# PRELIMINARY SEISMIC ASSESSMENT FOR CENTRE BLOCK, PARLIAMENT HILL

## Work Package 2

*Prepared for:*
Public Works and Government Services Canada
Parliamentary Precinct Branch

*Prepared by:*
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14Y160-113A

March 2015

Halsall

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PWGSC Client Reference Number: 20150336

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## LIST OF ACRONYMS

| | |
|---|---|
| CSA | Canadian Standards Association |
| DGM | Design Ground Motion |
| FEMA | Federal Emergency Management Agency |
| FRP | Fibre Reinforced Polymer |
| NBCC | National Building Code of Canada |
| PGA | Peak Ground Acceleration |
| OFC | Operational and Functional Component |
| PWGSC | Public Works and Government Services Canada |
| SFRS | Seismic Force Resisting System |
| RPS | Real Property Services |
| UBC | Uniform Building Code |

# 1. EXECUTIVE SUMMARY

## 1.1 Scope

In July 2014, Halsall Associates was engaged by PWGSC to complete a preliminary seismic assessment of the Centre Block on Parliament Hill in the context of the requirements of the latest edition of the National Building Code of Canada (2010 NBCC) and the Real Property Service (RPS) Policy on Seismic Resistance of PWGSC Buildings. Halsall was also engaged to complete targeted research on potential options to seismically rehabilitate heritage buildings based on work done on buildings, both nationally and internationally, that are of a similar scale, type and importance as the Centre Block, as well as to present further investigations and specific upgrade options for the Centre Block.

This report, for Work Package 2 – Preliminary Seismic Assessment, focusses on presenting the gap analysis, the results of the preliminary seismic assessment of the Centre Block, and the qualitative assessment of the non-structural and secondary structural elements.

## 1.2 General Description of Structure

The Centre Block is comprised of three connected components: the main Centre Block building, the Peace Tower and the Library of Parliament. This report considers only the Centre Block and Peace Tower. The Centre Block is a 6 storey building over one basement storey level and is comprised of brick, stone masonry, steel and concrete. It was constructed between 1916 and 1920. The upper floor structure is typically comprised of a cementitious topping on terra cotta flat arches that are supported on steel beams. The steel beams are typically supported on either steel columns or unreinforced stone and brick masonry walls. The lowest floor structure is reinforced concrete, while the sloped roof structure is typically expanded metal forms with a cementitious topping on sloping steel beams. The foundations generally consist of unreinforced concrete piers and walls bearing directly on limestone rock.

The Peace Tower was constructed between 1919 and 1927, with an approximate height of 92 m. It is connected to the Centre Block by a structural link at the first two stories. The Peace Tower is supported by unreinforced concrete piers and walls, with an integrally built outer wythe of stone masonry. The piers bear directly on limestone rock.

## 1.3 Geotechnical Investigation

Based on the preliminary desktop geotechnical study that was completed by Stantec Consulting, August 2014, PWGSC File No.fe173.EP764-150225, the Centre Block is believed to be founded directly on limestone bedrock and we have assumed site classification of A, based on the results of previous nearby tests that are presented in the geotechnical study.

## 1.4 Gap Analysis

Gaps in the information available on the Centre Block that would affect the results of a seismic analysis have been collected and are presented in this report. These gaps have been divided into four categories. *Structure and geometry* identifies areas where thickness or connections cannot be identified. *Material properties* discusses unknown quantities such as material strengths, stiffnesses and densities. *Geotechnical* identifies the required geotechnical information needed to complete a seismic analysis. *Non-structural components* lists areas where more information about architectural, mechanical and electrical features is required to complete a qualitative seismic analysis.

## 1.5 Preliminary Structural Analysis Results

Preliminary structural analysis of the main Centre Block structure indicates the numerous wall, floor and roof components do not have the required capacity to resist either 60 or 100% of the 2010 NBCC seismic loads. The floor and roof diaphragms, and their connections to the walls, have the lowest capacity/demand ratios between 0 and 20% at many critical locations. Almost all of the walls do not have adequate capacity to resist the 2010 NBCC seismic loads, including one fifth of the walls above Level 3 that have less than 30% of the required capacity. The masonry walls typically have sufficient capacity to resist out-of-plane seismic loading, except for a few localized areas around the pavilions.

Preliminary structural analysis of the Peace Tower component of the Centre Block indicates that many of the wall components of the Peace Tower do not have the required capacity to resist either 60 or 100% of the 2010 NBCC earthquake loads. Particular areas of concern are the wall piers on the north face that are defined by the large opening created by the sloped elevator and the spandrel beams immediately above the Memorial Chamber. In addition, the tower structure at the observation deck level, clock face and sloped roof have significantly less than the required capacity to resist either 60 or 100% of the 2010 NBCC earthquake loads.

The reduction in seismic hazard for the Ottawa area anticipated in the 2015 NBCC will improve the capacity/demand ratios for components in both the Centre Block and Peace Tower structures. However, numerous components will still have less than the required capacity to resist 60 or 100% of the 2015 NBCC seismic loads.

## 1.6 Non-structural and Secondary Structural Components

Architectural, mechanical, and electrical components and their connections were observed throughout the Centre Block. The main items that are discussed in this report include suspended ceilings, partition walls, light fixtures, exterior stonework and decorative features, and skylights. In general, these items were not seismically restrained according to the NBCC 2010.

Existing items do not require seismic restraint, but any objects that are moved or replaced during the upcoming renovations will require restraint, such as the mechanical and electrical equipment. Some of the lightweight items will be exempt from restraint requirements according to the building code in effect at the time of construction. If the NBCC 2015 comes into effect prior to construction, then exemptions are expected to change so that fewer items will require restraint.

It is recommended that all items with high historic value be restrained to protect the assets from damage. The existing connections of some of the items may be sufficient to act as restraint, although the capacity of the connections should be checked against seismic loads.

# 2. INTRODUCTION

## 2.1 Scope

In July 2014, Halsall Associates was engaged by PWGSC to complete a preliminary seismic assessment of the Centre Block on Parliament Hill in the context of the requirements of the latest edition of the National Building Code of Canada (2010 NBCC) and the Real Property Service (RPS) Policy on Seismic Resistance of PWGSC Buildings. Halsall was also engaged to complete targeted research on potential options to seismically rehabilitate heritage buildings based on work done on buildings, both nationally and internationally, that are of a similar scale, type and importance as the Centre Block, as well as to present further investigations and specific upgrade options for the Centre Block.

Our services include the following:

a) Research and present potential options for the seismic rehabilitation of heritage buildings based on previous work done on other structures with similar design, importance and earthquake forces, both nationally and internationally.

b) Conduct a gap analysis to determine what information of the building construction, materials and structural systems is missing from the available documentation and research and that would be required to complete a detailed seismic analysis in a separate study.

c) Determine the ability of the Centre Block to resist the seismic loads as specified in 2010 NBCC based on a preliminary analysis of the building structure.

d) Determine the ability of the non-structural and secondary structural elements to resist the seismic loads as specified in the 2010 NBCC based on a qualitative assessment.

e) Compare the proposed 2015 NBCC draft seismic loads to the 2010 NBCC loads and discuss the impact of revised loads on the preliminary seismic assessment.

f) Identify potential seismic upgrade options based on the results of the preliminary seismic analysis.

g) Identify potential opportunities, challenges and risks associated with completing a seismic upgrade of the Centre Block, including the potential lowering of the Centre Block basement and other adjacent construction projects.

This report, for Work Package 2 – Preliminary Seismic Assessment, focusses on presenting the gap analysis, the results of the preliminary seismic assessment of the Centre Block, and the qualitative assessment of the non-structural and secondary structural elements.

## 2.2 Limitations

No party other than the Client shall rely on the Consultant’s work without the express written consent of the Consultant. The scope of work and related responsibilities are defined in the Conditions of Assignment. Any use which a third party makes of this work, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Decisions made or actions taken as a result of our work shall be the responsibility of the parties directly involved in the decisions or actions. Any third party user of this report specifically denies any right to any claims, whether in contract, tort and/or any other cause of action in law, against the Consultant (including Sub-Consultants, their officers, agents and employees).

The work reflects the Consultant's best judgment in light of the information reviewed by them at the time of preparation. Unless otherwise agreed in writing by Halsall, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. This is not a certification of compliance with past or present regulations. No portion of this report may be used as a separate entity; it is written to be read in its entirety.

This work does not wholly eliminate uncertainty regarding the potential for existing or future costs, hazards or losses in connection with a property. No physical or destructive testing and no design calculations have been performed unless specifically recorded. Conditions existing but not recorded were not apparent given the level of study undertaken. Only conditions actually seen during examination of representative samples can be said to have been appraised and comments on the balance of the conditions are assumptions based upon extrapolation. We can perform further investigation on items of concern if so required.

Only the specific information identified has been reviewed. The Consultant is not obligated to identify mistakes or insufficiencies in the information obtained from the various sources or to verify the accuracy of the information.

Halsall is not investigating or providing advice about pollutants, contaminants or hazardous materials.

## 2.3 Existing Documents

The following existing documents were provided by PWGSC and were used:

- A variety of original architectural and structural drawings prepared by architects John Pearson and Jean-Omer Marchand, dated from 1916 to 1927;
- Original structural steel floor plan shop drawings prepared by the Dominion Bridge Co., dated to 1916;
- Centre Block "As-Found" drawings, prepared by the Heritage Conservation Directorate, Professional and Technical Service Management and PWGSC in 2002;
- Various Centre Block and Peace Tower alteration drawings, including:
    - Centre Block Underground Services Building
    - Centre Block Chimney Stabilization Phase I
    - Alterations & Additions Centre Block (1971 Courtyard Additions)
    - Fullers Gargoyle repair
    - Extension to East & West Penthouses at South Corridor Elevators
    - Centre Block South – Conservation

  - Centre Block Phase 1 Renovations (1987 Stairwell extensions)
  - Centre Block Ventilation Towers Rehabilitation Project
  - Peace Tower Alteration Parliament Hill (1980)
  - Peace Tower Conservation of Masonry (1994)

- Scans of selected specifications, construction reports and letters prepared by John Pearson during the original construction;
- Photographs from the original construction period and several more recent restoration and repair projects; and,
- Various Centre Block and Peace Tower studies and reports.

### 2.4 General Description of the Structure and Site

#### 2.4.1 Centre Block Main Building

Construction of the Centre Block main building structure was carried out from 1916 to 1920, following the fire that destroyed much of the old Centre Block. The Centre Block is comprised of six above grade storeys and one below grade storey. The building is comprised primarily of two main east-west office corridors, called the North and South Corridors, which are connected at varying levels by five main volumes which are separated by internal courtyards: the House of Commons, Senate Chamber, East and West Office Blocks and the Hall of Honour (formerly Hall of Fame).

The floor structure of the Centre Block is typically comprised of flat terra cotta arches supported by structural steel beams, covered with a cementitious topping. Lateral spread of the beams due to the arching forces is prevented by steel tie rods between the beams. The two exceptions are the lowest framed level, which consists of reinforced concrete slabs and beams, and the sloped roof sections, which consist of a cementitious product placed over expanded metal forms supported on steel beams or steel trusses.

The floor structures are typically supported on load bearing masonry walls, except for some larger volumes which are supported on steel columns. The exterior masonry walls are primarily comprised of brick masonry built integrally with an outer wythe of stone masonry, while the interior load bearing masonry walls are comprised of only brick masonry. The foundations which support the walls and columns typically consist of unreinforced concrete piers and walls that bear directly on the rock. One exception occurs in the Southeast corner of the building, where the unreinforced concrete walls were constructed on top of the existing limestone foundation walls from the original Centre Block construction, rather than directly on the bedrock.

The Centre Block is connected to the Library of Parliament structure, but the Library is not specifically considered in this report.

Based on the preliminary desktop geotechnical study that was completed by Stantec Consulting, August 2014, PWGSC File No.fe173.EP764-150225, the Centre Block is believed to be founded directly on limestone bedrock.

### 2.4.2 Peace Tower

The Peace Tower was constructed between 1919 and 1927, with an approximate height of 92 m. It is a component of the overall Centre Block building and is linked to the main structure at the first two stories. The Peace Tower is supported primarily by 4 unreinforced concrete piers in the corners of the tower that are constructed integrally with an outer wythe of stone masonry. The piers are connected by unreinforced concrete walls with a similar outer wythe of stone masonry that are punctuated by numerous openings. The piers are supported on unreinforced concrete foundations that bear directly on rock.

The floor structure of the Peace Tower typically consists of reinforced concrete slabs supported on structural steel beams encased in concrete, which bear on the concrete piers and walls. The sloped roof of the tower is comprised of reinforced concrete slabs, beams and piers.

Based on the preliminary desktop geotechnical study that was completed by Stantec Consulting, August 2014, PWGSC File No.fe173.EP764-150225, the Peace Tower is believed to be founded directly on limestone bedrock.

## 2.5 Geotechnical Investigation

Based on the preliminary desktop geotechnical study that was completed by Stantec Consulting, Project 122411046, the Centre Block is believed to be founded directly on limestone bedrock and we have assumed site classification of A, based on the results of previous nearby tests that are presented in the geotechnical study.

## 3. DOCUMENT REVIEW AND METHODOLOGY

Before beginning the seismic analysis, the existing Centre Block drawings and specifications, historical information and reports that had been prepared on the Centre Block, such as material tests and previous structural studies, were reviewed. The list of documents that were reviewed is given in Section 2.3.

Following the documentation review, three site visits were made to the Centre Block to conduct a visual review of the buildings heritage features and structural components where accessible. Additional meetings were held with Public Works' staff to understand valuable knowledge that has been learned during previous research or work that was done on the building.

In order to complete the preliminary seismic analysis, the first step was a careful consideration of the load paths of the seismic force resisting system of the existing building. A three dimensional computer model was built based on the available existing information and where information was unavailable, reasonable assumptions were made. Meetings were also held with Public Works' staff to discuss the modelling procedure and the assumptions made. Comments were addressed and incorporated into the results.

The structures were then analysed using both computer analysis and hand calculations and a sensitivity analysis was performed in regards to unavailable information. Detailed information of the seismic analysis procedure, as well as the gap analysis of missing information, is discussed in detail in the following chapters.

# 4. GAP ANALYSIS

In the process of reviewing the documentation that was provided by PWGSC and performing the seismic analysis of the Centre Block and Peace Tower, gaps were noted in the information available that would affect the results of the seismic analysis. For the purposes of the preliminary seismic assessment, where relevant information was missing or unavailable, reasonable assumptions have been made and sensitivity analyses performed as required. Each of these factors is discussed in detail in this report. Work Package 3 contains recommendations for further investigation, testing and analyses to verify the assumptions made, and to increase the accuracy of the preliminary seismic analysis.

The Centre Block and Peace Tower are components of the same building and were built around the same time period but different information on their structural systems and properties are available. For this reason, the summary of missing information has been separated for the two structural components, as well as sorted in four categories: Structure & geometry, Material properties, Geotechnical, and Non-structural components. Relevant gaps in information are listed below.

## 4.1 Structure & geometry

Gaps in information on the Centre Block component include:

a) Thickness and composition of the topping covering the terra cotta arch floors, as well as details of any connection to the steel beams. The topping is cinder concrete fill or Nail-a-Crete, the thickness of which is dependent on the depth of the supporting steel beams, the type of floor finish and the elevation of the tops of the terra cotta arches relative to the tops of the steel beams. Although this information affects the total mass of the structure and therefore the response of the structure in a seismic event, this effect is minimal. The primary importance of this information is that the floor composition will affect the requirements for any diaphragm strengthening required for a seismic upgrade.

b) Connections between structural steel elements, such as collector beams for diaphragms. Information on these connections is restricted to photographs from the time of construction and typical information obtained from historic material catalogues. These connections are important parts of the seismic force resisting system, as they allow the transfer of lateral loads from the diaphragms to the walls.

c) Steel to masonry and concrete to masonry connections. This includes the typical embedment of steel floor beams embedded in supporting masonry walls, steel elements entirely embedded in masonry, and concrete slabs supported on or abutting masonry walls. These connections are important parts of the seismic force resisting system, as they allow the transfer of lateral loads from the diaphragms to the walls.

d) Connection details between the Centre Block and the Library of Parliament. There is limited information this connection, which may be an expansion gap whose capacity to accommodate the deflections of the two structures will need to be determined. Alternatively, there may be a positive connection at the link between Centre Block and the Library, in which case the possibility for extensive cracking at the link exists. This would affect heritage features such as flooring, ornamental stone, and suspended ceilings.

e) Connection details between the main Centre Block structure and the Peace Tower. Detailed information on the link between Centre Block and the Peace Tower is mostly limited to construction photographs, which do not show all elements. The positive connection between the two structures means that the possibility for extensive cracking at the link during a seismic event exists. This would affect heritage features such as flooring, ornamental stone, and suspended ceilings.

f) Limited information and drawings of the four North towers. Few drawings are available to provide detailed information, and what is available is sometimes contradictory. Although additional information on these towers will have minimal effect on the total mass and response of Centre Block, construction details are needed to determine the capacity of the tower components.

g) Limited information on the various roof structures of the Centre Block. The drawings showing sections of the roofs are either poor quality scans or are not sufficiently detailed to draw conclusions on the assemblies. Although this information affects the total mass of the structure and therefore the response of the structure in a seismic event, this effect is minimal. The primary importance of this information is that the roof composition will affect the requirements for any strengthening required for a seismic upgrade.

h) Limited information on the four smaller porte cocheres and their foundations on the South, West and East sides of the building. Most of the original structural drawings do not show the porte cocheres, indicating that that they were perhaps a later addition to the structure. Knowledge of these structures will be important to determining their capacity to resist seismic loads.

For the Peace Tower component, gaps in information include:

a) Accurate wall thicknesses at most levels, and proportion of stone masonry to unreinforced concrete thickness in these walls and piers. These dimensions are important because they affect the mass and stiffness of the seismic force resisting system, and therefore the response of the structure in a seismic event.

b) Elevation of transition from composite concrete and stone walls to stone masonry walls. Near the top of the Peace tower, the exterior wall transitions from a composite wall with exterior and interior wythes of stone and a concrete core to simply stone masonry walls. This transition affects the stiffness and strength of the tower and therefore its response in a seismic event, although this effect will be minimal.

c) Connection details between the various floor structures and the exterior walls, as well as the type of floor construction at each level of the Peace Tower. The floors are typically concrete slabs supported by concrete encased steel beams, however there is limited information on the reinforcing of the floors and how the steel floor beams are connected to the walls. Although this information affects the total mass of the structure and therefore the response of the structure in a seismic event, this effect is minimal. The primary importance of this information is that the floor composition will affect the requirements for any diaphragm strengthening required for a seismic upgrade.

d) Structural elements spanning openings in the exterior walls. Limited or conflicting information is available on the lintels above large openings such as the clock faces and carillon openings. The lintels act as spandrel beams between the tall piers of the Peace Tower and thus are primary components of the seismic force resisting system. Their capacity and connections are consequently very important.

e) Construction details on the connection of the structure of the sloped concrete roof to the walls around the Clock Chamber. At this level, the structure transitions from reinforced concrete walls to unreinforced stone masonry walls, and limited information about this connection is available. This is critical component of the seismic force resisting system load path.

## 4.2 Material properties

Gaps in information on the Centre Block and the Peace Tower components include:

a) Density of exterior stone masonry. Because Centre Block was built over the course of several years and there is an inherent variability in the properties of stone, the mass of the stone masonry is uncertain. This may have a significant effect on the response of the structures.

b) Brick and stone masonry assembly mechanical properties, including: compressive strength, modulus of elasticity, shear strength, flexural tensile strength, elastic damping ratio and the ratio of the modulus of elasticity to the shear modulus. In some instances, mechanical properties of individual components are known but the properties of the whole assembly have not been tested. The properties used in modelling the structures outlined in this report have been assumed based on previous research, however the assemblies used in the Centre Block and the Peace Tower are unique and their stiffness and strength will have a significant effect on the response and strength of the structures.

c) Mechanical properties of the concrete and other cementitious products used throughout the building floor structures, such as “Nail-a-crete” and “Flex-or-crete”. These properties are required to determine the strength of the diaphragms.

d) Strength information of the terra cotta tile and mortar flat arch floors. This information is important because it can affect the strength of the diaphragm.

## 4.3 Geotechnical

A preliminary desktop geotechnical study was provided with assumed geotechnical parameters that were used in the preliminary seismic analysis in this report. However, for a complete seismic assessment, a final geotechnical investigation is required to confirm the geotechnical assumptions made for the seismic analysis of the Centre Block and Peace Tower and to obtain additional information for certain seismic upgrade options. The required geotechnical properties are:

a) Vs30

b) Dynamic soil pressure on basement walls

c) Coefficient of friction at underside of footings (static, sliding)

d) Bearing capacity

e) Site class

### 4.4 Non-structural components

Gaps in information on the Centre Block and Peace Tower non-structural components include:

a) Connection and construction details of arched stone masonry and ornamental stone masonry elements throughout the building. There is limited information on the specific connection details for stonework colonettes, turrets, pinnacles and other ornamental stonework. The original construction specification notes that stone is to be connected to other stone with brass dowels, stone dowels or wrought iron anchors, but it is not clear if this applies to all ornamental stone, interior and exterior. Original Architectural drawings only show dowels in a few, discrete locations. These details are an important part of determining the capacity of these heritage features to resist a seismic event.

b) Details of connections of copper roofing, spires, and lightning rods. There is limited information on the connections of spires and pieces of roofing. Connections of the roofing were observed in some locations; however, the connections of other items will also impact the capacity of the roofing connections to resist seismic loads.

c) Connection details for objects (lights, cameras, fire cabinets) that are connected through the stone masonry walls above the main roof level. The size of these connections should be noted, as well as the existing state of them, as some of the connectors are exposed and were observed to be rusting.

d) Details on the ornamental suspended ceilings throughout the Centre Block and the Peace Tower. These ceilings are typically composed of concrete and plaster applied to ribbed expanded metal and suspended from the floors above with steel tee and angle framing members. Limited information is available on these elements and their connections to the floor structure, including the thickness of the cementitious layer. These details, including the weight of the ceilings, are an important part of determining the capacity of these heritage features to resist a seismic event.

e) Details of the connection of the terra cotta tile partition walls to the floor structure. This information is important because it is important to determine how the walls are able to resist out of plane loads during a seismic event.

f) Details of the connections and weights of the suspended lights. This information was limited, and will determine whether or not a light will require seismic restraint, and the capacity of the connection to resist seismic loads.

g) Connections of sculptures and details of connections. No information was noted on the drawings about the sculptures and their installation methods other than their location. Connection details are required to determine the capacity of the sculptures to resist seismic loads.

h) Weights of mechanical and electrical objects. It is understood that all mechanical and electrical systems will be replaced during the renovations; however, if items such as the newer communications and security systems are left in place, then some items may require restraint. In this case, the weight of the objects will govern the requirement for restraint and restraint details.

# 5. SEISMIC ASSESSMENT

## 5.1 Seismic Hazard

Earthquakes usually cause damage to buildings due to any of the following:

- Ground shaking.
- Soil liquefaction and landslides.
- Surface fault ruptures.
- Tsunamis.

Only ground shaking is directly addressed by the 2010 NBCC. Potential for soil liquefaction is addressed by the Geotechnical Report.

Objectives of the 2010 NBCC, with respect to earthquake resistant design are:

- To protect the life and safety of building occupants and the general public as the building responds to strong ground shaking.
- To limit building damage during low to moderate levels of ground shaking.
- To ensure that post-disaster buildings can continue to be occupied and function following strong ground shaking, though minimal damage can be expected in such buildings.

According to the 2010 NBCC, strong ground motion is defined as having a probability of exceedance of 2% in 50 years at the median confidence level. This corresponds to a .04% annual probability of exceedance.

Although stronger ground shaking than this could occur, it would be economically impractical to design for such rare ground motions. Therefore, a ground motion having a probability of exceedance of 2% in 50 years is termed as the maximum earthquake ground motion to be considered. More simply, it is termed as the design ground motion (DGM).

## 5.2 Design Ground Motion

The design ground motion for a structure is expressed in the 2010 NBCC as a base acceleration. The base acceleration value is a function of the specific natural period of vibration of the structure. The 5% Damped Spectral Response Acceleration values for Ottawa, Ontario for natural periods of vibration of 0.2, 0.5, 1.0 and 2.0 seconds are shown in the table below. Peak Ground Acceleration (PGA) is also included.

**Table 5.1: 5% Damped Spectral Response Acceleration Values (Ottawa, Ontario)**

| 2010 NBCC - Values for 2% Probability Exceedance in 50 Years | | | | |
|---|---|---|---|---|
| PGA | $S_a(0.2)$ | $S_a(0.5)$ | $S_a(1.0)$ | $S_a(2.0)$ |
| 0.32 | 0.64 | 0.31 | 0.14 | 0.046 |

Note: All values are in decimal percentages of g (acceleration due to gravity).

The 2010 NBCC uses site coefficients $F_a$ and $F_v$ to modify the above spectral values to account for the specific site soil conditions. Based on the desktop geotechnical study carried out by Stantec Consulting, the Site Classification for seismic site response has been assumed to be Site Class A, with an approximate shear wave velocity of 2000 m/s. The structural analysis has been based on this Site Classification and shear wave velocity.

The acceleration-based site coefficient and velocity-based site coefficient are calculated to be $F_a$=0.655 and $F_v$=0.433, respectively. The resulting design spectral response acceleration values for Ottawa, Site Class A, $V_s$ = 2000 m/s, for periods of natural vibration of 0.2. 0.5, 1.0, 2.0, and 4.0 seconds are given below:

**Table 5.2: Design Spectral Response Acceleration Values (Ottawa, Ontario, Site Class A, Vs = 2000 m/s)**

| 2010 NBCC -Values for 2% Probability Exceedance in 50 Years | | | | |
|---|---|---|---|---|
| S(0.2) | S(0.5) | S(1.0) | S(2.0) | S(4.0) |
| 0.42 | 0.13 | 0.061 | 0.020 | 0.010 |

Note: All values are in decimal percentages of g (acceleration due to gravity).

# 6. CENTRE BLOCK PRELIMINARY SEISMIC ANALYSIS RESULTS

## 6.1 Centre Block Seismic Force Resisting System

The Centre Block seismic force resisting system (SFRS) consists of brick/stone masonry walls above grade and plain unreinforced concrete walls below grade. The floor and roof diaphragms consist of terra cotta tile flat arches infilled between supporting steel beams with a weak cementitious topping. The diaphragms are likely to exhibit flexible behaviour as the floors are discontinuous at the supporting walls. Inertial loads from individual floor areas will be carried only by the walls immediately adjacent to them.

## 6.2 Centre Block Structural Modelling

A 3D finite element analysis model of the Centre Block was created using ETABS version 13.2.1. Figure 6.1 displays the 3D computational model. The model was used to determine the building's fundamental period, to perform an equivalent static and dynamic analysis, and to test the sensitivity of the analysis results to the definition of the material properties.

The walls and floors were modelled using finite element shell objects. The stiffness of the floor elements was reduced to simulate a flexible diaphragm load distribution. Beams and columns were modelled with frame elements.

Reduced element stiffness resulting from cracking of the concrete and masonry sections was incorporated through modification of the element stiffness modifiers. As explained further in Section 6.3 and Section 6.4, an estimate of the potential range of the Centre Block's fundamental period was evaluated using upper and lower bound material properties. The upper bound material properties assumed an un-cracked wall condition and did not include a stiffness modifier. The lower bound material properties assumed a cracked wall condition and included a stiffness modifier.

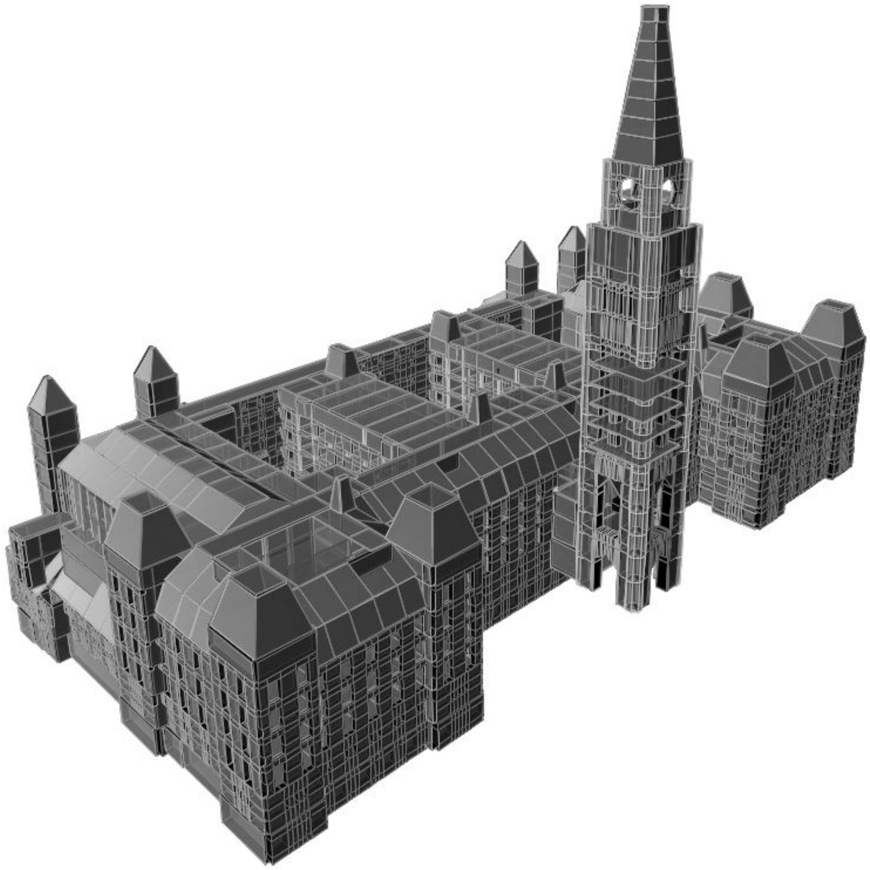

Figure 6.1 Centre Block ETABS Model

## 6.3 Centre Block Material Properties

### 6.3.1 Masonry Walls

The Centre Block perimeter and courtyard walls are composed of an exterior wythe of snecked sandstone masonry laid in random level beds with a multi-wythe common clay brick backing (see Figure 6.2). Interior load bearing walls are composed of clay brick masonry. Chemical analysis of the mortar has shown it to be a hard, Portland cement based mortar.

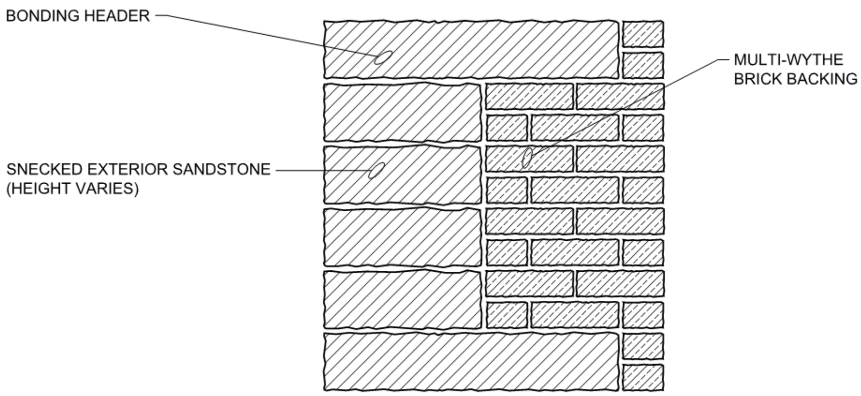


**Figure 6.2 Typical Centre Block Exterior Masonry Wall Section**

Previous studies of localized Centre Block areas and neighbouring structures have reported material property values for the individual wall components based on material sample testing. The key studies includes:

- Dynamic Analysis of the Centre Block Ventilation Towers, KIB Consultants Inc., May 2012.
- Report on Condition of Interior Brickwork Centre Block Parliament Hill West Courtyard Parapets No. 4 and No 10a, J.L. Richards & Associates Limited, June 2009.
- Testing of Walls Representative of those on Parliament Hill, Shrive, Parsekian, Sorour, University of Calgary, August 2008.
- Seismic Evaluation of the MacKenzie Tower as a Basis for Evaluation of the Parliament Buildings, NRC Report A-8006.2, June 1995.

The material property ranges presented in the above studies are summarized below:

**Table 6.1 Material Property Ranges for Masonry Walls**

| | Compressive Strength | | | Elastic Modulus | | |
|---|---|---|---|---|---|---|
| | Min | Max | Average | Min | Max | Average |
| Clay Brick Units | 26 MPa | 61 MPa | 40 MPa | - | - | - |
| Nepean Sandstone Units | 80 MPa | 150 MPa | 110 MPa | 59 GPa | 67 GPa | 62 GPa |
| Mortar | 22 MPa | 28 MPa | 24 MPa | - | - | - |
| Combined Wall Assemblage | - | - | - | - | - | - |

As indicated in the Table 6.1, although some material testing has been performed on the individual wall components, no full scale testing on the overall hybrid stone/brick wall assemblage has been performed. It can also be seen that both the clay brick and stone units exhibit a considerable range of potential material property values.

To evaluate the sensitivity of the seismic analysis to the wide range of potential material strengths, upper bound and lower bound values for the compressive strengths and Elastic Modulus have been determined.

### 6.3.2 Masonry Wall Elastic Modulus - Lower Bound Estimate

The relationship between clay brick unit strength and clay brick prism strength is well documented in material design codes and research literature. The relationship between clay masonry prism strength ($f'_m$) and clay masonry prism stiffness (E) is also reasonably well documented, although there is a wide variation between codes of different jurisdictions. Unfortunately, there is little corresponding information available to relate stone unit compression strength to either stone prism strength or prism stiffness.

In the absence of any specific stone prism test data, a reasonable lower bound estimate of the stiffness of the stone portion of the wall assemblies may be made by equating the stone portion to a similar high strength clay brick and assuming that it follows the same relationship between unit strength and prism strength. In reality, a stone prism made up of large individual stone units would likely be stiffer than a brick prism with equally strong but smaller units. As a result, the assumption will produce a conservative lower bound estimate of stiffness. The overall stiffness of the composite wall section can then be calculated based on a weighted average of the stone and brick components.

It should also be noted that material design codes intended for use in the design of new structures typically use 5 percentile, lower bound characteristic material strength values, to ensure that new designs have a reliable margin of safety against localized variations in material strength. The equations and tables presented in material design codes that define the relationships between clay masonry prism strength and stiffness are calibrated based on this assumption.

For existing structures, a lower bound characteristic material strength is typically unknown. Material sample testing, which is usually of a limited sample size, produces an indication of average material strength only. The use of an incompatible average material strength value in the typical material code equations would produce an unconservative lower bound estimate. Subsequently, relationships taken from literature that consistently correlate average unit strength to average prism strength and average prism stiffness should be used to calculate stiffness. Figure 4.11 in Drysdale and Hamid (2005)[1] can be used to determine an average prism strength from an average unit strength value. Test data also cited in Drysdale and Hamid (2005) gives an average relationship between the elastic modulus and average clay brick prism strength of approximately 420 x $f'_m$.

As estimate of the lower bound elastic modulus of the combined stone brick is calculated as follows:

[1] *Drysdale and, R.G and Hamid, A.A (2005). Masonry structures: Behaviour and Design. Canadian Masonry Design Centre. Mississauga. Ontario.*

| | | |
|---|---|---|
| Compressive Strength of Clay Brick Unit | = 40 MPa | |
| Compressive Strength of Brick Prism | = 16.5 MPa | |
| Elastic Modulus of Brick Prism | = 420 x 16.5 MPa | = 6.9 GPa |
| Compressive Strength of Stone Unit | = 110 MPa | |
| Compressive Strength of Stone Prism | = 38 MPa | |
| Elastic Modulus of Stone | = 420 x 38 MPa | = 16 GPa |

For a typical Centre Block exterior wall that consists of approximately one third stone and two third clay brick, the elastic modulus of the composite wall section based on the weighted average of the two components:

$E_{\text{Brick and Stone (Lower Bound)}}$ = 2/3 x (6.9 GPa) + 1/3 x (16 GPa) = 9.9 GPa

### 6.3.3 Masonry Wall Elastic Modulus – Upper Bound Estimate

An upper bound estimate of the combined stone/brick/mortar wall assemblage may be calculated using the fundamental mechanics of composite materials approach noted in the "Guidelines for the Seismic Assessment of Stone Masonry Structures" by PWGSC (2000).

$$E_{Stone-Masonry} = \frac{1}{\dfrac{\delta}{E_s} + \dfrac{(1-\delta)}{E_j}}$$

where:

$$\delta = \frac{t_s}{t_s + t_j}$$

$t_s$ = height of stone unit

$t_j$ = height of masonry joint

$E_s$ = Elastic modulus of stone unit

$E_j$ = Elastic modulus of mortar joint

The average height of a stone unit ($t_s$) can be taken as 7.5 inches (as noted in the Centre Block Structural History Report, 2014). The average height of the mortar joints ($t_j$) is taken as 0.5 inches. The Elastic modulus of the Nepean Sandstone is known from testing to be approximately 62 GPa. The elastic modulus of the mortar can be approximated from the material test compression data using the Elastic modulus equation in ASCE 5-13 Section 4.2.2.4 for grout. For the purposes of calculating an upper bound stiffness value, this assumption will produce a conservative upper bound estimate of stiffness.

The Elastic modulus of the stone masonry is therefore calculated as follows:

$$E_j = 500\ f_j^{'} = 500 \times 24 \quad MPa = 12 \quad GPa$$

where:

$f_j^{'}$ = compressive strength of mortar

$$\delta = \frac{t_s}{t_s + t_j} = \frac{7.5}{7.5 + 0.5} = 0.94$$

$$E_{Stone-Masonry} = \frac{1}{\frac{\delta}{E_s} + \frac{(1-\delta)}{E_j}} = \frac{1}{\frac{0.94}{62} + \frac{(1-0.94)}{12}} = 50 \quad GPa$$

As per the lower bound elastic modulus calculation, the upper bound elastic modulus of the composite wall section can be calculated from the weighted average of the two components:

$E_{\text{Brick and Stone (Upper Bound)}}$ = 2/3 x (6.9 GPa) + 1/3 x (50 GPa) = 21 GPa

### 6.3.4 Masonry Wall Shear Modulus

The masonry wall shear modulus (G) is assumed to equal to 40% of the Elastic modulus as recommended by ASCE 5-13. Upper and lower bound values corresponding to the upper and lower bound Elastic Modulus values have been used.

### 6.3.5 Masonry Wall Compressive Strength – Lower Bound Estimate

A lower bound estimate of the compressive strength of the composite stone/brick masonry wall can be determined from Table 3, CSA S304.1-04 and by assuming that the strength is predominantly controlled by the strength of the brick component. From Table 3, CSA S304.1-04:

$f_{\text{brick unit}}$ = 26 MPa => $f'_m$ = 9 MPa

### 6.3.6 Masonry Wall Compressive Strength – Upper Bound Estimate

As per the lower bound estimate, a conservative estimate of the upper bound stone/brick masonry compressive strength can be determined by assuming that the strength is predominantly controlled by the strength of the weaker brick component. Using Figure 4.11 in Drysdale and Hamid (2005).

$f_{\text{brick unit}}$ = 61 MPa => $f'_m$ = 26 MPa

### 6.3.7 Unreinforced Concrete Basement Walls

The compressive strength of the concrete basement walls is currently unknown. Some limited material testing has been performed on the concrete corner columns of the Peace Tower (refer to Section 7.3) which indicated a compressive strength of approximately 40MPa. For the purpose of assessing the sensitivity of the analysis results to the selection of concrete material strength, an upper strength of 40 MPa, and a lower bound strength of 25 MPa was adopted.

### 6.3.8 Terra-cotta tile flat arch topping

The Centre Block Structural History Report (2014) notes that a patented product called Nail-a-crete was used to form the lightweight cementitious topping over the terra cotta arch floors. The 1919 American patent application for Nail-a-crete states a compressive strength range of 1.4 MPa to 17MPa. These have been adopted as upper and lower bound material properties.

### 6.3.9 Summary of Material Property Values

A summary of the composite stone/brick exterior masonry wall assemblage strength is presented below:

Table 6.2: Exterior Masonry Wall Assemblage Compressive Strength and Stiffness Values

| | Lower Bound | Upper Bound |
|---|---|---|
| Masonry Wall<br>Compressive Strength ($f'_m$) | 9 MPa | 26 MPa |
| Masonry Wall<br>Elastic Modulus (E) | 9.9 GPa | 21 GPa |
| Plain Concrete Basement Wall<br>Compressive Strength | 25 MPa | 40 MPa |
| Terra cotta tile floor topping<br>Compressive Strength | 1.4 MPa | 17 MPa |

## 6.4 Centre Block Fundamental Period

Fundamental Period - 2010 NBCC Empirical Formula

The 2010 NBCC empirical formula for determination of a shear wall structure's fundamental period is evaluated for the Centre Block as follows:

$T = 0.05\ (h_n)^{0.75} = 0.05 \times 24^{0.75} = 0.54$ seconds

Fundamental Period - Modal Analysis

The fundamental periods of the Centre Block in the East-West and North-South building directions were evaluated from a modal analysis of the computation model. Fundamental periods corresponding to the assignment of both the potential lower bound and upper bound material properties were calculated and are presented in Table 6.3. Stiffness modifiers to simulate the effect of masonry wall cracking were incorporated in the lower bound material analysis.

Table 6.3: Centre Block Fundamental Periods – Modal Analysis

| | Lower Bound Material Properties | Upper Bound Material Properties |
|---|---|---|
| East-West Building Direction | 0.20 sec | < 0.20 sec |
| North-South Building Direction | 0.25 sec | < 0.20 sec |

It can be seen from Table 6.3 that the fundamental periods determined from modal analysis of the Centre Block computation model are lower than the fundamental period value determined using the 2010 NBCC empirical formula. This is not unexpected, as the 2010 NBCC empirical formula is intended for use in the new design of modern shear wall buildings, which would typically have significantly less walls than the Centre Block structure.

## 6.5 Centre Block Equivalent Static Base Shear

### 6.5.1 Calculation of Equivalent Static Base Shear

The 2010 NBCC formula for calculation of the Equivalent Static base shear is:

$$V = \frac{S(T_a)\, M_v\, I_E}{R_d\, R_o} W$$

Where:

$T_a$ = fundamental lateral period of vibration. (See Table 6.3.)

$S(T_a)$ = the design spectral response acceleration, expressed as a ratio to gravitational acceleration for a period of $T_a$, the fundamental lateral period of vibration. (See Table 5.2)

$M_v$ = factor to account for higher mode effect on base shear (= 1.0 for $T_a$ < 1.0 sec)

$I_e$ = Importance Factor for Earthquake Loads (= 1.0 for Normal Importance)

$R_d$ = the ductility-related force modification factor reflecting the capability of a structure to dissipate energy through inelastic behaviour (= 1.0 for unreinforced masonry)

$R_o$ = the over-strength-related force modification factor accounting for the dependable portion of reserve strength (= 1.0 for unreinforced masonry)

W = the weight of the building = 930 000 kN

Note: The maximum equivalent static base shear for an unreinforced masonry structure is the base shear corresponding to a fundamental period of 0.2 seconds.

The equivalent static base shears for the fundamental periods noted in Table 6.3, are evaluated as follows:

<u>T ≤ 0.20 seconds</u>

S(0.2) = 0.42 (See Table 5.2)

$$V = V_{Max} = \frac{S(T_a)\; I_E}{R_d\; R_o} W = \frac{0.42 \times I_E}{1.0 \times 1.0} W = 0.42\; W$$

<u>T = 0.25 seconds</u>

S(0.25) = 0.42 – (0.25-0.20)/(0.50-0.20) x (0.42-0.13) = 0.37 (Linear interpolation of Table 5.2)

$$V = \frac{S(T_a)\, M_v\; I_E}{R_d\; R_o} W = \frac{0.37 \times 1.0 \times 1.0}{1.0 \times 1.0} W = 0.37\; W$$

The fundamental periods and equivalent static base shears are summarized in Table 6.4 below.

Table 6.4: Centre Block Equivalent Static Base Shear

| **East - West Building Direction** | | | |
|---|---|---|---|
| | **Lower Bound Material Properties** | **Upper Bound Material Properties** | **Median Variance** |
| Fundamental Period | 0.20 sec | < 0.20 sec | |
| Equivalent Static Base Shear | $V = V_{max} = 0.42\ W = 390\ 000$ kN | $V = V_{max} = 0.42\ W = 390\ 000$ kN | ± 0 % |
| **North - South Building Direction** | | | |
| | **Lower Bound Material Properties** | **Upper Bound Material Properties** | **Median Variance** |
| Fundamental Period | 0.25 sec | < 0.20 sec | |
| Equivalent Static Base Shear | $V = 0.37\ W = 344\ 000$ kN | $V = V_{max} = 0.42\ W = 390\ 000$ kN | ± 6 % |

### 6.5.2 Sensitivity of Equivalent Static Base Shear to Material Property Selection

It can be seen from Table 6.4 that despite the relatively large range of potential Elastic Modulus values for the masonry walls, the potential range in equivalent static base shear for the Centre Block building is small due to the inherent stiffness of the structure. Even when the lower bound material values are assigned, the structure's fundamental period is short enough that it is at, or close to, the maximum code required equivalent static base shear. The equivalent static base shear is therefore insensitive to the choice of material properties.

## 6.6 Centre Block Dynamic Analysis

### 6.6.1 Calculation of Dynamic Base Shear

A linear dynamic response spectrum analysis was performed in accordance with the requirements of 2010 NBCC. Thirty modes in both the East-West and North-South directions were evaluated. The summation of the participating modal mass was 95% in both cases. The dynamic base shears in each of the primary building directions are summarized below:

Table 6.5: Centre Block Dynamic Base Shear

| East - West Building Direction | | | |
|---|---|---|---|
| | Lower Bound Material Properties | Upper Bound Material Properties | Median Variance |
| Dynamic Base Shear | $V_d$ = 444 000 kN | $V_d$ = 446 000 kN | ± 0.2 % |
| **North - South Building Direction** | | | |
| | Lower Bound Material Properties | Upper Bound Material Properties | Median Variance |
| Dynamic Base Shear | $V_d$ = 424 000 kN | $V_d$ = 445 000 kN | ± 3 % |

6.6.2 Sensitivity of Dynamic Analysis Base Shear to Material Property Selection

Similar to the bases shears calculated using the Equivalent Static Analysis, the base shears determined from a dynamic analysis are also relatively insensitive to the choice of material property due to the structure primarily responding in the short period range.

## 6.7 Centre Block Seismic Capacity Evaluation

6.7.1 Masonry Walls – In-plane Capacity/Demand Evaluation

The in-plane seismic capacities of the principle Centre Block wall lines have been evaluated. The majority of the wall lines have a significant amount of openings and localized failure of the masonry piers (vertical wall segments between the openings) controls the overall capacity of each wall line. In-plane failure of the individual pier elements is governed by one of several potential modes of failure as illustrated in Figure 6.3 and outlined below:

a) Rocking

This failure mode is characteristic of masonry piers with low axial compression loads and large overturning moments. This condition leads to tension-controlled cracking normal to the bed joints, followed by overturning or rocking. The capacity of a pier to resist rocking was evaluated using the NRC Guidelines for Seismic Evaluation of Existing Buildings (December 1992):

$V_r = P_D \cdot (0.9 \cdot D) / H$

where:

$P_D$ = Axial compressive dead load on pier

D = in-plane length of pier

H = height of pier

b) Crushing

This failure mode occurs when the combined axial stresses in a wall due to gravity and lateral loads exceeds the compressive strength of the masonry. From CSA S304.1-04, the maximum compressive strength of masonry is $\varphi_m \cdot f'_m$.

c) Shear (Diagonal Tension Cracking)

This failure mechanism is characteristic of loading cases with high shear and high axial loads. The in-plane shear capacity was evaluated using clause 7.10.1.1 of CSA S304.1-04.

d) Sliding Shear

This failure mode occurs when the shear force exceeds the sliding resistance of the wall along a bed joint. The in-plane sliding shear capacity was evaluated using clause 7.10.4.1 of CSA S304.1-04.

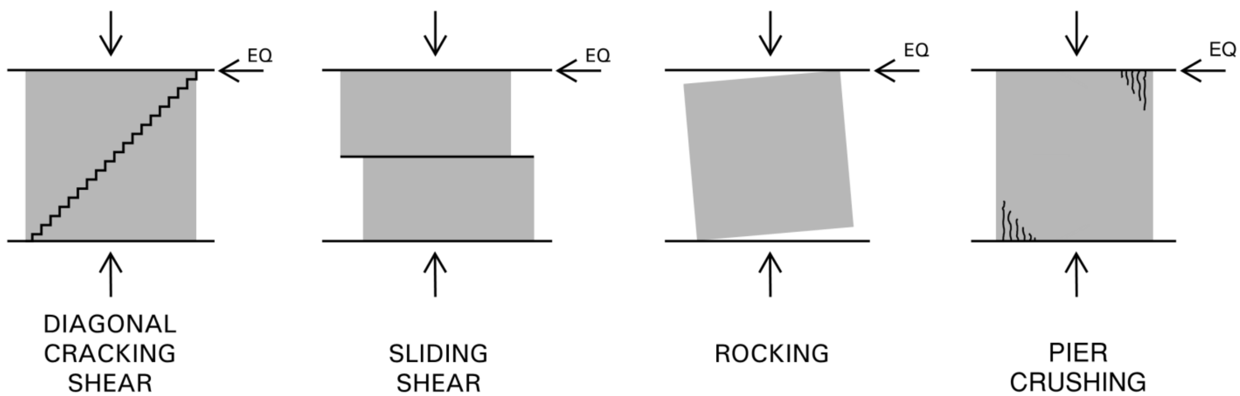


**Figure 6.3: Masonry Pier Potential Modes of Failure**

Based on the masonry pier failure mode method of analysis described above, the ultimate capacity of the principle wall lines of the Centre Block in the East-West and North-South directions were evaluated. The capacity/demand ratios are presented in Figure 6.4 and Figure 6.5 below:


Figure 6.4: L3 to Roof (Combined) – Wall Capacity/Demand Ratios (2010 NBCC)


Figure 6.5: L1 and L2 (Combined) – Wall Capacity/Demand Ratios (2010 NBCC)

From Figure 6.4, it can be seen that none of the masonry walls above Level 3 have sufficient capacity to resist full 2010 NBCC seismic loads. A range of capacity/demand ratios is evident with approximately 20% of the walls possessing less than 30% of the capacity required to resist 2010 NBCC seismic loads.

From Figure 6.5, it can be seen that, in general, the wall capacity/demand ratios improve in the lower levels of the structure. This is caused by the increasing quantity of axial load in the walls which typically improves the lateral load resistance of the walls. The exception is the East-West south corridor wall line that has a decrease in it capacity/demand ratio due to a significant increase in the quantity of openings at the lower levels.

A discussion of upgrade options to resist the 2010 NBCC seismic loads will be presented in the Work Package 3 Supplement Seismic Report.

### 6.7.2 Sensitivity of Masonry Wall Capacity/Demand Ratio to Material Property Selection

The masonry wall capacity/demand ratios presented above are based on an assumed average masonry wall compressive strength capacity of $f'_m$ = 17.5 MPa. The sensitivity of the masonry wall capacity/demand ratios to the potential upper and lower bound range of masonry wall material strengths was investigated. Refer to Table 6.6. A variation of only ± 5% and ±10% to the capacity/demand ratios was observed in the East-West and North-South directions respectively. The lack of sensitivity to masonry wall material strength selection was due to a high proportion of the masonry wall pier capacities being controlled by either the rocking or sliding shear mode of failure. The failure capacities in these two modes are primarily a function of axial load only. Assuming a higher value for the masonry compressive strength has little impact on the rocking failure mode or sliding failure mode capacities.

**Table 6.6: Masonry Wall Capacity/Demand Ratio Sensitivity to Material Strength**

| | **Masonry Wall Compressive Strength Range**<br>$f'_m$ = 9 MPa → 26 MPa |
|---|---|
| **East-West Direction**<br>Capacity/Demand Variation: | ± 5 % |
| **North-South Direction**<br>Capacity/Demand Variation: | ± 10 % |

It can be seen that the capacity/demand ratio are relatively insensitive to the masonry wall material strength selection.

### 6.7.3 Basement Concrete Walls – In-plane Capacity/Demand Evaluation

The capacity of the Centre Block's unreinforced plain concrete basement walls to resist seismic loads was assessed using a similar failure mode method of analysis as described for the above grade masonry walls. The capacity/demand ratios for 2010 NBCC seismic loads were found to be approximately 55% in the East-West direction and 90% in the North-South direction. The sensitivity of the plain concrete basement wall capacity/demand ratios to the potential range in concrete material strength was also assessed for the assumed upper and lower bound material strength range. Similar to the above grade masonry walls, the capacity/demand ratios were found to be relatively insensitive to the concrete material strength selection due to the predominance of the rocking and sliding shear modes of failure. As noted in Section 6.7.2 above, assuming a higher value for the concrete compressive strength has little impact on the rocking failure mode or sliding failure mode capacities. The results are presented in Table 6.7 below:

**Table 6.7: Plain Concrete Basement Walls Capacity Demand Ratio**

| | **Capacity/Demand Ratio (2010 NBCC Seismic Loads)** | **Sensitivity to Material Strength Range** $f'_c$ = 25 MPa → 40 MPa |
|---|---|---|
| **East-West Direction** | 55% | ± 5 |
| **North-South Direction** | 90% | ± 10 % |

### 6.7.4 Masonry Wall - Out-of-Plane Flexural Capacity/Demand Evaluation

Out-of-plane seismic wall loads (seismic loads applied to the face of the masonry walls) were calculated using section 4.1.8.18 of the 2010 NBCC.

Flexural out-of-plane unreinforced wall capacities were evaluated using CSA S304.1-04. It was assumed that the mortar joints of the existing brick masonry possess limited capacity to resist tensile stresses and that the out-of-plane flexural resistance of the walls is provided by axial compression in the walls resulting from the gravity loads only. The most critical wall spans occur in the upper levels of the structure where the seismic loads are the largest and the wall gravity loads are the lowest. Other critical wall spans include the double, triple and quadruple wall spans besides the Caucus rooms, Senate Chamber and House of Commons respectively.

It was found that in all cases, due to the substantial thickness of the masonry walls, that the Centre Block walls typically have sufficient out-of-plane flexural capacity to carry the out-of-plane seismic loads.

### 6.7.5 Masonry Wall Out-of-Plane Connection to Floor Diaphragms Evaluation

The out-of-plane connection of the walls to the floor diaphragms was also reviewed. Out-of-plane walls restraint is typically provided by pocketed steel floor beams framing in perpendicular to the face of the walls. It was found that in most areas, the steel floor beams bearing on the walls had adequate capacity to restrain the walls via friction beneath their bearing surfaces.

One exception was noted at the upper levels of the pavilion structures located at the South East and South-West corners of the Centre Block. See Figure 6.6. The span of the steel beam floor framing at the re-entrant corners of the pavilion structures alternates direction from floor to floor providing restraint at alternate levels to alternate exterior wall faces. At roof level, one side of the exterior walls at the re-entrant corners cantilevers from the sixth floor level.

A remedial program that partially addressed this deficiency was conducted in 1995. Cintec anchors were drilled through the exterior face of the facade walls and grouted into the roof level structure behind. Based on a review of the remedial work construction documents, it appears that the work was primarily limited to the south facade only. A similar unrestrained condition at roof level also exists at the north-east and north-west corners of pavilion structures as well. As the masonry walls have very limited capacity to cantilever, this condition should be remediated in a similar manner to the work already performed on the south facade walls.

A discussion of upgrade options to resist the 2010 NBCC seismic loads will be presented in the Work Package 3 Supplement Seismic Report.

#### 6.7.6 Centre Block Ventilation and Water Towers

The Centre Block has four towers located on the North side of the building. Two are currently in use as ventilation towers and two are obsolete water towers. The two ventilations towers have been the subject of a previous study and seismic strengthening program. See *Dynamic Analysis of the Centre Block Ventilation Towers, KIB Consultants Inc., 2012*. A similar specific study and strengthening program for the water towers should also be completed.

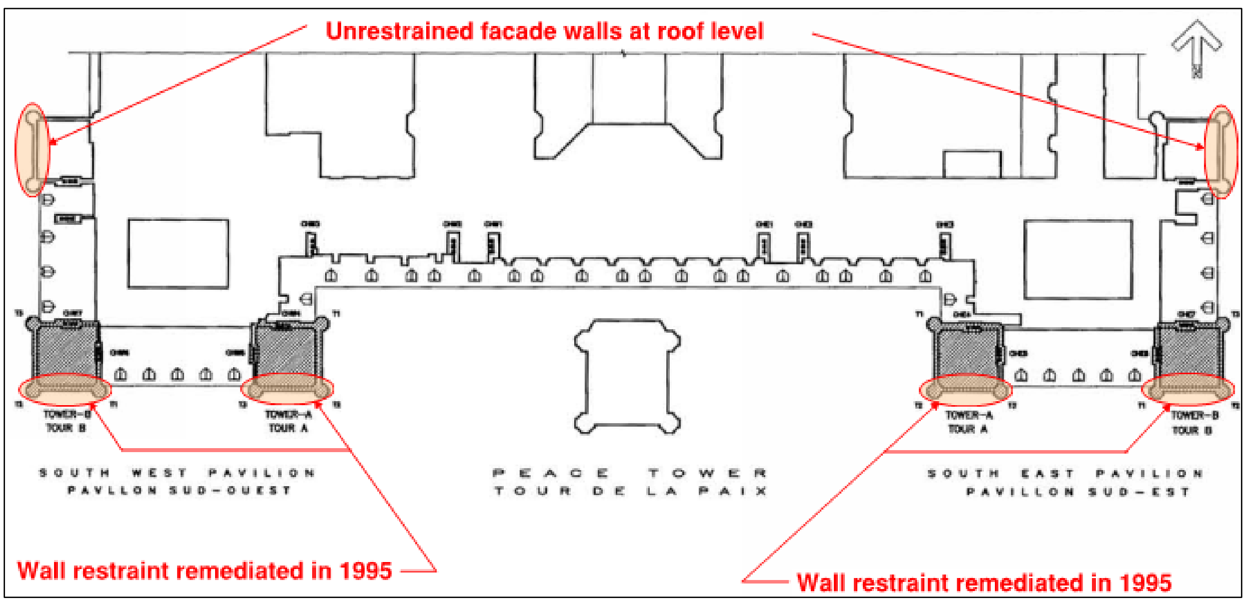


Figure 6.6: 1995 South Facade Remedial Work

### 6.8 Centre Block Diaphragm Capacities

#### 6.8.1 Description of Floor/Flat Roof Diaphragm System

The Centre Block floor and flat roof construction consists of terra cotta tile flat arches infilled between supporting steel beams with a weak cementitious topping. The thickness of the topping throughout the building is believed to have been varied depending on the intended floor finish. It is thought to be generally about 2.5 inches thick. Figure 6.7 provides an illustration of a typical terra-cotta tile flat arch floor system. A photo taken at the Old War Museum in Ottawa (a contemporary building to the Centre Block building with a similar floor system) showing a cross section of the floor system is also included for illustrative purposes.

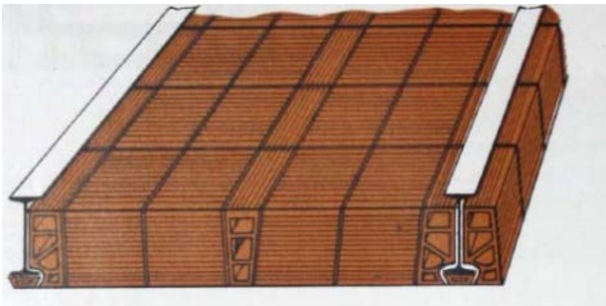

Figure 4-2: Flat arch construction as depicted by the National Fireproofing Company of Canada.

(a) Illustration of terra-cotta flat arch floor


(b) Example of terra-cotta flat arch floor (Old War Museum, Ottawa)

**Figure 6.7: Illustration of terra cotta flat arch floor construction**

The Centre Block Structural History Report (2014) notes that a patented product called Nail-a-crete was used to form the lightweight cementitious topping over the terra cotta flat arch floors. The 1919 American patent application for Nail-a-crete states a large possible compressive strength range of 1.4 MPa to 17MPa. The actual strength of the existing Centre Block topping is currently unknown, however, since it was intended to be soft enough to nail directly into it is likely to be very weak.

The steel floor beams that support the terra-cotta tiles are embedded in pockets in the load bearing masonry walls. This is often the only connection of the floor system to the load bearing walls. A gap between the perimeter steel members parallel to the load bearing masonry wall typically exists. Figure 6.8 illustrates a typical condition.

(a) Typical floor edge condition (Old War Museum, Ottawa – Similar to Centre Block)

(b) Gap between floor edge and load bearing wall (Centre Block)

**Figure 6.8: Floor edge condition example**

### 6.8.2 Diaphragm Capacity/Demand Evaluation

The function of a structural floor diaphragm is to transmit the floor plate inertia forces generated by seismic shaking to the vertical elements of the seismic force resisting system. A structural floor diaphragm also serves to tie the vertical elements of the seismic force resisting system together and to distribute seismic load amongst the individual elements.

The Centre Block floor assemblies, therefore, need to have the capacity to span like horizontal beams between adjacent walls and also to collect and transfer the lateral seismic floor loads into the walls.

Commentary regarding these to specific diaphragm actions is included below.

Diaphragm Span Capacity

The Centre Block building contains many large open spaces. These include the House of Commons, two caucus rooms, the Hall of Honour, and the Senate Chamber. The absence of lateral load resisting walls in these spaces results in some significant diaphragm spans for the floor/roof structure above these areas.

Figure 6.9 highlights some key areas with large diaphragm spans. The floor/roof structures in these areas are typically required to span horizontally between the north corridor and south corridors wall lines. An exaggerated deflected shape of the diaphragms under East-West earthquake loading is shown to illustrate the critical span direction.

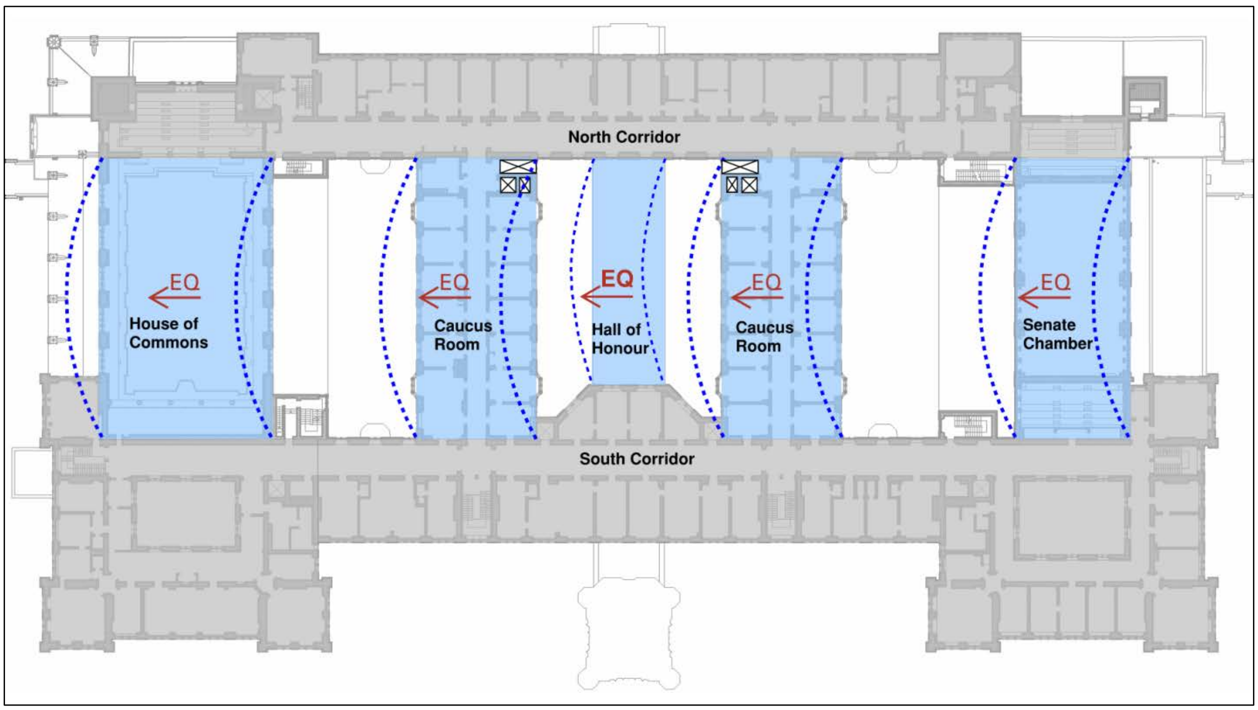


**Figure 6.9: Key Plan of Large Diaphragm Span Areas**

The floor structure has two potential sources of diaphragm span capacity.

Initially, the thin cementitious topping may provide some limited in-plane flexural and shear strength up until flexure of the diaphragm induces stresses exceeding the flexural tensile strength of the unreinforced topping. A potential range for the topping diaphragm span capacity can be estimated using the range of material values noted in section 6.3.8. Once the cementitious topping has cracked, it is expected that its contribution to diaphragm span capacity will rapidly degrade.

Some subsequent residual diaphragm capacity may then be achieved through infill arch action of the terra cotta floor assembly. As the deformation of the diaphragms increases, inclined compression struts through the terra-cotta units could develop, with the grillage of steel floor beams acting as confining tension ties. It is anticipated that the two potential diaphragm span mechanisms will be unable to act concurrently, as a reasonable level of in-plane deflection will be required to activate the in-plane arching mechanism at which point the diaphragm span contribution from the unreinforced cementitious topping will likely have already been compromised due to flexural cracking.

An accurate assessment of the floor assembly's ability to span as a diaphragm using the infill arch mechanism can only be definitively established through experimental testing. An approximate assessment may be made, however, by assuming that the diaphragm span capacity of this mechanism is controlled by a sliding shear failure along the mortar joint lines between the terra-cotta units. Since little is known regarding the actual material strengths of either the infill terra-cotta units or the mortar used between the units, the following assumptions are made:

$f_{unit}$ = 15 MPa (assumed)

Mortar = Type N (assumed)

$f'_m$ = 6 MPa (Table 3, CSA S304.1-04)

The sliding shear stress (cohesion) capacity is calculated as:

$V_r = \phi\ 0.16\ f'_m{}^{0.5} = 0.6 \times 0.16 \times 6.0^{0.5} = 0.24$ MPa

Table 6.8 summarizes the range of estimated capacity/demand ratios for the critical East-West earthquake diaphragm spans over the House of Commons, the two caucus rooms, the Hall of Honour and the Senate Chamber.

The capacity/demand ratios considering the two potential diaphragm mechanisms are listed separately.

**Table 6.8: Diaphragms Capacity/Demand Ratio**

| | | Floor Diaphragm Above House of Commons | Roof and Floor Diaphragms Above Caucus Rooms | Floor Diaphragm Above Senate Chamber |
|---|---|---|---|---|
| **Cementitious Topping Shear Strength** | $V_r$ / $V_f$ | 0.03 - 0.11 | 0.02 - 0.08 | 0.03 - 0.10 |
| **Cementitious Topping Flexural Strength** | $M_r$ / $M_f$ | 0.03 - 0.12 | 0.05 - 0.17 | 0.02 - 0.06 |
| **Infill Terra-Cotta Arching** | $V_r$ / $V_f$ | 0.20 | 0.17 | 0.18 |

Table 6.8 indicates that the weak cementitious floor topping may provide a diaphragm span capacity in the range of 2% - 11% of 2010 NBCC seismic demand levels. It is estimated that the diaphragm span mechanism using rigid arching through the infill Terra-cotta flat arch floor units infill may be able to provide up to 20% of the 2010 NBCC seismic loads.

Diaphragm Transfer Capacity

A mechanism for transferring floor plate inertia forces out of the floor assemblies and into the adjacent load bearing masonry walls is required in order for the Centre Block's load bearing masonry walls to provide any lateral restraint to the structure.

In its current state, the existing floor structure assembly has no, or very limited capacity to perform this function. Often the only connection of the floor assemblies to the masonry walls is where the steel floor beams are embedded in wall pockets. A limited amount of lateral load may be transferred from the floor assembly to the walls through bearing on the faces of the pocketed beam webs. At locations where the walls are parallel to the span of the terra-cotta infill units, there is no connection of the floor assembly to the walls at all and subsequently no load lateral load transfer mechanism.

The most critical diaphragm force transfer critical areas occur at the ends of the long span diaphragms noted above where the transfer forces are concentrated.

Figure 6.10 illustrates some examples of areas with walls that have no connection to the floor assemblies.

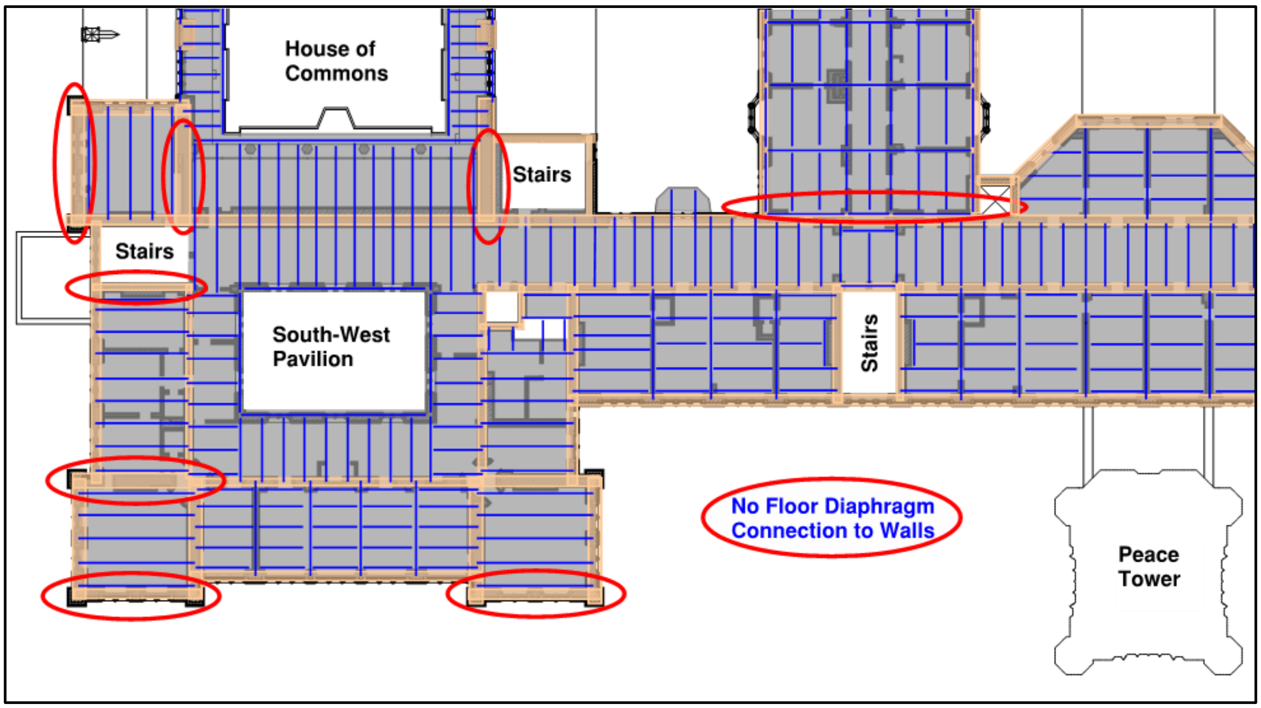


**Figure 6.10: Example of Walls Unconnected to Diaphragms**

Diaphragm Collector Capacity

A further diaphragm capacity requirement also occurs at the ends of the long span diaphragms. In these areas, the area of adjacent floor structure needs to be able to collect and distribute the localized load concentration to walls segments further along the wall line.

Neither the terra-cotta infill floor nor the unreinforced topping slab areas have any capacity to act as a diaphragm collector element.

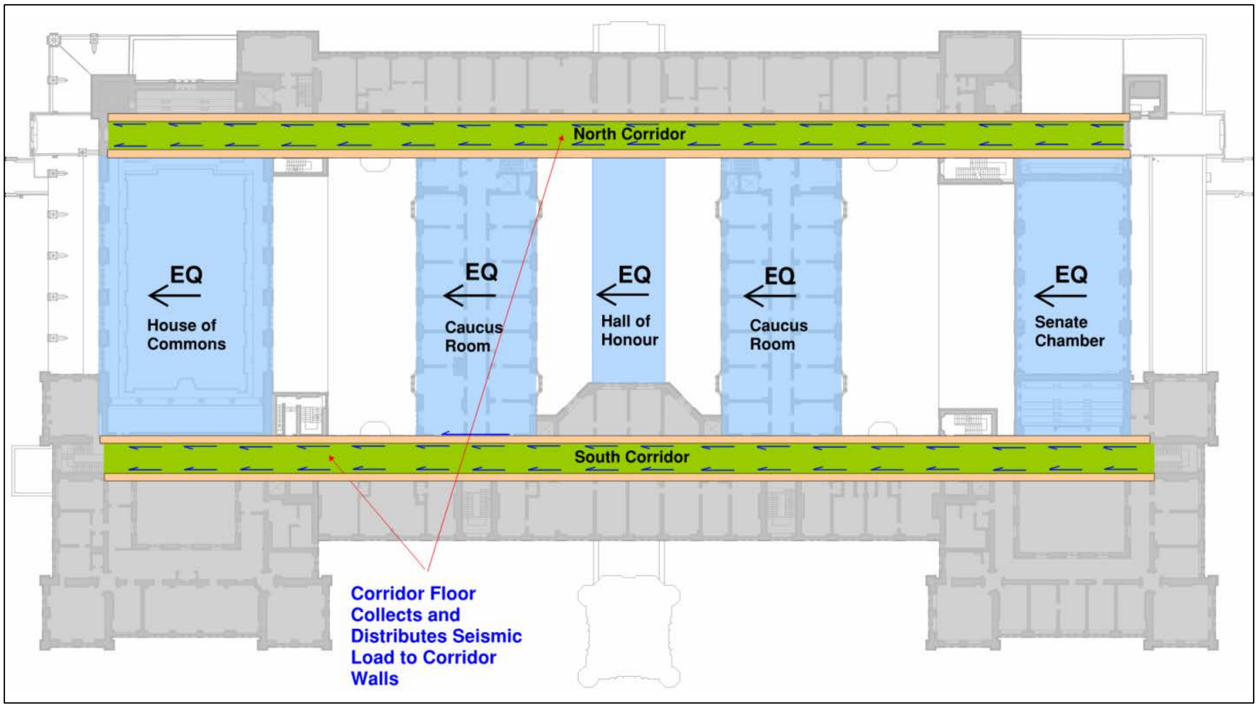


Figure 6.11: Corridor Diaphragm Collectors

Additional Diaphragm Issues

Some other diaphragm deficiencies have also been identified and are discussed below.

1) Level 3 Floor Structure Over Hall of Honour

As can be seen in Figure 6.12, the double storey height space enclosing the Hall of Honour largely divides the Centre Block third floor plate in half. The two halves are connected by a few floor segments at the north and south ends of the Hall of Honour only. Any differential response under seismic loading between the east and west halves of the third floor plate will impose significant strains upon these areas making them vulnerable to significant localized damage and/or partial collapse.

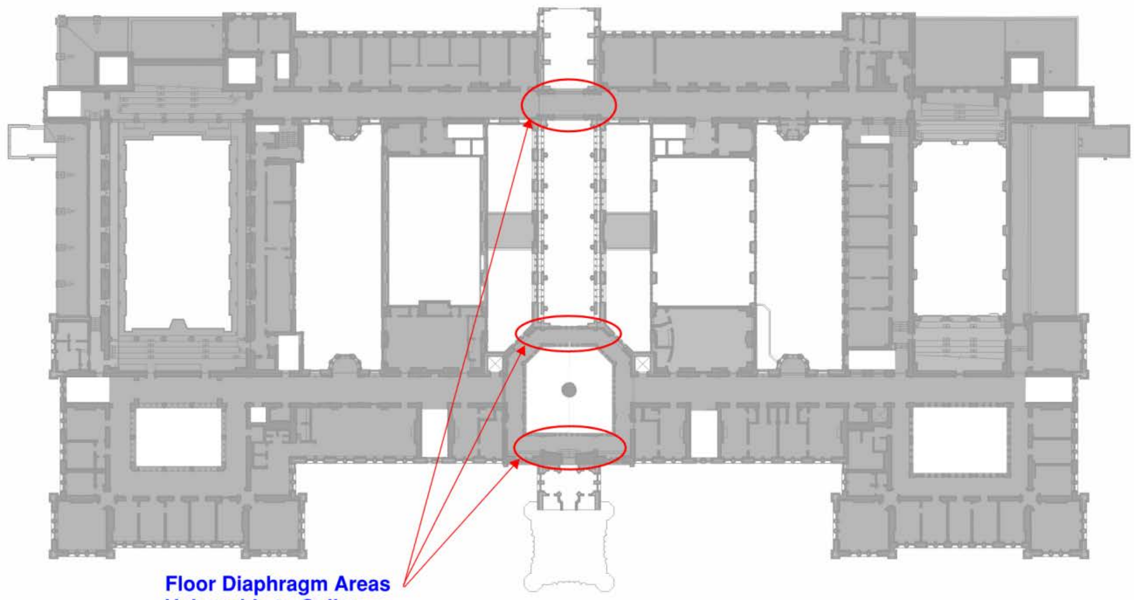


Figure 6.12: Vulnerable Level 3 Floor Diaphragm Areas

2) Diaphragm Connection of Sloped Roofs to Perimeter Walls

Copper clad sloped roofs exist around the perimeter of the building. The sloped roofs typically start at the fifth floor level, fly past the 6th floor level and then terminate at the main roof level. The sloped roofs are constructed of a thin layer of concrete applied to a metal lathe and supported on sloped steel framing. Figure 6.13 is a photo taken during construction. The partially erected sloped roof on the North face of the Centre Block can be seen.

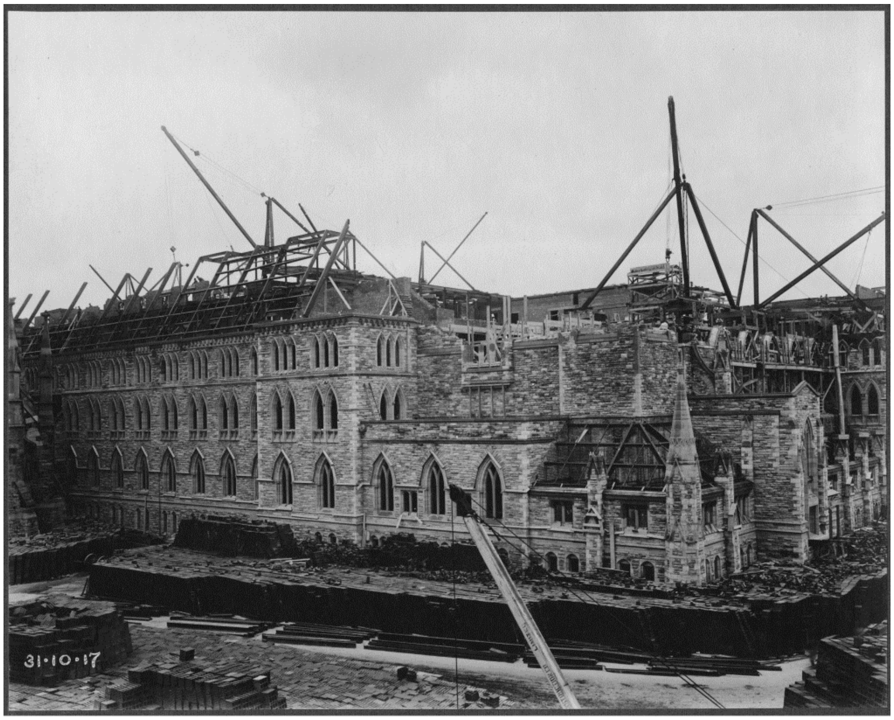


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Figure 6.13: Sloped Roof Construction

In the building's current state, the sloped roofs are required to act like inclined wall elements to provide lateral restraint to portions of both the Main Roof structure and the 6th level floor structure. Seismic loads from these levels need to be transferred down to the tops of the walls at fifth floor level. The thin unreinforced concrete on metal lathe has no or very limited capacity to transfer these loads. The connection at the base of the sloped roof structures to the top of the fifth floor level walls is unknown, but unlikely to be adequate to transfer any significant lateral loads into the walls below.

# 7. PEACE TOWER PRELIMINARY SEISMIC ANALYSIS RESULTS

## 7.1 Peace Tower Seismic Force Resisting System

The Peace Tower seismic force resisting system (SFRS) primarily consists of unreinforced concrete walls with an exterior wythe of stone masonry. The foundations are constructed of unreinforced concrete piers.

Above the observation deck level, the SFRS consists of four unreinforced concrete and stone piers, as well as steel moment frames to the underside of the clock face. At the clock face, the walls are constructed of stone masonry and have been previously reinforced in plane with steel cross bracing. The sloped roof above the clock faces is constructed of reinforced concrete walls and piers.

## 7.2 Peace Tower Structural Modelling

A 3D finite element analysis model of the Peace Tower was created using ETABS version 13.2.1. Figure 7.1 displays the 3D computational model. The model was used to determine the building's fundamental period, to perform an equivalent static and dynamic analysis, and to test the sensitivity of the analysis results to the definition of the material properties. The Peace Tower was modelled both combined with the Centre Block and separately from the Centre Block. It was found that the results of the Peace Tower were not significantly affected by the connection between the two structures because they behave independently from each other.

The walls and floors were modelled using finite element shell objects. Beam and columns were modelled with frame elements. Reduced element stiffness resulting from cracking of the concrete and masonry sections was incorporated through modification of the element stiffness modifiers.

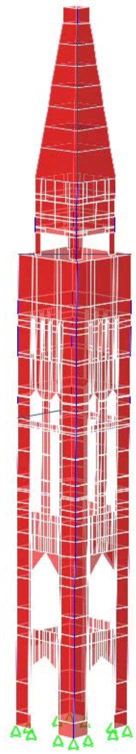


Figure 7.1: Peace Tower ETABS Model

## 7.3 Peace Tower Material Properties

The Peace Tower walls are composed of an exterior wythe of snecked sandstone masonry with an unreinforced concrete backing. Chemical analysis of the mortar has shown it to be a hard, Portland cement based mortar, as discussed in Section 6.3. An example of the construction is shown in Figure 7.2 and Figure 7.3.

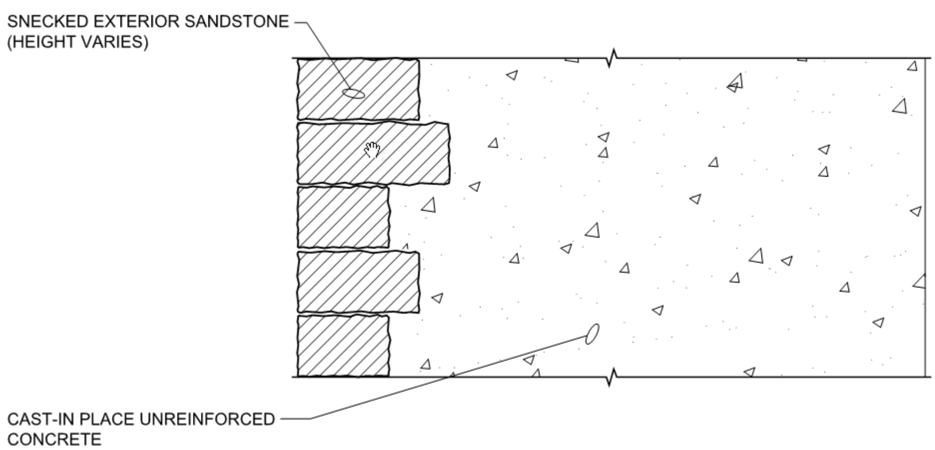



Figure 7.2: Typical Peace Tower Wall Section


PA-800437

Figure 7.3: Historic photo of concrete being placed behind exterior stone wythe

To evaluate the sensitivity of the seismic analysis to the range of potential material strengths, upper bound and lower bound values for the compressive strengths and elastic modulus have been determined.

Concrete strength tests from "The Peace Tower Stone Conversation Report" by Herbert Read Ltd., in 1996 and from tests completed during a 1990 restoration project on the Peace Tower for which Halsall was the consultant give an approximate concrete strength of 40 MPa. This number was used as an upper bound for the concrete compressive strength because it is already higher than a typical concrete strength from that era.

A concrete strength of 25 MPa was assumed as a lower bound because it is a typical value for concrete from this time period. It is unlikely that the average concrete strength would be lower than this value based on the results of the strength tests that have been completed.

The elastic modulus for concrete was calculated using the following equation from the concrete standard CSA A23.3:

$E_c = 4500 \cdot (f'_c)^{0.5}$

A summary of the concrete compressive strengths and elastic moduli is summarized below, in

Table 7.1.

**Table 7.1: Peace Tower Concrete Property Ranges**

| | Compressive Strength | | | Elastic Modulus | | |
|---|---|---|---|---|---|---|
| | Min | Max | Average | Min | Max | Average |
| Concrete | 25 MPa | 40 MPa | 32.5 MPa | 22.5 GPa | 28.5 GPa | 25.7 GPa |

The stone masonry wythe typically comprises 10% to 20% of the overall wall thickness, increasing in percentage with height of the tower as the wall tapers. The strengths and stiffnesses of the concrete and the stone masonry are reasonably similar, as given in the Centre Block material properties discussion ($f'_m$ = 38 MPa, $E_m$ = 16 MPa as a lower bound). Given the relatively small contribution of the stone masonry, the walls were modelled using only the concrete properties as a reasonable estimate.

## 7.4 Peace Tower Fundamental Period

The fundamental periods of the Peace Tower in the East-West and North-South building directions were evaluated from a modal analysis of the computer model. The fundamental periods in both directions did not vary significantly whether or not the link between the Peace Tower and the Centre Block was considered.

Fundamental periods corresponding to the assignment of both the potential lower bound and upper bound material properties were calculated and are presented in Table 7.2 below:

**Table 7.2: Peace Tower Fundamental Periods**

| | Lower Bound Material Properties | Upper Bound Material Properties |
|---|---|---|
| East-West Tower Direction | 0.81 sec | 0.72 sec |
| North-South Tower Direction | 0.74 sec | 0.66 sec |

These analytical periods are based on a torsionally locked model in accordance with the dynamic analysis procedure recommended by the 2010 NBCC. This procedure has the effect of stiffening the model during the scaling process, which results in the slightly lower periods given above. The final analysis model, which is not torsionally locked, has slightly longer periods.

These periods compare well with the periods that have been measured by accelerometers in the tower in previous studies. The measured periods have typically been in the range of 0.8 to 0.9 seconds.

The fundamental periods of the Peace Tower were evaluated both with the model built integrally with the main Centre Block structure and separately as an independent structure (See Figure 6.1 and Figure 7.1, respectively). The fundamental periods varied by only 2% in either direction, indicating almost completely independent response of the Peace Tower from the main Centre Block structure.

### 7.5 Peace Tower Equivalent Static Base Shear

The 2010 NBCC formula for calculation of the Equivalent Static base shear is:

$$V = \frac{S(T_a)\, M_v\, I_E}{R_d\, R_o} W$$

Where:

$T_a$ = fundamental lateral period of vibration

$S(T_a)$ = the design spectral response acceleration, expressed as a ratio to gravitational acceleration for a period of $T_a$, the fundamental lateral period of vibration. See Table 7.2.

$M_v$ = factor to account for higher mode effect on base shear (= 1.0 for $T_a$ < 1.0 sec)

$I_e$ = Importance Factor for Earthquake Loads (= 1.0 for Normal Importance)

$R_d$ = the ductility-related force modification factor reflecting the capability of a structure to dissipate energy through inelastic behaviour (= 1.0 for unreinforced concrete)

$R_o$ = the over-strength-related force modification factor accounting for the dependable portion of reserve strength (= 1.0 for unreinforced concrete)

W = the weight of the building = 76,000 kN

Note: The maximum equivalent static base shear for an unreinforced concrete structure is the base shear corresponding to a fundamental period of 0.2 seconds. The evaluation of the Peace Tower is not governed by the maximum equivalent static base shear.

Table 7.3: Peace Tower Equivalent Static Base Shear

| East - West Tower Direction | | | |
|---|---|---|---|
| | Lower Bound Material Properties | Upper Bound Material Properties | Median Variance |
| Fundamental Period | 0.81 sec | 0.72 sec | |
| Equivalent Static Base Shear | V = 0.088 W = 6,710 kN | V = 0.10 W = 7,690 kN | ± 7 % |
| **North - South Tower Direction** | | | |
| | **Lower Bound Material Properties** | **Upper Bound Material Properties** | **Median Variance** |
| Fundamental Period | 0.74 sec | 0.66 sec | |
| Equivalent Static Base Shear | V = 0.10 W = 7,560 kN | V = 0.11 W = 8,410 kN | ± 6 % |

Unlike the main Centre Block structure, the range of elastic modulus values do affect the equivalent static base shear because the structure is not stiff enough to be governed by the maximum code required equivalent static base shear.

### 7.6 Peace Tower Dynamic Analysis

A linear dynamic response spectrum analysis was performed in accordance with the requirements of 2010 NBCC. The Peace Tower was determined to be an irregular structure. The 2010 NBCC dynamic analysis provisions for irregular buildings require the dynamic base shear to be scaled such that it is at least 100% of the equivalent static base shear. After the initial analysis, the dynamic base shears in both the north-south and east-west directions of the Peace Tower were found to be greater than 100% of the equivalent static base shear. The dynamic base shears in each of the primary tower directions are summarized below:

Table 7.4: Peace Tower Dynamic Base Shear

| East - West Building Direction | | | |
|---|---|---|---|
| | Lower Bound Material Properties | Upper Bound Material Properties | Median Variance |
| Dynamic Base Shear | $V_d$ = 7,680 kN | $V_d$ = 8,310 kN | ± 4 % |
| **North - South Building Direction** | | | |
| | Lower Bound Material Properties | Upper Bound Material Properties | Median Variance |
| Dynamic Base Shear | $V_d$ = 8,520 kN | $V_d$ = 9,150 kN | ± 4 % |

Similar to the bases shears calculated using the Equivalent Static Analysis, the range of elastic modulus values do affect the dynamic base shears as well because the structures is not stiff enough to be governed by the maximum code required base shears.

### 7.7 Peace Tower Seismic Capacity Evaluation

The walls on each face of the Peace Tower are punctuated by several large openings that define the piers and spandrel beams that make up the overall structure of the tower. The diagram below highlights the piers (in red) and spandrel beams (in blue) on the respective faces of the Peace Tower.

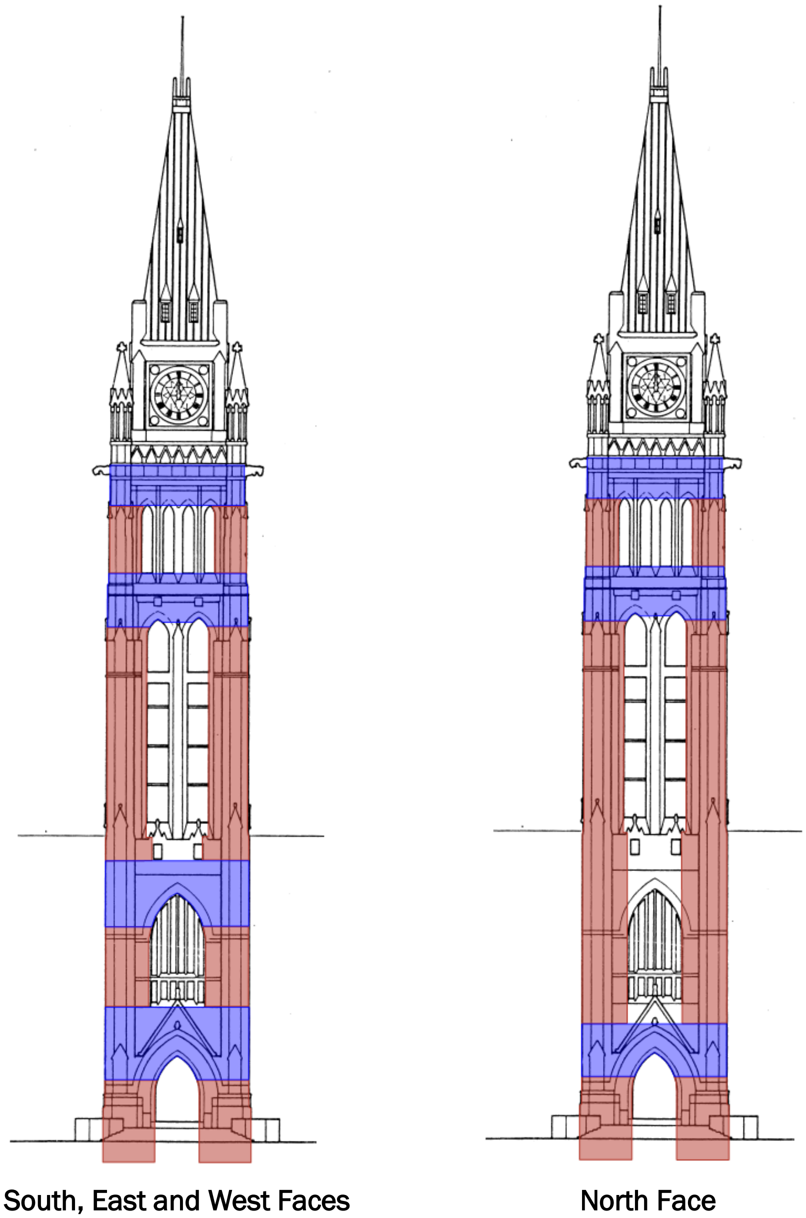



Figure 7.4: Piers and spandrel beams highlighted on Peace Tower elevations

The tall, slender pier in the middle opening of the South, East and West faces, as well as the small spandrel beam that connects it to the outer piers at about 75% of its height, were also evaluated but were found to not have sufficient strength to contribute significantly to the SFRS. These elements will fail to resist lateral loads at roughly 10% of the 2010 NBCC seismic loads, at which point they will no longer contribute the SFRS.

The north face of the Peace Tower is punctuated by a larger opening in the structure because of the sloping elevator that was installed in the 1980's. The existing wall on the north face at that location is made up of a stone veneer with a backup wall and steel space frame. However, this structure cannot contribute significantly to the strength of the SFRS.

The observation deck level, clock face and sloped roof together comprise the "Upper Tower" and account for only 7% of the mass of the whole structure. The calculation results for these components are discussed separately in Section 7.7.6.

The seismic capacity of the Peace Tower has been evaluated for: global overturning resistance, pier capacity and spandrel capacity. Each of these will be discussed in the following sections in detail.

### 7.7.1 Global Overturning Capacity/Demand Evaluation

The ability of the Peace Tower to resist overturning as a whole tower relies entirely on the self-weight of the structure because there is no tensile reinforcing. The Peace Tower has sufficient capacity to resist 100% of the overturning caused by the 2010 NBCC seismic loads, provided spandrel beams have sufficient strength to tie the piers together as a frame.

As a lower bound of the capacity of the structure, it is possible to consider the four corner piers without the benefit of framing (for example, if the spandrels fail). In this scenario, each of the four corner piers behaves independently and acts as a tall cantilever, with a much smaller footprint than the overall plan of the tower. However, piers on their own do not have sufficient capacity to resist either 60 or 100% of the 2010 NBCC seismic loads, so shear and moment transfer between the piers via the spandrel beams is required to resist overturning.

### 7.7.2 Pier Capacity

The capacities of the piers of the Peace Tower have been evaluated using the same criteria as the piers of the Centre Block walls, as discussed in Section 6.7.1, except using unreinforced concrete properties instead of brick masonry where appropriate. The failure modes are:

a) Rocking

This failure mode is the same as described in Section 6.7.1.

b) Crushing

This failure mode occurs when the combined axial stresses in a wall due to gravity and lateral loads exceeds the compressive strength of the concrete. From CSA A23.3, clause 22, the maximum compressive strength of concrete is $0.75 \cdot \varphi_c \cdot f'_c$.

c) Shear

This failure mechanism is characteristic of loading cases with high shear. The in-plane shear capacity of the unreinforced concrete was evaluated using clause 22 of CSA A23.3.

d) Sliding Shear

This failure mechanism is characteristic of loading cases with high shear and low axial load. The sliding shear capacity of the unreinforced concrete piers was evaluated using clause 11.5 of CSA A23.3.

### 7.7.3 Spandrel Beam Capacity

The capacities of the spandrel beams of the Peace Tower have been evaluated for their ability to transfer shear and the bending moment associated with that shear between the corner piers.

Some of the spandrel beams consist only of unreinforced concrete and stone masonry. Their shear and sliding shear capacities were calculated the same as the piers. The bending capacity of the section was calculated assuming a maximum tensile strength in the concrete of $0.37 \cdot \varphi_c \cdot (f'_c)^{0.5}$ in accordance with CSA A23.3, clause 22.

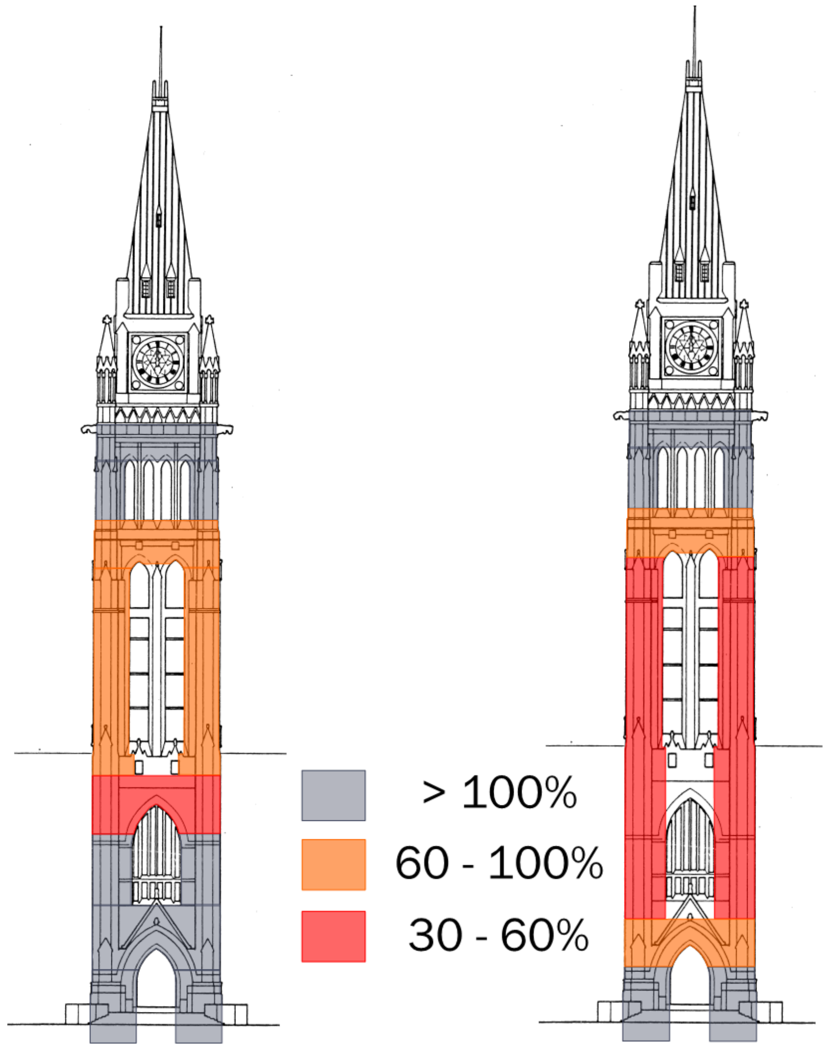
Other spandrel beams have steel beams embedded in the concrete or attached to the side of the concrete. These spandrel beams were evaluated for the moment and flexural capacity in accordance with CSA S16. A yield strength of 210 MPa was used for steel from the original construction (as per the original specifications), and a yield strength of 345 MPa for structural steel added in the 1980's renovation. The steel beams were also checked for their embedment into the concrete piers because they must be able to transfer their end moment into the pier without the web of the steel beam buckling or the concrete crushing.

### 7.7.4 Peace Tower to Centre Block Connection Capacity

As discussed in the gap analysis, the details of the connection between the Peace Tower and the link that connects it to the Centre Block at its base are not known. However, given that the two structures behave and will move independently from each other, it is likely that any masonry connection at this level would be significantly damaged by the relative displacements of the two structures in the event of an earthquake. The connection could be damaged either by cracking as the two structures move away from each other or by crushing failure as the structures move towards each other.

### 7.7.5 Pier and Spandrel Beam Capacity/Demand Evaluation

Based on the pier and spandrel beam failure mode method of analysis described above, the ultimate capacity of each face of the Peace Tower in the East-West and North-South directions was evaluated. The capacity/demand ratios are presented in Figure 7.5 below:




**South, East and West Faces** **North Face**

**Figure 7.5: Piers and Spandrel Beam Capacity/Demand Ratios (2010 NBCC)**

On the South, East and West faces, the weakest link is the spandrel beam immediately above the memorial chamber. This spandrel beam consists of 4 structural steel beams encased in concrete, which extend into the corner piers. This element does not have sufficient capacity to resist either 60 or 100% of the 2010 NBCC seismic loads.

On the North face, the weakest link is the two tall, slender piers on either side of the main opening. These piers do not have the sufficient capacity to resist 60 or 100% of the 2010 NBCC seismic loads.

Several other piers and spandrel beams, as highlighted in orange, also have between 60 and 100% of the required capacity to resist the 2010 NBCC seismic loads.

A discussion of upgrade options to resist the 2010 NBCC seismic loads will be presented in the Work Package 3 Supplement Seismic Report.

#### 7.7.6 Upper Tower Capacity/Demand Evaluation

The Upper Tower of the Peace Tower, consisting of the structure above the observation deck level, is constructed of three main sections. The lowest section, at the observation deck level, consists of four unreinforced concrete and stone piers, as well as steel moment frames to the underside of the clock face. At the clock face, the walls are constructed of stone masonry and have been previously reinforced in plane with steel cross bracing. The sloped roof above the clock faces is constructed of reinforced concrete walls and piers. A simplified elevation of the Upper Tower is show below in Figure 7.6.

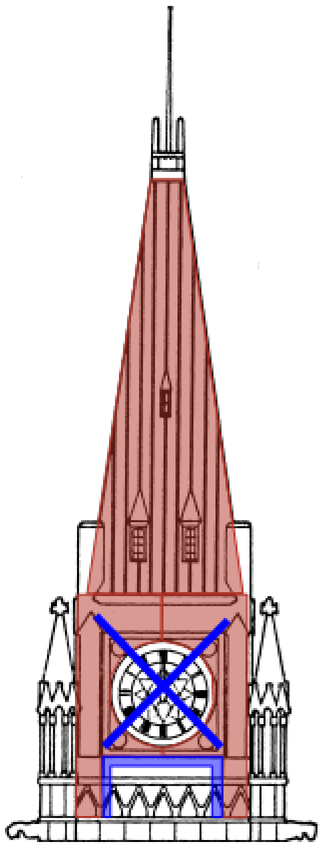


**Figure 7.6: Upper Tower elevation of structural elements**

The steel moment frame and cable bracing, which were both added in renovations to the tower, are shown in blue.

The same range of material properties used as described in section 7.3. In addition, the reinforcing steel in the sloped concrete roof was assumed to be 210 MPa, which is typical for reinforcing steel of that era.

The sloped concrete roof of the Upper Tower is comprised of reinforced concrete walls, piers and purlins. These elements primarily act as shear walls to resist lateral loads and have between 60 and 100% of the capacity required to resist the 2010 NBCC seismic loads. However, the concrete elements do not appear to be positively connected to the masonry walls surrounding the clock face below. As a result, global overturning is resisted only by the mass of the roof and there is insufficient capacity to resist either 60 or 100% of the 2010 NBCC seismic loads.

The structure around the clock faces is comprised of unreinforced stone masonry, with a large circular opening on each side for the clocks. Additional steel bracing cables were installed in the early 1990's to resist added wind loads from scaffolding and remain in place. The cables provide the primary lateral load path at this level but do not have the required tensile capacity to resist either 60 or 100% of the 2010 NBCC seismic loads.

Finally, the observation deck level is comprised of four stone and concrete piers as well as a steel moment frame that replaced a section of wall on each face that was removed in the early 1980's. The stone and concrete piers are much stiffer than the steel moment frame and so will initially attract most of the load. However, they do not have the overturning capacity to resist either 60 or 100% of the 2010 NBCC. The steel moment frame will add additional overturning capacity but a more refined analysis is required to determine exactly how the structure will perform at the large displacements required to engage the flexible moment frame.

Significantly, another weakness of the Upper Tower is that there is no effective load path between the three main components described above. Even if their individual weaknesses are addressed, a complete load path for the seismic forces must be provided.

A discussion of upgrade options to resist the 2010 NBCC seismic loads will be presented in the Work Package 3 Supplement Seismic Report.

#### 7.7.7 Sensitivity of Capacity/Demand Ratios to Material Property Selection

The primary material property assumption required for the Peace Tower is the strength of the concrete and its associated stiffness. However, the impact of the assumption is somewhat minimized because stronger concrete strengths result in a stiffer structure and higher loads. Oppositely, lower concrete strengths result in a softer structure and lower loads.

The capacity/demand ratios presented above are based on an average concrete compressive strength of 32.5 MPa. The sensitivity of the concrete pier and spandrel beam capacity/demand ratios to the potential upper and lower bound range of concrete material strengths was investigated. A variation of up to ± 10% of the capacity/demand ratios was observed in both the North-South and East-West directions of the Peace Tower.

Although the assumed range of concrete strengths varied by approximately ± 25%, the results varied by only ± 10% because the controlling failure mechanism of many of the pier sections was rocking, which is insensitive to compressive strength. Also, many of the failing spandrel beams are controlled by the strength of the steel beams, the strength of which is better documented.

# 8. SEISMIC ANALYSIS OF NON-STRUCTURAL AND SECONDARY STRUCTURAL COMPONENTS

Seismic analysis of non-structural and secondary structural components involves the determination of the capacity of the connection to the structure, and comparison to the seismic loads imparted to these components as calculated according to the 2010 NBCC. Non-structural components generally include mechanical, electrical, and architectural features. This would include items such as ductwork, air handling units, cable trays, suspended ceilings, and light fixtures. Secondary structural components include masonry walls which do not necessarily contribute to the seismic resistance of the building, but form part of the gravity load-resisting system of the building.

As indicated by PWGSC, most of the mechanical and electrical systems will be removed as part of the planned upgrades to the Centre Block. A general description of the existing conditions will be indicated for reference, as well as a comment on the existing seismic restraint of these systems. The focus of this report will be the historic architectural features, namely the arched ceiling structures, parapets, chimneys, and light fixtures. Other suspended ceilings and light fixtures will be mentioned.

## 8.1 Methodology

The Centre Block was visited on July 31, 2014 and September 11, 2014 to observe the non-structural and secondary structural components. The level of analysis included a count of the number of each type of item, and a visual review of the connection to the structure to determine if the item is seismically restrained.

Also reviewed in the assessment of the non-structural components were the original drawings showing the connection of the ceilings and other features to the structure, as well as drawings indicating the upgrades to the chimneys, the Peace Tower, and the South Façade. A full list of references can be found in Section 2.3.

## 8.2 Background on OFC Seismic Restraint Requirements

According to the 2010 NBCC, all new buildings and renovations require seismic restraint for Operational and Functional Components (OFC). It is sometimes desirable to perform a retrofit even though it is not required by the NBCC as a measure of protection for the occupants, and to protect the asset against damage. A typical building is assessed for the Normal importance level so that occupants can exit the building without serious injury, with other buildings being assessed to a High or Post-Disaster importance level based on the requirements for allowable damage and allowable down-time of critical mechanical and electrical systems.

Guidance for deciding which components require seismic restraint at a Normal importance level is based on CSA S832-06 (Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings), and the additional standards that it references. There are certain components that are exempt from restraint at a Normal importance level, based upon observations of which OFCs were not damaged during previous earthquakes. On the other hand, due to heritage concerns or cost of replacement, certain buildings or components may be required by the owner to be restrained when they otherwise would not need to be restrained by the indicated standards.

### 8.2.1 Exemptions from Seismic Restraint

Exemptions from seismic restraint are provided by industry guidelines by the NFPA (National Fire Protection Association), SMACNA (Sheet Metal and Air Conditioning Contractors' National Association), ASHRAE (American Society of Heating, Refrigerating and Air-Conditioning Engineers), and CISCA (Ceilings and Interior Systems Construction Association). Most of this information comes from observations of damage in previous earthquakes.

Large or heavy suspended items and tall or heavy base-mounted units are required to be restrained for a Normal importance level building. The typical items requiring seismic restraint are indicated in Table 8.1. This Table does not include exemptions specifically for the mechanical equipment (piping and ductwork, including sprinkler systems) as that is understood that it will all be removed. Any equipment containing hazardous materials would require seismic restraint.

**Table 8.1: OFCs requiring seismic restraint for a Normal Importance Level building**

| Component | Description |
|---|---|
| Suspended Equipment | Independently hung, weighing more than 9kg (20 lb). |
| Base-mounted Equipment | More than 180kg (400 lb) weight; OR |
| | Has an overturning moment; OR |
| | Mounted on a frame more than 1.2m (4 ft) tall, or is more than 1.8m (6ft) tall |
| Wall-mounted Equipment | Weighing more than 9kg (20lbs) |
| Architectural components | Skylights, cladding, ceiling systems, and partition walls |

In Table 8.1, independently hung suspended equipment refers to items that are not connected to other items sharing the same gravity restraint. For example, a light that is suspended in a tee-bar ceiling system would not be independently hung, while a chandelier could be considered independently hung. The light that is suspended in the ceiling would be restrained with the ceiling grid.

Base-mounted equipment would be considered to have an overturning moment if the seismic load would overcome the gravity load, which typically happens to items that are tall and narrow. Base-mounted items that are mounted off of the ground or are particularly tall are considered more likely to injure someone than a similar item which is closer to the floor. In Centre Block, this is likely to be a sculpture on a stone pedestal. Similarly, the architectural components listed in Table 8.1 are more likely to be located high enough above the ground to cause injury in a seismic event.

Ceilings are exempt in small rooms (less than 13.4m$^2$) with full-height walls that are connected to the structure above, or in areas with rigid suspended ceilings (i.e. lath and plaster) where the ceiling is connected to the walls on all sides. Some areas in the Centre Block will fall into each of these categories, with the connection to the wall assumed to be constructed as per the original drawings as it was not visible during our review.

### 8.3 Qualitative Assessment

This section of the report will describe the features which are existing in the Centre Block, their current connection to the structure, and the extent to which they can be considered seismically restrained according to the 2010 NBCC. This was determined with a visual review, and items where the connections were not visible are noted as such. Most suspended architectural, mechanical, and electrical items were suspended from the slab above, with hangers connected to the beams above. Very few items were seismically restrained, with an exception being some new cable trays running around the perimeter of the basement. Other items, including typical office light fixtures, small wall-mounted electrical items, and small-diameter conduit running along the ceiling are exempt from seismic restraint according to the various standards referenced in CSA S832-06.

#### 8.3.1 Exterior

There are many large pieces of stonework on the exterior of the Centre Block and the Peace Tower that are historically significant, including the grotesques, towers, and chimneys. Some items have been repaired and seismically restrained, while other items are not sufficiently restrained. Cladding should also be restrained to provide uninhibited egress from the building in a seismic event.

The flat roofing assembly is a protected roofing type, with ballast paving stones and rigid extruded polystyrene insulation. The roofing is deteriorating; some stones are cracked and the insulation is degrading. There is the potential for the roofing stones or pieces to fall off of the building. Loose ballast should be removed from the roof. In other locations, the roof is a sloped copper roof. There are also decorative fences, spires, and lightning rods connected to the top of the copper roofing. Most of these decorative items are lightweight and the standard screwed or welded connections should be sufficient to resist the seismic loads. The connections of the items to the roof and the roof to the structure below should be confirmed, but it is not expected to require additional restraint if the connections are in good condition.

There are several turrets and chimneys rising from the building. The pinnacles at each corner of the Peace Tower each have 8 columns to the stone below. According to original drawings, the interior of the pinnacle is grouted solid and has a rod inserted into the grout, and the columns are pinned at all joints, including the top and bottom of the columns. Additional pinning of the turrets was undertaken in the 1995/1996 stonework repairs, but the size and embedment of the dowels are unknown, and therefore the level of seismic restraint is unknown. There are several turrets on each corner of the East and West Pavilions. Most of these were repaired and anchored to the structure during the South façade project in the mid-1990s. The northern corners of the East and West Pavilions were not included in that repair work, and should also be anchored to the structure. Twenty-two of the chimneys have been replaced and dowels were provided between pieces of stone along with connections to the structure below, as part of the South Façade project in the mid-1990s, the Chimney Stabilization Phase 1 in 2007-2008, and the Northeast Chimney Masonry Repair project in 2009-2010. The short chimney above the House of Commons office block still requires seismic restraint. The masonry parapet walls also need to be seismically restrained. It was indicated that the parapet walls were rebuilt and pinned, but similar to other items, the details of the connections should be confirmed to be adequate for seismic loads.

Some of the more notable stonework includes the large grotesques at each corner of the towers. There are four grotesques on each water or ventilation tower on the north side of the Centre Block, in addition to four grotesques on the Peace Tower just below the Observation Deck. During upgrades to the structure of the ventilation towers, the grotesques were not specifically anchored to the structure; however, the stonework was generally connected together and anchored to the structure. The upgrades to the water towers specifically included anchorage of the grotesques as well as anchorage of the stonework. The grotesques in the Peace Tower are clamped to the ends of steel beams which were originally running diagonally across the Tower and are currently connected to perimeter plate girders, and may be sufficiently restrained for seismic loads. The condition of the steel connections should be checked, as drainage issues have been previously noted in the Peace Tower and water towers, and corrosion of the connections may lead to a need to replace the connections. There are also smaller grotesques and statues: one soldier statue on each face of the Peace Tower, three small grotesques on the buttresses at each corner of the Peace Tower just above the top of the bells, and two grotesques on the south face of the Centre Block. The grotesques and statues on the Peace Tower were reinforced with epoxied dowels during the 1995/1996 stone repairs. One of the two grotesques on the south face of the Centre Block ("Fuller's Gargoyle") was reinforced. For all of these upgrades, the design load was not given so it is not known if the repair method would be sufficient for seismic loads. This should be confirmed during the detailed design of seismic upgrades.

There are a variety of lights and cameras located around the exterior of the building. Approximately 20 lights/cameras and four fire hose boxes were observed that are connected to the stone walls. It appeared as though the lights and cameras were bolted through holes in the stone and connected to steel framing on the inside of the walls, where it was possible to view the connection. This should be investigated further, in addition to confirming that the steel framing and the stone wall are sufficient for the seismic loads. Some of the lights are likely to be exempt from seismic restraint due to the weight; however, it is recommended that they still be restrained due to the hazard they pose to people exiting the building. Conduit that is running up the side of the building or Peace Tower is exempt due to its weight, and would pose a minimal hazard to egress. Approximately 25 ballasted stands were provided for additional lighting, as well as a few lights connected to stands that were sitting on top of the roof paving stones. These stands cannot be considered as seismically restrained according to the 2010 NBCC, as frictional resistance cannot be relied upon during seismic events due to the potential for vertical motion in an earthquake, and should be connected to the structural roof elements.

Cladding should be connected to the structural walls. The stone masonry around the stairwells at the north and south ends of the House of Commons (West) Courtyards have been repaired and anchored to the structural back-up walls. The Senate (East) Courtyard walls at the north and south stairwells have been anchored to the back-up wall only at the corners. The Peace Tower cladding on the North elevation was significantly repaired and anchored to the concrete structural members; however, it was indicated in the report that additional work would be required and that maintenance to the stonework should be provided at regular intervals. Repairs to stone masonry should continue as required where stones are broken or mortar has deteriorated, as loose stone provides a hazard to people exiting the building during a seismic event, particularly around exits.

### 8.3.2 Centre Block Interior

Items reviewed inside the main building include the ceilings, decorative arches, lights, partition walls, and mechanical and electrical equipment.

a) Terra Cotta Tiles

The floor structure of the Centre Block is typically comprised of flat terra cotta arches supported by structural steel beams, covered with a cementitious topping. Lateral spread of the beams due to the arching forces is prevented by steel tie rods between the beams. Additional terra cotta tiles are used on structural elements such as steel beams as a fireproofing method. The fireproofing terra cotta tiles can be a hazard in a seismic event if they separate from the structural element and damage the ceilings or injure people. Where the fireproofing tiles have already fallen off, the adjacent tiles are more likely to fall off due to the lack of friction from the adjacent mortar joint.

b) Ceilings

In the Centre Block, the ceilings are generally plaster. Some ceilings are flat and the plaster is applied directly to the underside of the terra cotta flat arches. Where the plaster is applied directly to the underside of the terra cotta bricks, seismic restraint is not required. This is believed to be the case in most offices and hallways above the first floor (with some exceptions near the House of Commons and Senate chambers).

Other plaster ceilings are suspended from the beams above, and are either flat or arched. The suspended ceilings are sometimes indicated on the original architectural drawings as being composed of Hy-Rib steel forms, with a plaster layer on the bottom and a cementitious fill layer on the top. The Hy-Rib spans between the supporting frames which are hung from hanger rods clamped to the bottom flanges of the steel beams supporting the floor above. The supporting frames vary, with steel tees, single or double angles, and reinforcing bars being used to form the arches or connect the arches. Other suspended ceilings may not have Hy-Rib forms where it is not indicated on the architectural drawings, but are assumed to be constructed in a similar manner with metal lath or a related material providing the form for the plaster and cement fill layers, as was observed in the attic above the entrance to the 6$^{th}$ floor restaurant. In the larger offices in the corners of the pavilions, there are shaped plaster ceilings (some shaped and painted to look like wood), which are similarly suspended. The distance between the terra cotta arches and the plaster ceiling varies, with hanger lengths up to 900mm shown on the original drawings at the lower edges of the arches. The plaster ceiling in some locations is exempt from restraint due to the fact that it is connected to walls on all sides, as indicated on the original drawings. Locations where the ceiling is exempt include the House of Commons Reading Room and the Government Caucus Room. Many of the hallways and narrow rooms or offices beside the House of Commons or Senate chamber (such as the Parliamentary and Opposition lobbies on the East and West sides of the House of Commons chamber) only have connections on two sides of the room.

In the House of Commons chamber, the ceiling is suspended fabric which is connected to a frame that is hung from girders at the floor above. The connection to the frame is unknown and may be sufficient for the seismic loads, but the frame itself should be seismically restrained. The catwalk just below the ceiling is also built as a frame which is suspended from the girders above, and should be restrained as well or sufficiently connected to the walls on all sides.

In the East and West Pavilions and in the restaurant on the Sixth Floor, there are “skylights” which are lit by electrical lights from above. These are suspended from the beams above. Original architectural drawings show hangers cast into the reinforced concrete beams which support the glass panels. There are also metal skylight frames at the northern end of the building. The primary concern for the skylights is glass breaking and injuring people. If the supports move differentially from glass, this may cause the glass to break.

The catwalks around the West Pavilion and restaurant skylights are supported by additional steel beams framing between the columns and suspended from the steel trusses above, which may be sufficiently connected for seismic loads. The capacity of the catwalk connections should be confirmed for vertical uplift loads.

There are suspended drop-in tile ceilings in large portions of the ground floor and parts of the basement. During the site visits, ceiling tiles in several areas on the ground floor and basement were raised to examine the typical connections. Seismic restraint was not seen in any of the locations that were observed. In addition to these locations, in the cafeteria on the fifth floor, there is a combination of drywall and ceiling tiles, with drywall bulkheads around ducts and concrete beams. The connections of the suspended ceiling structure were not observed in the cafeteria area, but it is assumed that these ceilings are also not restrained. A few of these areas may be exempt if they are less than $13.4m^2$ and span between walls that are connected to the structure above; however, this should be confirmed in the detailed design phase.

c) <u>Decorative Arches</u>

Decorative arches are a part of the ceiling structure in certain locations, and can be either discrete arches located in the hallways on the third floor and below, or arcading in important areas, such as the entrance to the Memorial Chamber, the Hall of Honour, Confederation Hall, and the main visitor’s entrance in the basement.

The discrete arches are built on a frame suspended from rods, and are suspended with the arched plaster ceilings. These are only designed for vertical support. Some restraint for out-of-plane lateral loads would be provided by the ceilings on each side of the arch, and for in-plane lateral loads by bearing on the walls on either side of the arch.

Other arches, such as the arcading in the entrance link to the Memorial Chamber, the Hall of Honour, Confederation Hall, and in the main visitors’ entrance in the basement, appear to be built solely of stone and are gravity supported without connection to the surrounding structural elements. Some restraint for lateral loads would be provided by friction between the blocks, self-weight of the arches, and architectural features (such as walls) on each side of the arch. The arcading in the Senate Entrance Hall has dowels through the columns, providing additional resistance. Dowels were not indicated in any other location and are not believed to be present throughout the building. The arcading in the entrance link to the Memorial Chamber, the Hall of Honour, Confederation Hall, and in the main visitors' entrance in the basement should be reinforced and the ceiling between them should be restrained.

d) Walls

There are a variety of wall types in the Centre Block. Load-bearing walls that contribute to the seismic resistance of the building will not require seismic restraint. Original partition walls are typically hollow terra cotta brick, covered in plaster. It is believed from the original specifications that they are built tight to the underside of the terra cotta flat arches above. It is recommended that investigative openings be made to confirm this prior to designing the restraint system. There are some drywall, concrete block, and brick partition walls, mostly noted in the basement or in the attic space below the sloped roof, but also present in other locations, that did not extend the full height of the floor. These walls should also be restrained, although the method of restraint will be different from the full-height partition walls. There is a wall on the east side of the catwalk around the House of Commons chamber that has frequent openings at midheight of the wall, and did not extend the full height to the floor structure above. It will require restraint for both the top and bottom sections of the wall.

Elevator shaft walls made of unreinforced masonry are also of concern for seismic restraint. Elevators should not be used for egress in case of an earthquake, although there may be people already inside the elevator. If the walls are damaged, then the elevator may not be operable after the earthquake.

e) Decorative Items

Inside the Centre Block, there are several sculptures on stone bases located throughout the building on the lower levels. Many of the sculptures appear to be connected to their bases, although the height of the sculptures means that they may be prone to overturning. The connection between the sculpture and the base, the connection between the base and the floor structure, and the sculpture's weight should be confirmed. Other carvings, as per the original architectural drawings, appear to be attached to the ceiling by tongue-and-groove slots. This includes two large murals carved into the stone near the link to the library building. Other sculptures are carved into the stone walls or columns. The ones that are part of a load-bearing wall or column do not need to be restrained, and original architectural drawings show dowels from the column through the capital in a few locations. The carved stonework in the House of Commons Chamber at the balcony level is anchored to steel framing of the balcony, according to the original drawings.

The wall-mounted letter chute, which extends the height of the building, will likely require seismic restraint due to its weight

f) Light fixtures

Light fixtures vary throughout the building, from fluorescent lights screwed into the ceiling to elaborate chandeliers suspended on long hanger rods from the structure above. Large chandeliers were noted in the lobby of the dining room on the Sixth Floor, six chandeliers in the House of Commons chamber, and two in each of the Reading Room, Railway Committee Room, and Commonwealth Room. Smaller suspended lights are on the lower levels, with approximately 50 being noted on the Second Floor, several in the Kitchen on the Sixth floor and in the balcony overlooking the House of Commons chamber. These suspended lights will need seismic restraint if they weigh more than 9kg or are on a hanger that is sufficiently long that the light will be able to impact other items, break, and injure a person who is attempting to exit the building. This should include most of the chandeliers described above. Additionally, any suspended lights that are of particularly high historic value which are desired to be protected against breakage should be restrained as a cautionary measure.

The lights that are screwed into the structure above are exempt from seismic restraint requirements. The lights which are connected to columns or walls, including those connected above each column in the House of Commons chamber, are likely sufficiently lightweight so that restraint will not be required. In addition, most of the observed lighting fixtures in the offices and other hallways are connected directly to the drywall or drop-in tile ceilings, and will likely not need to be restrained separately from the ceiling structure. The lights connected directly to these ceilings would only need a slack cable for backup support, unless they weigh more than 4.5kg. This is not the case for the observed lights, although there may be others that were not observed and are heavy enough to require restraint.

g) Mechanical and electrical equipment

Mechanical and electrical equipment were reviewed; however, it is understood that mechanical and electrical systems will be removed during upcoming renovations. This includes communication systems, most of which were not observed in either the Centre Block or Peace Tower. All new mechanical and electrical equipment that is installed during renovations is required to be seismically restrained according to the 2010 NBCC, but old equipment that remains does not require restraint as long as damage to the systems can be tolerated.

Mechanical equipment located throughout the building was generally not restrained. Wall-mounted equipment, small diameter conduit connected directly to the slab above, small diameter piping, and most ductwork may be exempt from restraint requirements. Piping that distributes water throughout the building would be recommended to be restrained even if the existing system is left in place, due to the potential damage that the pipe breaking could cause for the historic features of the building.

Distributed ventilation was limited throughout the building, and most ducts are exempt due to their low weight. Only large ducts, such as those observed in the mechanical rooms and the kitchen, would require restraint.

A few pipes throughout the building and most pipes in the mechanical rooms will require restraint. In mechanical rooms, the diameter of pipe that requires restraint decreases due to the density of items leading to an increased likelihood of pipes impacting each other. Small diameter pipes were observed in the bathrooms, in some towers, and in mechanical rooms. Larger pipes require restraint; pipes that would require restraint were observed in the basement mechanical area, the kitchen on the Sixth floor, and the mechanical room on the Fifth floor near the Cafeteria. In the balcony above the House of Commons chamber, a mechanical shaft was created along the east side of the room. There are trapezed pipes suspended from the ceiling that would require restraint. Piping in the ventilation shaft in the northwest corner of the building on the Fifth floor would likely require restraint; however, cracks in the wall may mean that the walls would need reinforcement in order to be capable of supporting the seismic load.

In some of the mechanical areas, ductwork and trapezed pipes were supported on posts cast into the slab below, and may be exempt from seismic restraint requirements. A water tank in the shower in the basement was sitting on a wooden platform, and did not appear to be connected to the platform in any way. An air handling unit in one of the towers was connected to supporting beams which are supported on pockets in the stone walls of the tower, which may require seismic restraint. Elevator machinery was sitting on the roof slab and restraint should be provided. Vibration isolated equipment is not seismically restrained and the connections which dampen vibrations can increase the seismic load, so most vibration isolated equipment is not sufficiently seismically restrained. A suspended, vibration isolated air handling unit was located outside the catwalk around the House of Commons chamber, and there are some base-mounted, vibration isolated units in the basement. The kitchen equipment did not appear to be seismically restrained, and should generally be restrained based on their weight or height.

Electrical equipment (including cameras, televisions, speakers, and similar items) are located throughout the building, and varied in the connection from suspended, wall-mounted, and sitting on the slab. The cameras in the House of Commons chamber, which are suspended from the balcony, may be exempt from seismic restraint due to their weight. Televisions mounted on the walls of the building should be restrained. Speakers in the window wells of the Railway Committee Room should be restrained due to their height above the ground. Wall-mounted electrical boxes should be sufficiently connected that additional seismic restraint will not be necessary; however, some boxes were sufficiently wide (as measured perpendicular to the wall) and others may be sufficiently heavy, that any electrical boxes that are going to remain should be reviewed during the renovations.

Most of the large mechanical units are located in the basement, along with significant amounts of distribution systems. There were approximately 60 large mechanical and electrical units (including pumps, motors, air handling units, transformers, and cable towers) sitting on the slab, and none of the units appeared to be seismically restrained. The entire north end of the basement was not visible due to the density of piping and ductwork, including a large duct running the length of the north corridor. It is assumed that nothing in the north section of the basement has been restrained except for the large cable tray, as was typical throughout the rest of the basement. Other rooms were locked and not available for review, including the Hydro vault. Additional equipment in the locked rooms should be considered when determining the quantities of equipment requiring restraint. One large cable tray in the basement was seismically restrained. Some of the smaller pipes, ducts, and conduit would be exempt. Significant portions of the lighting would also be exempt as they are lightweight and attached directly to the slab above. There are a lot of pipes and ducts in the basement that would require seismic restraint, but the density of the equipment would make typical restraint systems difficult to install.

Elevator equipment is also planned to be replaced during the upcoming renovations. The existing equipment is generally base-mounted, and where indicated on the drawings, is sufficiently heavy that the connections will need to be reinforced for seismic loads. When new elevators are installed, seismic requirements should be considered for all new equipment.

#### 8.3.3 Peace Tower Interior

There were suspended drop-in tile ceilings located at several of the lower levels of the Peace Tower. The area for each individual floor is less than $13.4m^2$ and the ceiling spans between walls, and therefore the ceilings are exempt from restraint. The decorative LeafLite ceiling at the observation deck level is suspended from slab above with hangers located along the framing, but is not laterally restrained for seismic loads. Due to the high concentration of visitors to the observation deck and the fragile material, this ceiling should be restrained to prevent injury if it impacts other items or the walls and breaks. In addition to the ceiling, the lights in the Observation Deck should be restrained due to their long hanger length and their potential for breakage during a seismic event.

The clock face in the Peace Tower should be connected to the masonry walls to seismically restrain it. The adequacy of the existing bolts into the masonry should be confirmed for seismic loads. The flagpole above the Peace Tower should also be confirmed to be adequately restrained for seismic loads according to the 2010 NBCC. The flagpole baseplate is bolted through sleeves in concrete beams in the Peace Tower. The bell support frame details and connections are also unknown, and should be confirmed for seismic loads.

It was assumed that the mechanical and electrical systems in the Peace Tower will also be removed during the building renovation. Most of the suspended items that were observed were small in cross-sectional area and would have been exempt from seismic restraint requirements, with the exception of some of the piping. The electrical conduit, which was generally bolted directly to the wall or ceiling, and the electrical panels and other small wall mounted boxes are also likely to be exempt due to their size. Elevator equipment, such as the motor, were observed to be base mounted and were not seismically restrained. Elevator equipment is typically sufficiently heavy that restraint will be required. Some of the wall-mounted elevator control panels were likely to be exempt due to their weight.

Some items, such as wall-mounted electrical items, were assumed to be restrained due to previous experience designing seismic restraint for similar items; however, without knowledge of the weight of the items or the actual fastening pattern, it is not known for certain that the items are restrained. In any case, it is assumed that all mechanical and electrical items, particularly in the basement, would be removed and replaced during upcoming building renovations. This includes communications equipment, some of which was not available for observations. If any items are to remain, the seismic restraint should be examined and addressed as required.

## 9. ANTICIPATED 2015 NBCC LOADS COMMENTARY

The proposed 2015 NBCC draft version was released in 2014 for public comment. The seismic hazard data for the Ottawa region contains some significant changes. Table 9.1 below compares the 2010 NBCC and 2015 NBCC Ottawa seismic hazard data.

Table 9.1: NBCC Values for 2% Probability of Exceedance in 50 Years

| Spectral Acceleration Values | | | | |
|---|---|---|---|---|
| | $S_a(0.2)$ | $S_a(0.5)$ | $S_a(1.0)$ | $S_a(2.0)$ |
| 2010 NBCC | 0.64 | 0.31 | 0.14 | 0.046 |
| 2015 NBCC (Anticipated) | 0.439 | 0.237 | 0.118 | 0.056 |

As illustrated in Figure 9.1, it is anticipated that the 2015 NBCC will decrease the seismic hazard for short and medium period structures (T < 1.0 seconds) in Ottawa by approximately 30%.

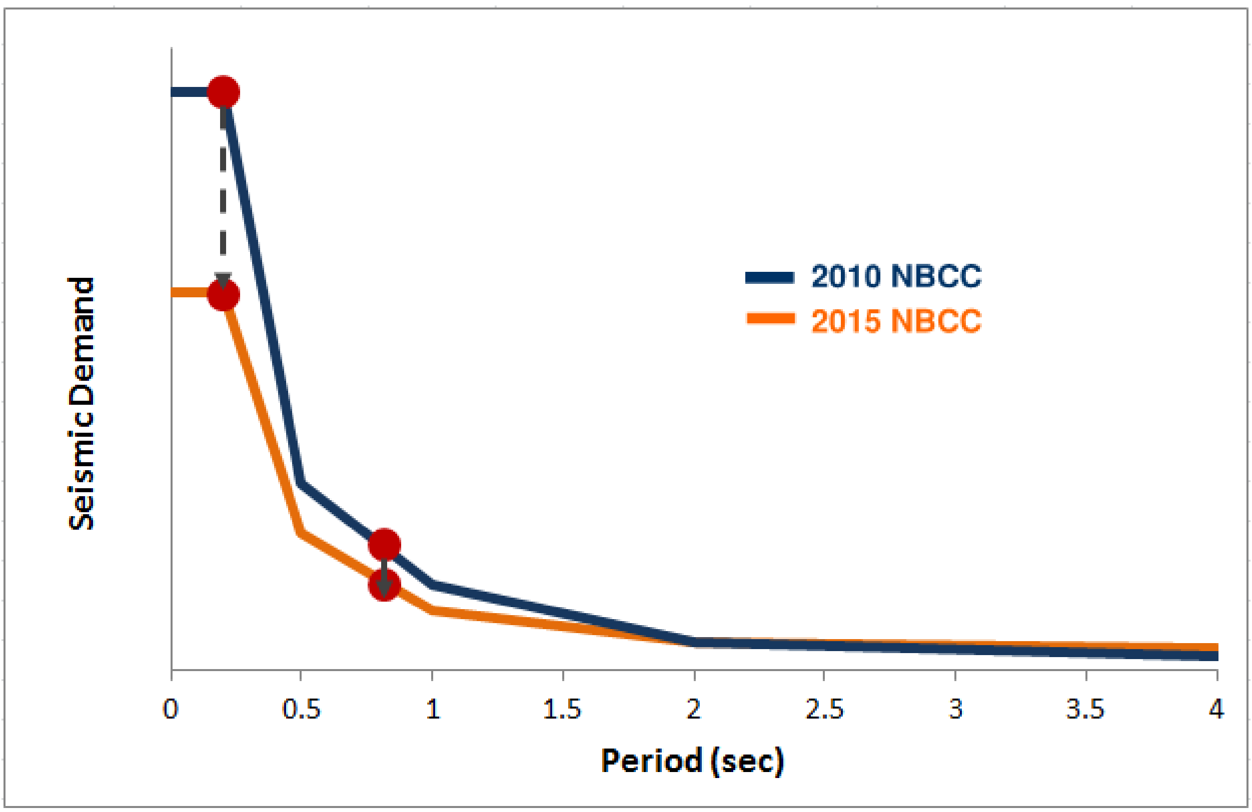



Figure 9.1: Comparison of the 2015 and 2010 NBCC Seismic Hazards (Ottawa)

The reduction in 2015 NBCC seismic hazards will have the following effect on the Centre Block component capacity/demand ratios:

a) The masonry wall capacity/demand ratio's illustrated in Figure 6.4 and Figure 6.5 will typically jump to a higher category. The upper stories will still have a significant portion of walls with a capacity/demand ratio of less than 30%. In the lower stories, the majority of the walls will likely have a capacity/demand ratio of 60% or better with the exception of the south corridor wall which will be at approximately 30%.
b) The plain unreinforced concrete basement walls will have their capacity/demand ratio improve to 80% and 130% in the East-West and North-South Directions respectively.
c) The capacity/demand ratio for the terra-cotta flat arch floor diaphragms will increase to approximately 25%.

The reduction in 2015 NBCC seismic hazards will have the following effect on the Peace Tower component capacity/demand ratios:

a) The spandrel beams over the memorial chamber on the south, east and west faces will have less than 60% of the required capacity to resist the 2015 NBCC seismic loads.
b) The tall piers on either side of the main opening of the north face will have less than 60% of the required capacity to resist the 2015 NBCC seismic loads.
c) The tall piers that frame the large opening above the Memorial Chamber on the south, east and west faces, as well as the spandrel beam above them, will have between 60 and 100% of the required capacity to resist the 2015 NBCC seismic loads.
d) The Upper Tower will have significantly less than 60% of the required capacity to resist the 2015 NBCC seismic loads.

The reduction in 2015 NBCC seismic hazards will have the following effect on the non-structural and secondary structural components:

a) The seismic hazard index, $I_E F_a S_a(0.2)$, is expected to drop below the threshold value of 0.35.
b) If the seismic hazard index drops below a value of 0.35, then the only features that will require seismic restraint are the partition walls, parapet walls, chimneys, grotesques, and other similar features. The suspended ceilings, lights, masonry veneer, and all mechanical and electrical equipment will no longer require seismic restraint.
c) Seismic restraint may still be desirable to protect historic architectural features or prevent hazardous materials from being emitted by broken mechanical systems.

# 10. CONCLUSIONS

## 10.1 Gap Analysis

a) There are significant gaps in the information available on the Centre Block that affect the results of a seismic analysis. The key gaps relate to: areas where the geometry or connections of the structure are not clear, unknown material properties, particularly of the masonry assemblies, final geotechnical information that was not available, primarily the site classification, and configurations and connections of the secondary- and non-structural components.

b) The gaps in information are important to address because these parameters either affect the loads imposed on the structure by affecting the mass or stiffness of the building, or by affecting the strength of the components to resist seismic loads.

## 10.2 Centre Block Seismic Analysis Conclusions

<u>Overall</u>

a) The structure's seismic force resisting load path has multiple elements, all with varying capacity/demand ratios. The weakest link is typically the connection of the floor diaphragms to the walls. In many places this is either very limited or non-existent. Assuming this is corrected, the worst case diaphragms (the longest span diaphragms– See section 6.8.2) have the capacity to resist between 2% to 20% of 2010 NBCC seismic loads. See points b) to e) below for specific conclusions regarding the floor/roof diaphragms. The worst case walls can resist between 11% to 15% of 2010 NBCC seismic loads. See points g) to k) below for specific conclusions regarding the wall capacities.

<u>Horizontal SFRS – Floor Diaphragms</u>

b) The capacity of the Centre Block's floor assemblies to act as structural diaphragms and transfer floor plate inertia forces generated by seismic shaking to the lateral load resisting walls is typically very limited.

c) The weak cementitious floor topping provides a diaphragm span capacity of approximately 2% to 11% of that required to resist 2010 NBCC seismic loads.

d) The capacity of the floor structure to provide diaphragm span capacity using rigid arching through the infill terra-cotta floor unit is estimated to be no more than 20% of the 2010 NBCC seismic demand level.

e) The only existing mechanism for transferring seismic loads from the floor plates to the lateral load resisting walls is through bearing of the pocketed floor beam webs on the masonry walls. At many locations there is no connection of the floor diaphragm to the walls at all.

Note: the above comment is referring to seismic load transfer only. The floor structure as detailed on the original structural drawings provides an adequate load path for gravity loads.

f) The capacity of the multi-storey sloped roofs to provide lateral restraint to portions of both the Main Roof structure and the 6th level floor structure is negligible.

Vertical SFRS – Masonry Walls

g) None of the masonry walls above Level 3 have sufficient capacity to resist full 2010 NBCC seismic loads. A range of capacity/demand ratios exist with approximately 20% of the walls possessing less than 30% of the capacity required to resist 2010 NBCC seismic loads.

h) Some of the masonry walls in the lower levels (Level 1 and Level 2) can resist full 2010 NBCC seismic loads. The weakest walls are the south corridor walls which have a capacity of less than 30% of that required to resist 2010 NBCC seismic loads.

i) The unreinforced concrete basement walls have approximately 55% and 90% of the capacity required to resist full 2010 NBCC seismic loads in the East-West and North-South directions respectively.

j) The masonry walls generally have adequate capacity to resist out-of-plane seismic loads. The connection of the masonry walls to the floor structure is also typically adequate to provide out-of-plane restraint to the masonry walls. A localized wall area on the East and West facades of the pavilion structures that cantilevers from the sixth floor level is inadequate and requires remedial work.

k) The reduction in seismic hazard for the Ottawa area anticipated in the 2015 NBCC may significantly improve the calculated structural component capacity/demand ratios, however, numerous walls will still have less than 60% of the capacity to resist full 2015 NBCC seismic loads.

### 10.3 Peace Tower Seismic Analysis Conclusions

a) Many of the Peace Tower's piers and spandrel beams have insufficient capacity to resist either 60 or 100% of the 2010 NBCC seismic loads. The weakest links occur at the spandrel beams above the Memorial Chamber on the South, East and West faces and the piers on either side of the main opening on the North face.

b) The tower structure at the observation deck level, clock face and sloped roof have insufficient capacity to resist either 60 or 100% of the 2010 NBCC seismic loads.

c) The reduction is seismic hazard for the Ottawa area anticipated in the 2015 NBCC may improve the calculated structural component capacity/demand ratios but several piers and spandrels, as well as the Upper Tower, will still have insufficient capacity to resist either 60 or 100 % of the 2015 NBCC seismic loads.

### 10.4 Non-structural and Secondary Structural Components

*a)* Seismic restraint was generally not observed for architectural, mechanical, or electrical components in the Centre Block, with the exception of a large cable tray in the basement.

*b)* Existing items do not require seismic restraint according to the NBCC 2010. It is recommended, however, that historic architectural components such as interior finishes, chandeliers, and large stonework should be restrained to protect the assets from damage. It is also recommended that items that could impede egress from the building in the event of an earthquake be restrained to prevent injury to people trying to exit the building.

c) Any items that are replaced during the renovations will require seismic restraint according to the building code in effect at that time. This is likely to include mechanical and electrical equipment, and may also include interior partition walls that are moved to reconfigure the office layouts.

d) Some lightweight objects may be exempt from restraint requirements. This is expected to include typical office light fixtures, small wall-mounted electrical items, and small-diameter conduit, among other things.

e) Items that require seismic restraint and have positive connections to the structure, such as the clock faces or the reinforced chimneys, should have their connections assessed to determine if they are adequately restrained for the required seismic loads.

f) Requirements for seismic restraint of non-structural components are expected to decrease in the upcoming NBCC 2015.

APPENDIX A: PHOTOGRAPHS OF EXISTING CONNECTIONS FOR SEISMIC RESTRAINT

Figure A.1. Typical exempt wall or ceiling mounted conduit

Figure A.2. Typical exempt wall mounted unit

Figure A.3. Typical condition of roofing on the flat roof of the Centre Block

Figure A.4. Typical connections to the copper roofing in the Peace Tower

Figure A.5. Typical welded copper roof ornamentation

Figure A.6. Brick cladding at lightwells requiring seismic restraint

Figure A.7. Typical corroding connections into the wall on the exterior of the Centre Block

Figure A.8. Typical ballasted frame on the roof of the Centre Block which will require restraint

Figure A.9. Connection of fireproofing terra cotta tiles to the bottom flange of the steel beam

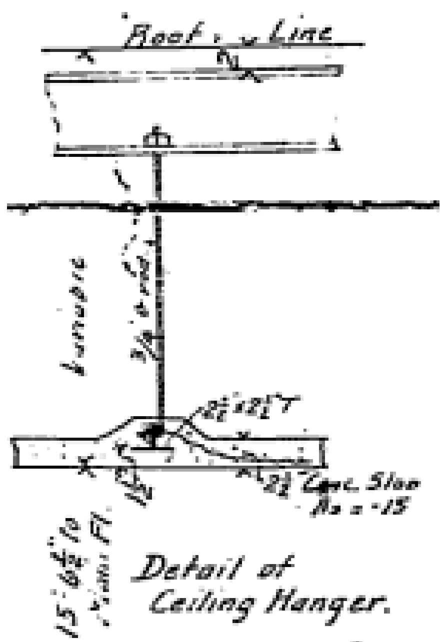



Figure A.10. Detail of ceiling hanger from original architectural drawing 131b

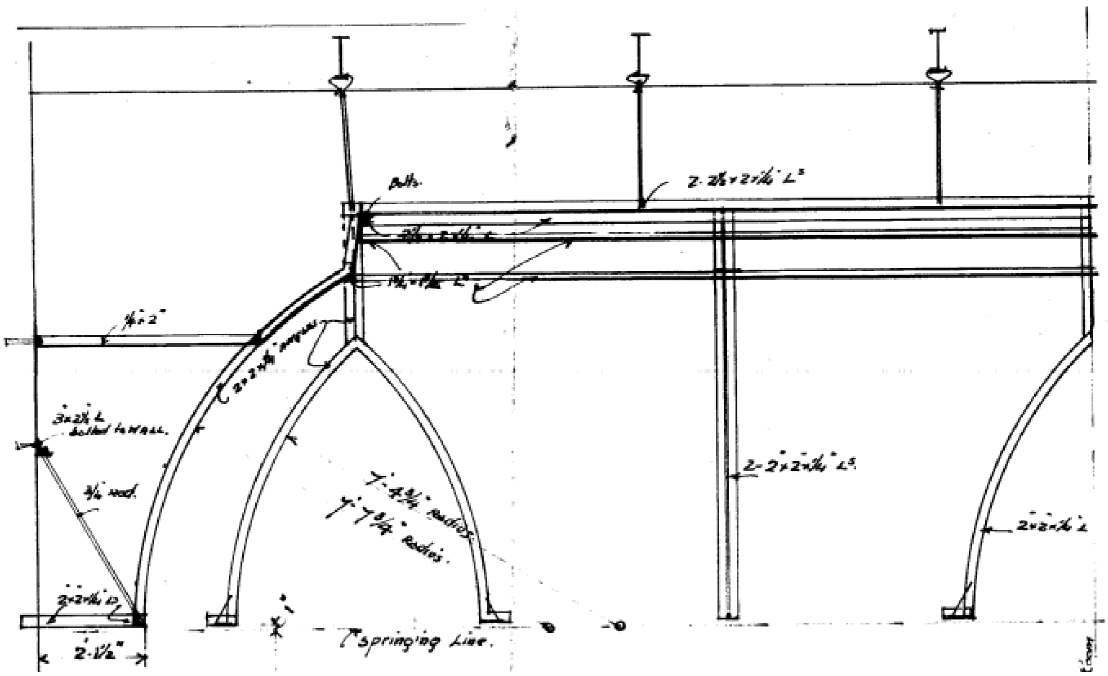



Figure A.11. Details of the framing supporting the ceiling at the north end of the Government Caucus Room, including connection into the wall, from original architectural drawing r622 (original drawing number 1149 dated April 18, 1919 by John Pearson Architects)

Figure A.12. Support for suspended arched ceiling over the hallway.

Figure A.13. Typical clamp supporting the ceiling hanger rod, connecting to the bottom flange of the steel beam

Figure A.14. Typical hanger rod, embedded in the cementitious material above the ceilings

Figure A.15. Connection of attic catwalk around the skylights

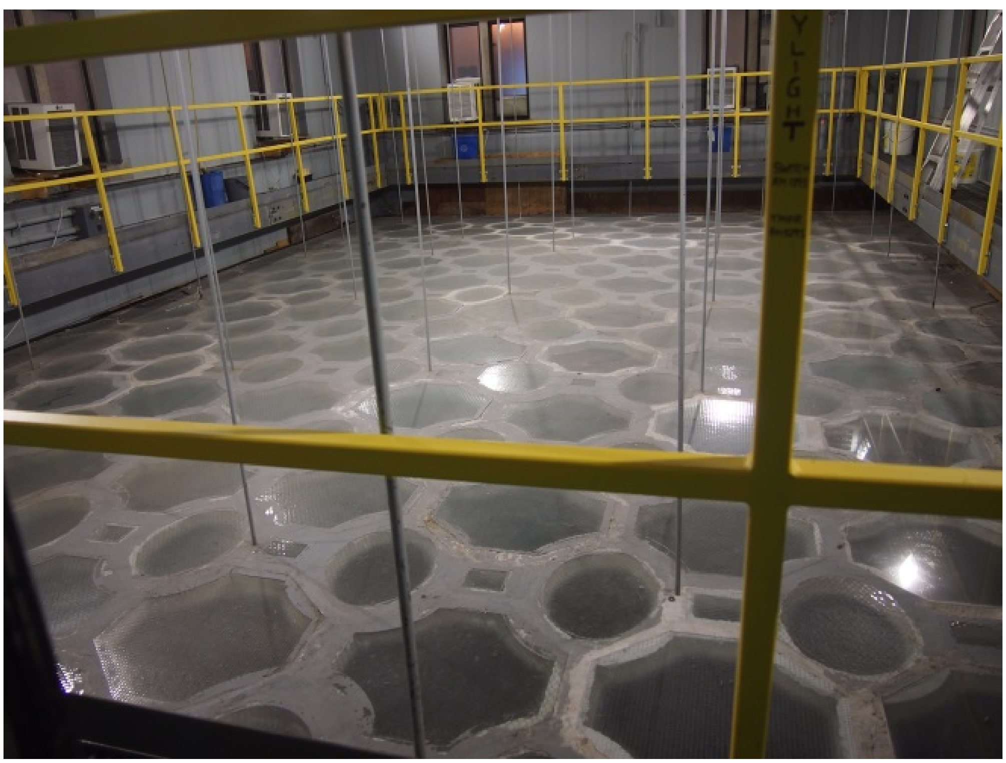


Figure A.16. Suspended lightwell over the entranceway to the House of Commons

Figure A.17. Suspended catwalk around the House of Commons chamber

Figure A.18. Typical drop-in tile ceiling in the ground floor hallways

Figure A.19. Decorative arch suspended with the arched plaster ceilings


Figure A.20. Arcading in the Visitor's Entrance

Figure A.21. Partial height terra cotta partition wall requiring seismic restraint

Figure A.22. Base mounted statue requiring seismic restraint

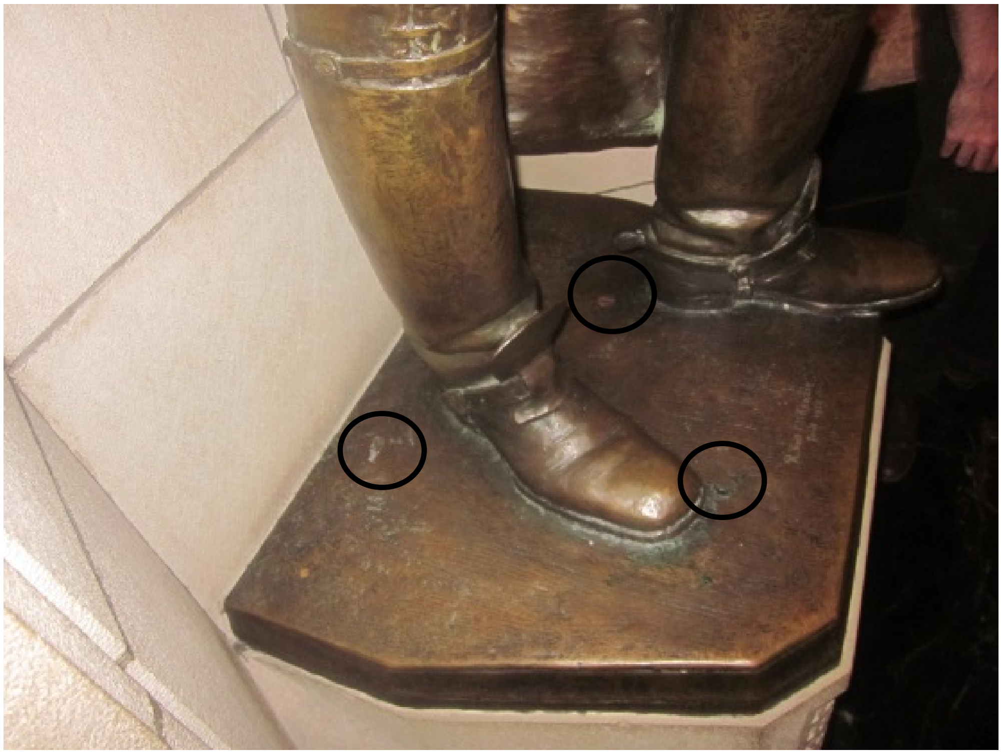


Figure A.23. Nominal connections between the statue and the base


Figure A.24. Wall mounted letter chute requiring seismic restraint

Figure A.25. Large chandelier requiring seismic restraint

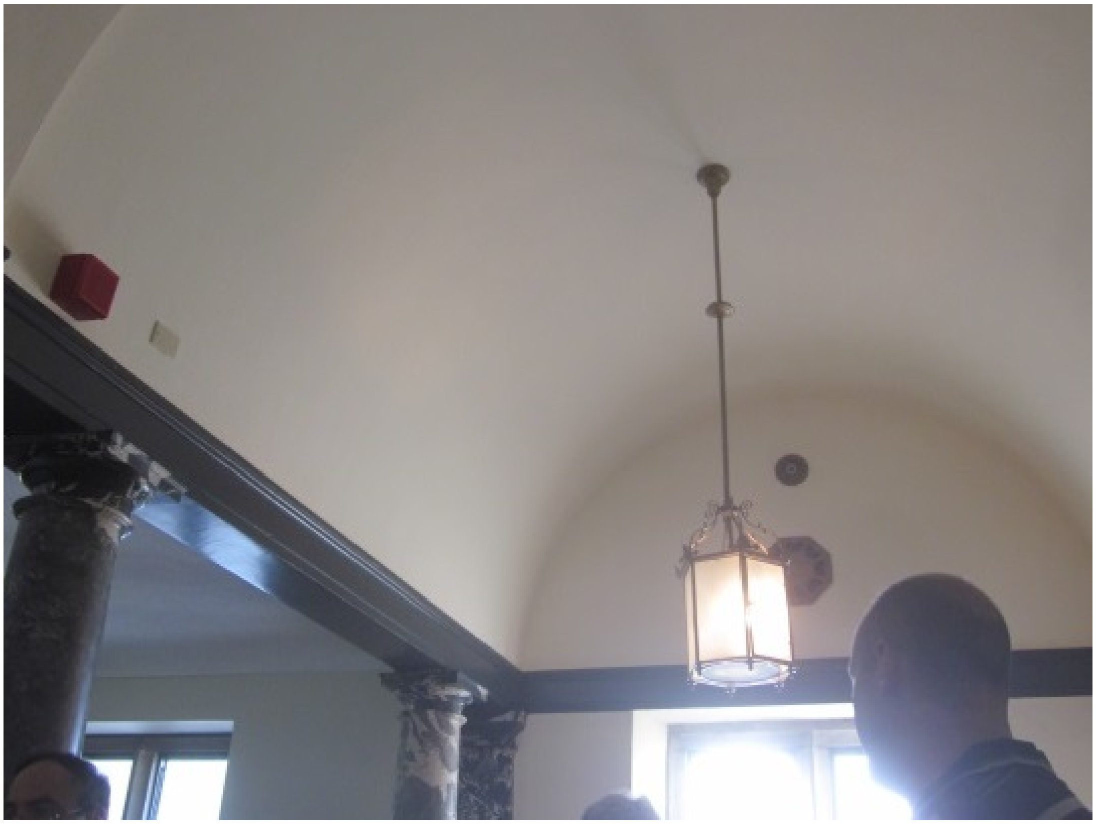


Figure A.26. Long hanger on a light fixture which will require seismic restrain

Figure A.27. Typical exempt light fixture in the offices and hallways on the upper floors

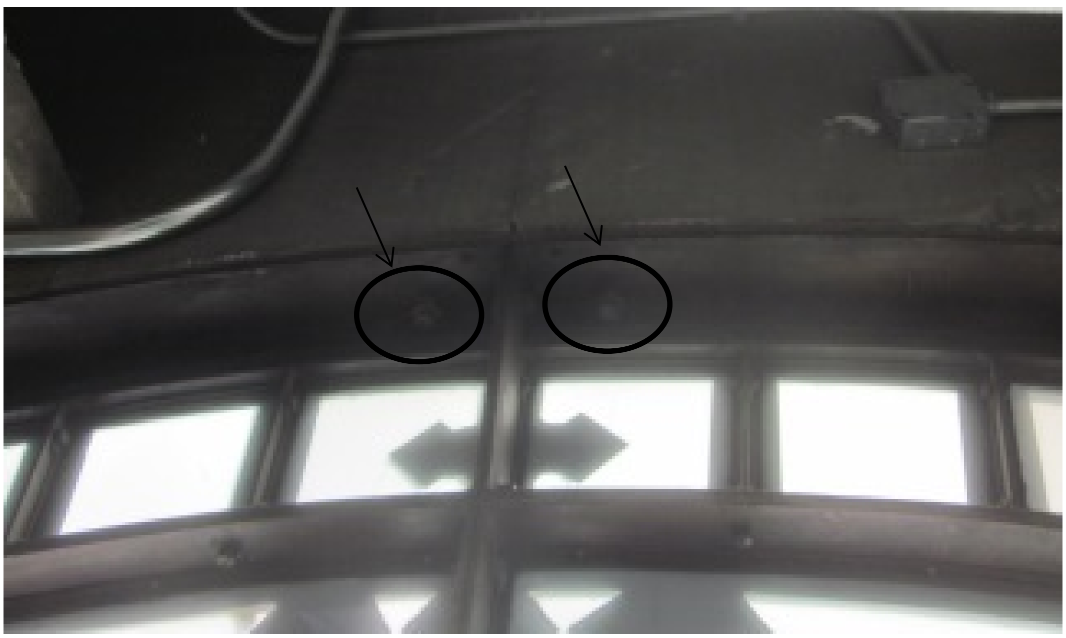


Figure A.28. Connection from the clock face in the Peace Tower to the surrounding concrete wall

Figure A.29. Restrained cable tray in the basement