the top flange and the back face of the exterior stone.

5.3 Structural Analysis

A structural analysis of the steel girder was performed using two design approaches, working stress design, which would have been the design methodology used in the original design of the member, and limit states design using the requirements of CAN/CSA S16-09 – Design of Steel Structures, which is the current methodology used in the design of steel structures.

The structural analysis was based on the following assumptions:

- The working stress design analysis limited the member stresses to 10,000 psi (69 MPa), or one third of the yield stress for shear loading, and 15,000 psi (103 MPa), or one half of the yield stress for bending moments.

- The limit states design analysis utilized a yield stress of 30,000 psi (207 MPa) and a material resistance factor of 0.9.

- No significant reduction in section area due to corrosion of the top flange plates was considered.

- Sufficient rivets are provided between the built-up steel elements to resist the shear flow.

- High roof structure self weight is 3.4 kPa (71 psf) including architectural finishes. Given the copper roofing material and slope of the high roof structure, snow loading was considered negligible on the high roof.

- Floor structure self weight is 4.3 kPa (90 psf).

- Live Load applied to all floor areas is 4.8 kPa (100 psf).

- House of Commons Public Gallery structure total self weight is 153 kN (34.4 Kips).

- The combined unit weight of the brick/stone wall assembly is 21.5 kN/m^3 (137 lb/ft^3).

- Low roof structure self weight is 5.8 kPa (121 psf) including roofing.

- Built-up snow loading on the low roof structure was considered in the analysis.

- Suspended ceiling above Room 216N self weight is 2.4 kPa (50 psf).

Based on the assumed built-up girder construction identified on Sketch SK1 in Appendix ‘B’, the following structural properties of the girder were calculated:

- Depth: d = 1619 mm (5’-3¾”)

- Area: A = 104×10^3 mm^2 (161.2 in^2)

- Moment of Inertia about the Strong Axis: $I_x = 49.7 \times 10^9$ mm^4 (119.4×10^3 in^4)

- Strong Axis Section Modulus: $S_x = 61.4 \times 10^6$ mm^3 (3747 in^3)

Based on the assumptions noted above, the maximum bending moment and shear loads applied to the steel girder were calculated, as were the member bending moment and shear capacities.

The results of the structural analysis for both Working Stress and Limit States Design are summarized in Tables 1 and 2 below. A Demand/Capacity ratio less than 1.0 indicates that the existing steel girder is not overstressed.

Table 1 - BENDING MOMENT

	Applied	Resistance	Demand/Capacity
Working Stress	5425 kNm (4000 K-ft)	6350 kNm (4685 K-ft)	0.85
Limit States Design	6955 kNm (5130 K-ft)	11436 kNm (8435 K-ft)	0.61

Table 2 - SHEAR

	Applied	Resistance	Demand/Capacity
Working Stress	49.8 MPa (7222 psi)	68.9 MPa (10,000 psi)	0.72
Limit States Design	1822 kN (410 Kips)	2436 kN (548 Kips)	0.75

6.0 DISCUSSION OF FINDINGS

6.1 Existing Steel Girder Capacity

Tables 1 and 2 in Section 5.3 show that the steel girder has sufficient capacity to resist the assumed applied loading.

Some surface rusting of the top flange was noted during the site visits; however, this rust appears to be limited to the top plate of the three-plate built-up top flange and did not appear to show signs of significant loss of section. The areas of the web plates exposed by selective masonry removals did not show any signs of significant corrosion.

In order for steel corrosion to occur, two elements are required – water and oxygen. The presence of chlorides such as salt will increase the rate of corrosion. Given that the steel girder is embedded within a masonry wall; the most critical consideration to mitigate the rate of corrosion is to reduce the ingress of water into the masonry assembly, and ensure that moisture present within the assembly can readily egress from the masonry assembly.

6.2 Exterior Face of Wall

The broken stones noted at the top flange of the steel girder are likely caused by a combination of water penetration, freeze-thaw cycles, and steel expansion due to corrosion. Although the amount of structural flaking noted on the outside edge of the steel girder top flange, it is possible that corrosion may be a factor in the broken stones noted.

Given the severe exposure of the masonry wall above the steel girder (north-west orientation and subject to unobstructed wind and driving rain from the Ottawa River corridor), it is very important that a properly functioning wall assembly be provided by means of sound stone and mortar joints to prevent water from entering the wall assembly.

Previous repointing work has been met with varying degrees of success due to the varying nature of the mortars used. As discussed in the referenced 1999 Condition Assessment report, hard cement-based mortars reduce breathability of the masonry wall assembly which may cause premature degradation of the masonry due to accelerated freeze-thaw action. This freeze-thaw action may cause the mortar or stone to crack or spall, allowing additional moisture into the wall assembly.

Cement-rich mortars have high compressive strengths, but have relatively poor workability and hence have reduced bond. These mortars are also subject to higher rates of shrinkage and are less deformable, which can yield to a reduction in bond to the stone/brick over time. All of these factors can contribute to cracking of the mortar joints resulting in moisture penetrating into the masonry assembly.

Lime-rich mortars have low compressive strength, but have high workability and are more deformable, ensuring a higher degree of bond to the surrounding stone/brick, which will reduce the ingress of moisture into the masonry assembly.

The Cement Mortar mix defined in the referenced 1916 Specification is 1:3 Portland Cement:Sand, tempered with a small amount of lime. This mix is similar to that defined in Annex A of CSA A179-04 (R2009) – Mortar and Grout for Unit Masonry as a Type M mortar (with lime) which has the mix proportions of 1:1/4:3-1/2 Portland Cement:Lime:Sand. Annex A of CSA A179-04 defines Type M mortar as:

> *A high-strength mortar recommended when maximum masonry compressive strength is required, or for masonry below grade and in contact with earth. If sulphates are present, a sulphate-resisting cement is sometimes needed.*

Annex A of CSA A179-04 suggests that Type O mortar, a low-strength, high-lime mortar be generally used in the restoration of old masonry structures. The mix proportions of this mortar are noted to be 1:2:9 Portland Cement:Lime:Sand. In areas subject to freezing and higher levels of moisture, an air-entraining agent is recommended for the mortar to achieve an air content of freshly mixed mortar in the range of 10 to 16%. It is also noted that lower-strength mortars are generally less forgiving to errors in design and construction, and the CSA A179-04 committee is currently working to provide better guidance on mortars suitable for traditional masonry.

Representatives from PWGSC indicated that the current mortar used by masons for repointing above grade at Centre Block consists of 1:2.5:8 with an air entrainment of 9 to 12%, which is similar to that of a Type O mortar as defined by CSA A179-04.

ASTM C270-12a – Standard Specification for Mortar for Unit Masonry suggests that mortar used for Tuck-Pointing (repointing) for exterior exposure, exposed to frozen conditions and subject to high wind, use a Type N mortar. The mix proportions of this mortar are noted in ASTM C270 to be 1:1¼:5 to 6¾ Portland Cement:Lime:Sand and in CSA A179-04 to be 1:1:4½ to 6 Portland Cement:Lime:Sand.

ASTM E2260-03 – Standard Guide for Repointing (Tuckpointing) Historic Masonry makes the following recommendations:

> *Substantial disparity between the compressive strengths or other physical properties of the in-situ mortar and the repointing mortar, or the repointing mortar and the masonry units can lead to spalling or other distress of the masonry units. A common approach to ensure compatibility of physical properties is to evaluate the composition of the in-situ mortar and approximate the compressive strength of the mortar before selecting the repointing mortar. The repointing mortar should have similar or lower compressive strength and greater water vapor permeability than the masonry units.*

Black soiling of the exterior stone was noted on either side of the North Gable, but not directly below the North Gable. This soiling may be evidence of an inadequate drip edge at the roof/wall interface at the bottom of the sloped roof. Poor flashing detailing at the wall/roof interface may allow water to penetrate into the masonry wall assembly and run down inside the wall until if finds a way out. Inadequate drip edges may permit the rain runoff to either run down or get blown back onto the face of the wall and subsequently enter the wall assembly at deteriorated mortar joints and cracked stones.

Some windows on the North Gable are noted to be at the end of their service life in previous reports. Poorly functioning flashings and sealants may permit water to enter the wall assembly around the window openings and cause deterioration of the masonry wall assembly.

6.3 Interior Face of Wall

Efflorescence was noted on the interior face of the masonry wall assembly, this could be an indication that moisture is travelling through the wall from the exterior to the interior and depositing dissolved calcium and chlorides present in the stone and mortar on the surface as the water evaporates. This type of moisture travel is opposite to that of traditional masonry wall assemblies, in which the moisture typically travels from the warm, moist interior towards the colder exterior. One possible reason for the atypical efflorescence is that the area above Room 216N and below the low roof is under negative pressure, possibly due to stack effects caused by the adjacent Ventilation Tower and Water Tower.

There is an asphaltic damp proofing paint applied to the interior face of the masonry wall above the level of the low roof. If the cause of efflorescence on the interior face of the wall is due to the area being under negative pressure, the presence of damp proofing on the wall face could be detrimental to the wall assembly as it may restrict water movement through the wall. If the damp proofing is the original asphaltic paint applied during the 1916-1927 construction period, it is likely that it has reached the end of its service life and may be dry and brittle, allowing some moisture to egress through the wall, evident in the deposit of efflorescence on the interior face of the wall.

7.0 RECOMMENDATIONS

Based on the findings of this investigation, the existing steel girder embedded in the masonry wall has sufficient capacity to support the loading applied and has not been subject to significant corrosion over its approximate 85 to 95 year service life. This risk to the structural capacity of the existing steel girder posed by the level of corrosion noted on site is low; however, in order to ensure that the existing steel girder maintains its current condition and to extend its service life, the following recommendations should be considered by PWGSC. Options involving building envelop maintenance have not been priced as this was outside of the scope of this report.

1. Install a protective coating on the outside face and the first 200 mm (8") below the outer wythe of stone at the top flange of the steel girder. This would involve removing the masonry surrounding the top flange, preparing the area steel to receive coating in accordance with manufacturer's recommendations, applying a protective coating to the steel, and reinstating the masonry around the top flange. The cost of this option would be approximately $10,000/m of top flange repaired. This cost does not include mobilization and site access costs which would likely be a fixed price regardless of the option chosen below. The cost assumes that there is a minimal cost associated with the replacement of any broken stones. There are three options associated with this recommendation:
    a. Do nothing – this option proposes leaving the steel girder "as is" and concentrating on keeping water away from the steel. It involves no steel intervention and hence is the lowest cost option.
    b. Install a protective coating on an "as needed" basis – this option proposes that if damaged masonry is noted in future inspections, the steel girder top flange receive a protective coating as part of the repairs to the area of broken stones.
    c. Install a protective coating over the entire length of the steel girder top flange – this option proposes that a progressive installation, removing and reinstating a small section of masonry at a time, be applied over the full length of the steel girder top flange. This option would have the highest installation cost, but would maximize the remaining service life of the existing steel girder.

2. Repoint the exterior face of the wall to limit the amount of water entering the wall assembly. As noted in Section 6.2 of this report, the selection of the mortar type is

crucial to the effectiveness of the repointing to ensure an optimally functioning wall assembly.

3. Increase drip edges at sloped roof. This will ensure that water is transmitted away from the face of the exterior wall.

4. Repair or replace windows, sealants and flashing on the north elevation to prevent water from entering into the wall assembly at window openings.

5. Investigate if the area above Room 216N is under negative pressure. If the area is subjected to negative pressure, additional investigation outside of the scope of this report may be required to determine what, if any, negative effects may result as a result of the atypical conditions. Regardless of the findings of any additional investigation, the existing efflorescence should be removed from the face of the wall and damaged brick units replaced and cracked mortar joints repointed. The removal of existing efflorescence will allow for monitoring of any future efflorescence deposited on the inside face of the wall.

6. Maintain an annual inspection program and identify areas of cracked mortar or stone to be replaced or repointed to ensure an optimally functioning wall assembly.

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