SEISMIC RESEARCH STUDY FOR CENTRE BLOCK, PARLIAMENT HILL

Work Package 1

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

Prepared by:
Halsall Associates
210 Gladstone Avenue, Suite 4001
Ottawa, Ontario
K2P 0Y6
(613) 237-2462

14Y160-113A

February 2015

Halsall

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210 Gladstone Avenue, Suite 4001
Ottawa, ON K2P 0Y6
613-237-2462
Attn: Dan Carson, P.Eng.

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TABLE OF CONTENTS

LIST OF APPENDICES

LIST OF ACRONYMS

DGM	Design Ground Motion
FEMA	Federal Emergency Management Agency
FRP	Fibre Reinforced Polymer
NBCC	National Building Code of Canada
PGA	Peak Ground Acceleration
PWGSC	Public Works and Government Services Canada
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 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. Halsall was also engaged to complete a 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.

This report presents the research of seismic upgrade options of heritage masonry structures.

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 floor system of the upper floors 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 Methodology

Existing documentation and drawings on the Centre Block were reviewed and site visits were made to the building to visually assess the structure. Based on this information, targeted research was then completed on similar projects identified in relevant publications, such as FEMA 274: NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings, earthquake conference proceedings, and from the knowledge of team members on the project.

1.4 Seismic Upgrades of Heritage Structures

Detailed results of 11 seismic retrofit projects that are similar in importance and construction to the Centre Block are presented in the report. Based upon the literature reviewed, adding reinforced concrete shear walls, steel braced frames or base isolators, or some combination of these methods, are the most common seismic upgrade schemes.

Concrete shear walls and steel braced frames have been proven to be cost effective, and are well understood within both the design and construction communities. These methods have been effectively applied within the Ottawa region on many seismic retrofit projects. However, these methods generally require a large degree of interference to the existing structure as they take up physical space within the building and must be anchored to the existing structure.

Base isolation (also referred to as seismic isolation) is becoming more common as a seismic retrofit method because it protects heritage buildings in two ways. Firstly, it reduces the level of intervention to the building during construction by typically concentrating the required construction in the basement spaces. Secondly, it reduces the risk of damage to both the structural and non-structural components during an earthquake by significantly reducing the forces the building experiences. In most seismic retrofit projects that have included base isolation, there is still a need for a reduced amount of interference in the building, such as strengthening the floor and roof diaphragms, introducing a limited scope of steel bracing or shear walls, or improving connections and load path.

Friction and viscous dampers were reviewed and found to not typically suit very stiff brittle masonry wall structures. Adding tensile and shear capacity to the unreinforced masonry has been achieved on limited projects with drilled steel anchors and fibre reinforced polymer (FRP) fabrics, but have been generally limited in their usefulness, especially for heritage structures.

1.5 Understanding Performance Levels

A building that is designed with conventional method to resist 100% of the 2010 NBCC seismic loads is still expected to be damaged in the design level earthquake. This includes damage to both structural and non-structural components and the building may not be useable after the earthquake.

The performance level specified by the code for normal buildings is to protect the life safety of the building occupants and the general public. For the post disaster buildings, the intent is to provide a high likelihood that the building will remain functional after the earthquake, although damage is still expected.

Base isolation can offer a higher performance level by providing life safety, building functionality after an earthquake, and greater protection from damage for both structural and non-structural components.

2. INTRODUCTION

2.1 Scope

In July 2014, Halsall Associates was engaged by PWGSC to examine potential options to seismically rehabilitate heritage buildings based on work done on buildings that are of a similar scale, type and importance as the Centre Block through targeted research at an international level, and to complete a 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 RPS Policy on Seismic Resistance of PWGSC Buildings.

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 identify further investigations that would be required to obtain the missing information in order 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 of the Centre Block 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) Discuss potential seismic upgrade options for the Centre Block 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.

These studies do not include a seismic evaluation of the Library of Parliament, Centre Block Underground Services building or the underground tunnels that connect the Centre Block to adjacent structures.

This report, for Work Package 1 – Seismic Research Study, focusses on presenting the seismic upgrade research.

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 Façade 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;
- Draft Geotechnical Data Gap Analysis, Centre Block Project, Parliament Hill. Prepared by Stantec Consulting, August 2014, PWGSC File No.fe173.EP764-150225; 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 on structural steel beams, covered with a cementitious topping. 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.

2.4.3 Seismic Hazard of the Site

The 2010 National Building Code of Canada (NBCC) addresses the seismic hazard associated with strong ground shaking. The 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 provide a high likelihood that post-disaster buildings can continue to be occupied and function following strong ground shaking, though some damage can still 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).

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 2.1: 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 Fa and Fv 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 better than Site Class A, based on 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 Fa=0.655 and Fv=0.433, respectively. The resulting design spectral response acceleration values for Ottawa, Site Class A, Vs = 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 2.2: Design Spectral Response Acceleration Values (Ottawa, Ontario, Site Class A+)

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)**
0.419	0.134	0.061	0.020	0.010

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

3. METHODOLOGY

3.1 Research sources

Before beginning the research, the existing the 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. Following the documentation review, three sites visits were made to the Centre Block to conduct a visual review of the buildings heritage features and structural components where accessible.

The process of researching similar projects that have undergone seismic upgrades has involved an extensive literature review of numerous published sources. The first key sources that provided descriptions of projects in the United States that had be seismically retrofitted were reports prepared by the Federal Emergency Management Agency (FEMA), such as FEMA 274: NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings. This particular report, in addition to describing general seismic upgrading practices in the United States, lists many large American projects that were seismically retrofitted.

Papers were also reviewed from the proceedings of major earthquake engineering conferences, such as the World Conference on Earthquake Engineering and the Seismic Retrofit of Historic Buildings Conference. In some instances the papers produced at these conferences give direct information about seismic retrofits and in other cases, merely point to relevant projects for which further literature review was conducted. Specific references are noted throughout the text.

Finally, information of certain projects was obtained directly from team members who worked on that particular seismic retrofit project, including Halsall's sub-consultant Ausenco (and their specialist sub-consultants Forell Elsessor and Seismic Isolation Engineering), who also have a broad range of experience with conventional upgrades, as well as base isolation retrofits.

A representative sample of seismic retrofit projects was selected, focusing on buildings that were supported by unreinforced load bearing masonry walls, contained heritage value and/or were of similar importance. Preference was given to projects with similar scales of importance (such as other houses of government), similar levels of seismic hazard, or with unique lessons to be learned.

The following projects were not included in the report because there was either insufficient information available, the type of structure was too different than the Centre Block or the details and type of upgrade were very similar to other presented projects:

- Cecil H. Green Library, California (concrete walls + diaphragms);
- East Memorial Building, Ottawa (friction dampers);
- Iasi City Hall, Romania (base isolation);
- Los Angeles City Hall, California (base isolation + concrete walls);
- Oakland City Hall, California (base isolation + concrete walls + steel bracing);
- Saint John Baptist Cathedral, Haiti (base isolation + steel bracing);
- Sheraton Palace Hotel, California (concrete walls);
- Saint Vincent Hospital (friction dampers);
- U.S. Court of Appeals, California (base isolation + concrete walls);
- Utah State Capitol, Utah (base isolation + concrete walls); and
- Wellington Building, Ottawa (concrete walls).

4. GAP ANALYSIS

4.1 Documentation Review

In the process of reviewing the documentation that was provided by PWGSC 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. Details of the gaps in information can be found in the Work Package 2 report – Preliminary Seismic Assessment of Centre Block.

5. DETAILED RESULTS OF RESEARCH

5.1 Research Introduction

Detailed information on the seismic retrofit projects of 11 heritage, masonry structures is given below. While the buildings range in layout, function and seismic hazard, the research is focused on buildings of a similar age, construction type and heritage importance as the Centre Block. Examples of base isolation, drilled steel anchors and fibre reinforced polymer fabrics are given, as well as more conventional methods, such as steel braced frames and reinforced concrete shear walls.

5.1.1 Understanding Performance Levels

A building that is designed with conventional method to resist 100% of the 2010 NBCC seismic loads is still expected to be damaged in the design level earthquake. This includes damage to both structural and non-structural components and the building may not be useable after the earthquake.

The performance level specified by the NBCC for normal buildings is to protect of the life safety of the building occupants and the general public. For the post disaster buildings, the intent is to provide a high likelihood that the building will remain functional after the earthquake, although damage is still expected.

Base isolation can offer a higher performance level by providing life safety, building functionality, and greater protection from damage for both structural and non-structural components. As discussed in the projects below, many building owners have chosen a higher performance target to protect their heritage structures and have used base isolation to achieve that performance level.

5.1.2 Comparison of Seismic Demand to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below for each individual project. The numbers for Ottawa are based on the numbers provided in the 2010 National Building Code of Canada climatic data. The numbers provided for the individual projects are for the city (or a nearby city, where the exact city is not available) from the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad[1]. The values in the Geological Survey of Canada's report are all normalized to a site classification of B and so site class B values are given for Ottawa, as well.

It should be cautioned that these values provide only a very rough estimate of the comparison of the predicted seismic demand for a building outside of Ottawa to the Centre Block. Firstly, as identified in the Geologic Survey of Canada's report, the values provided are only approximations based on the best information and estimation techniques available. Secondly, they do not consider the specific soil conditions of each site because this information was not available in the literature. Finally, the way in which loads and the design of structures are determined in other countries can vary significantly, which may affect the type of seismic retrofit plan selected.

Seismic demand for a region is not equivalent to the design seismic forces that a building must resist. The driving rationale for the selection of a seismic retrofit should be based on the results of a capacity/demand analysis for a given structure, combined with the desired level of performance, not on the level of seismic hazard for a region. The retrofit methods identified can be applicable to any structure that is determined to not be able to resist the specified seismic forces, nor achieve the performance expectation for the building.

5.1.3 Performance Targets and Heritage Impact

For all of the upgrades researched, where the information was available, the designed level of life safety was never specified less than 100% of the applicable building code. Regardless of performance target, effort was made in the design of the seismic retrofit to minimize the heritage impact of the existing building. However, the decision to seismically upgrade the building and to use particular performance targets was made irrespective of the heritage impact, with life safety and building protection during an earthquake being the primary concerns. Once a level of protection was selected, proposed seismic retrofit methods were developed with the goal of balancing heritage impact, functional impacts and project costs.

5.1.4 Project Costs

Although some total project costs were available for certain projects, the numbers are not included in this report because there is no consistency to the type or quality of the information provided. The seismic upgrades were typically undertaken as part of larger upgrade projects but the breakdown of the structural work from the total costs was not given. In many cases, the type of currency (e.g. U.S. dollar, Canadian dollar, New Zealand dollar) and the year of currency are not explicitly stated.

5.2 New Zealand Parliament House[2]

The Parliament House is the primary building of the New Zealand Parliament Buildings, which house the national government in Wellington, New Zealand. The building was reconstructed in 1922, after the original Parliament House was destroyed by fire, and consists of 4 levels above grade and one basement level. The original lateral load resisting system of the building consisted of brick and stone masonry bearing walls. The floor system consisted of simply reinforced concrete slabs bearing on steel beams. In 1989, the New Zealand government decided to strengthen the existing building as part of an overall refurbishment of the existing facilities and construction was completed in 1995.

The Parliament House is relevant to the Centre Block because of the similarity of age and type of construction, and because of the similar importance of function of the buildings. It also provides an example of how another government went through a process of evaluating different levels of intervention to the building and their associated costs, as well as performance targets of either life safety or building protection of a “national monument”.

5.2.1 Seismic Retrofit

Refer to Appendix A for details of the upgrade that were included in the referenced report.

The Parliament House was seismically retrofitted by inserting new reinforced concrete shear walls, base isolating the structure and reinforcing the diaphragms. The lead-rubber bearings were installed in the middle of the basement level, beneath all of the walls and columns. In order to distribute gravity loads from the walls above to the bearings, and then distribute loads from the bearings back over the length of the walls, concrete “sandwich beams” on either side of the existing walls were installed. After the sandwich beams are installed, the existing foundations can be cut to allow the structure above to move freely on the new bearings.

Where the sandwich beams could not be installed for architectural reasons, the walls beneath the ground were replaced with new concrete beams that sat on the new bearings, which required extensive shoring during construction.

Around isolated columns, large concrete capitals were built immediately beneath the ground floor. The capitals first facilitate temporary shoring of the columns, while the column below is removed and the bearing inserted, and second, enhance the floor connection so that loads can be transferred to the new bearings. Below the bearing, the remainder of the pier that was cut was reinforced either with stub walls or foundation beams.

Above the ground floor new reinforced concrete walls were inserted around light wells and where the space allowed. On many of the existing brick masonry walls, reinforced concrete was cast immediately adjacent to the wall. Reinforcing dowels were placed between the masonry and the concrete, and pouring pressures on the existing walls were limited by limiting the heights of concrete pours.

At the ground floor, the diaphragm was strengthened by adding a 75 mm reinforced concrete topping over the existing floor. The stresses at this level are greatest because the shears are redistributed to the bearings. At the second and third floors, it was not possible to increase the floor thickness by 75 mm, so 5 mm thick steel plates were added instead. Specifics are not given in the literature, but it is noted that there were a number of challenging details associated with connecting the steel plates through existing walls and to existing floors.

In order to accommodate the movement of the base isolators, a 400 mm wide moat was constructed around the entire building at the ground floor. The moat was topped with a sliding cover plate and the main stairs were required to be partially reconstructed to introduce a horizontal movement joint.

5.2.2 Type of Analysis Considered

The analysis of the base isolation system was completed in 3 stages in increasing order of complexity. The first stage of analysis was a single degree of freedom analysis to obtain an estimate design of the bearings, displacements and transmitted base shear. This analysis was done using a proprietary computer program. The second stage of analysis was a multi degree of freedom analysis, using ETABS, to refine the accuracy of the results. The final stage of the analysis was a multi-degree of freedom time history analysis, which incorporates the non-linearity of the base isolators. Three earthquake histories were simulated.

5.2.3 Retrofit Options Considered and Rationale

The two options considered for the seismic retrofit for the Parliament House were adding only concrete shear walls, or adding less concrete shear walls with a base isolation system. Based on the two schemes developed, the design team believed there would approximately a 3% cost premium associated with the base isolation method. The premium of the base isolation method over concrete shear walls alone would have been higher for a more conventional building but the design team anticipated very difficult detailing of the concrete shear walls that would add to their expense. This complication arose because the layout of the concrete shear walls was optimized to reduce the impact on the heritage features rather than to improve structural performance. The availability of the basement for locating the lead-rubber bearings was advantageous to the base isolated system.

Despite the higher cost, the base isolation method was selected because it provided a better level of protection to both the occupants and the heritage structure. It also reduced the impact of the retrofit on the heritage features by reducing the loads enough to allow certain new concrete walls in sensitive areas to be removed from the scope of work. Finally, it increased the likelihood that the building would remain operable after a seismic event.

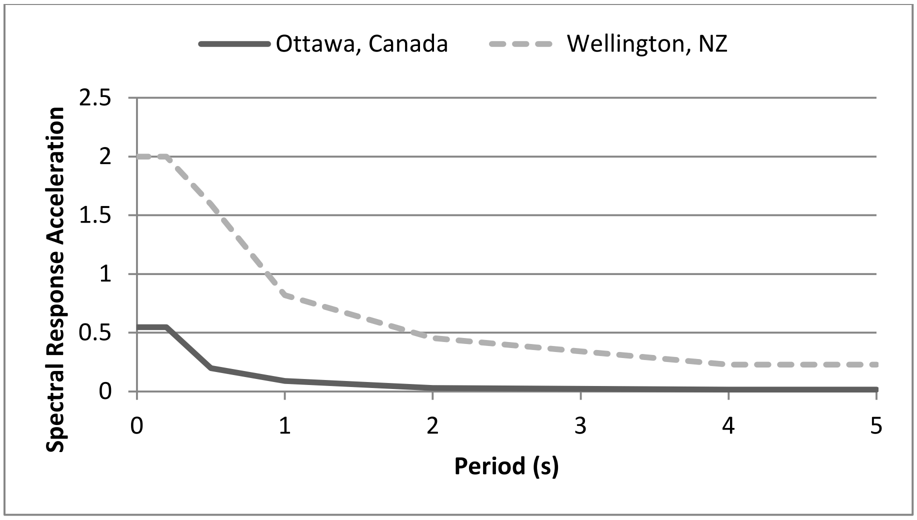
5.2.4 Performance targets

The design process produced six retrofit schemes for the seismic upgrade of the Parliament House. The schemes considered three levels of conservation of heritage fabric in the retrofit program: maximum, moderate and minimum, which are associated with increasing amounts of interruption to the existing building. For each level of conservation, they also needed to consider a level of protection appropriate to either a “National monument” or to only ensure the safety of the occupants.

The performance targets of the selected option of base isolation were a moderate conservation scheme, which considered a level of protection appropriate to a National monument.

5.2.5 How forces compare to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below based on the numbers provided for Ottawa in the 2010 National Building Code of Canada climatic data and the numbers provided for Wellington, New Zealand, by the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad. All values are normalized to a site classification of B. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be approximately 4 times greater in Wellington than in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.3 Salt Lake City and County Building[3, 4]

The Salt Lake City and County Building (City-County Building) houses the municipal government of Salt Lake City, Utah. Constructed in 1894, the main building has an approximate height of 30 m, while the main tower is approximately 76 m tall. The gravity and lateral load resisting system is primarily comprised of unreinforced sandstone and brick masonry walls, typically varying in thickness from 0.6 m to 0.9 m, but increasing up to 2.5 m thick at the base of the piers of the tower. The floor structures consist of either wood joists or iron beams with brick arches. In 1984, the city decided to proceed with the seismic strengthening.

The City-County Building is relevant to the Centre Block because of the similarity of the construction materials and layout of these two heritage buildings, including a prominent tower that rises above a more massive office block.

5.3.1 Seismic Retrofit

Refer to Appendix A for details of the upgrade that were included in the referenced report.

The City-County Building was seismically retrofitted by base isolating the structure with lead rubber bearings. In order to distribute gravity loads from the walls above to the bearings, concrete “sandwich beams” on either side of the existing walls were installed. The beams are fastened to the existing masonry walls by beams of post-tensioned rods and temporarily supported on a bed of mortar, while openings for the new isolators are cut.

The lead-rubber bearings were installed on top of the existing footings beneath all of the walls and columns in the newly cut openings. The isolators sit on a grillage of structural steel beams in order to distribute the load to the existing footings. There is no basement to the structure, so in order to provide enough height to install the isolators, the existing wood first floor was removed and replaced with a new steel and concrete floor approximately 350 mm higher, which also acts as a rigid diaphragm to distribute the load to the isolators. The isolators were pre-loaded to 2/3 of the dead weight in order to limit building settlement and potential cracking. Once the isolators were installed, the temporary mortar joint was removed.

In addition to the base isolation, strengthening measures also included: constructing a structural steel space frame within the tower, strengthening of existing diaphragms with a lightweight reinforced concrete topping, as well as anchorage of the diaphragm to the existing masonry walls and addition of continuous steel tension ties through interior walls, new structural plywood framing and diaphragms to stabilize the existing roof structure and anchorage of all exterior masonry appendages, such as chimneys, statues and parapets.

5.3.2 Type of Analysis Considered

The detailed design of the base isolating retrofit method was completed with a dynamic time history analysis of a 3D computer model.

5.3.3 Retrofit Options Considered and Rationale

Three schemes were developed for the seismic retrofit of the City-County Building: two involving either shotcreting existing masonry walls or replacing existing masonry walls with reinforced concrete walls, and a third that involved base isolation with less substantial interventions to the existing building structure. Although the base isolation scheme was more expensive than the other two options presented, it was selected by the Salt Lake City mayor and council because it would require less intervention to the heritage fabric. In addition, the city hoped to be able to use this building to coordinate emergency relief in the event of an earthquake and base isolation would increase in the probability of the building remaining operable. Finally, the building was identified as a symbol of Salt Lake City and base isolation was believed to offer the best system of protection for non-structural components.

5.3.4 Performance targets

The seismic retrofit was designed based on earthquake time history records that were believed to be similar to what could be expected in Salt Lake City. The other two schemes that were considered were designed to the Uniform Building Code of the time.

5.3.5 How forces compare to Ottawa

No estimated spectral accelerations were available for Salt Lake City or any nearby regions in the Geological Survey of Canada’s report. However, the seismic retrofit was designed to a seismic hazard level of Zone 3 from the Uniform Building Code (UBC) of the time. The UBC ranked seismic hazard on a scale from 0 to 4, with 0 being associated with no seismic risk and 4 being associated primarily with the west coast of California. As a rough comparison, by extending Zone classifications given in the United States near to the border, Ottawa would likely be considered as a Zone 2, which is a lower seismic hazard than Salt Lake City. For a discussion of relative seismic hazards, please refer to section 5.1.1.

5.4 The Parliament Building of Macedonia[5]

The Parliament Building of the Republic of Macedonia in Skopje, was originally constructed between 1936 and 1939, although there were many enlargements and additions added over the subsequent years. The building forms an unequal pentagon with one basement level and 3 storeys above grade. The floor structures are comprised of reinforced concrete slabs bearing directly on brick masonry walls and concrete columns. The building suffered extensive damage during an earthquake in 1963, primarily at the ground floor, and was repaired the following 2 years but not strengthened. Due to the need to add a fourth storey to the structure, it was decided to complete a seismic upgrade program, and construction began in 2010.

The Macedonia Parliament Building is relevant to the Centre Block because of the similarity of construction materials, as well as function and importance of the building. It also represents an example of a seismic retrofit where importance was given to preserving functional spaces but not the heritage fabric of the building.

5.4.1 Seismic Retrofit

The seismic retrofit of the Macedonia Parliament Building consisted of many strengthening components. In some locations, existing masonry walls were removed and replaced with reinforced concrete walls and new concrete foundations, and in other locations, existing masonry walls were jacketed with reinforced concrete. Many existing concrete columns were wrapped with reinforced concrete to prevent the failure of the gravity load system under expected seismic deformations. The connection of the existing concrete floor diaphragm to the existing masonry walls was strengthened by installing reinforcing dowels.

5.4.2 Type of Analysis Considered

For the evaluation of both the existing structure and the seismic retrofit, nonlinear dynamic analysis was performed.

5.4.3 Retrofit Options Considered and Rationale

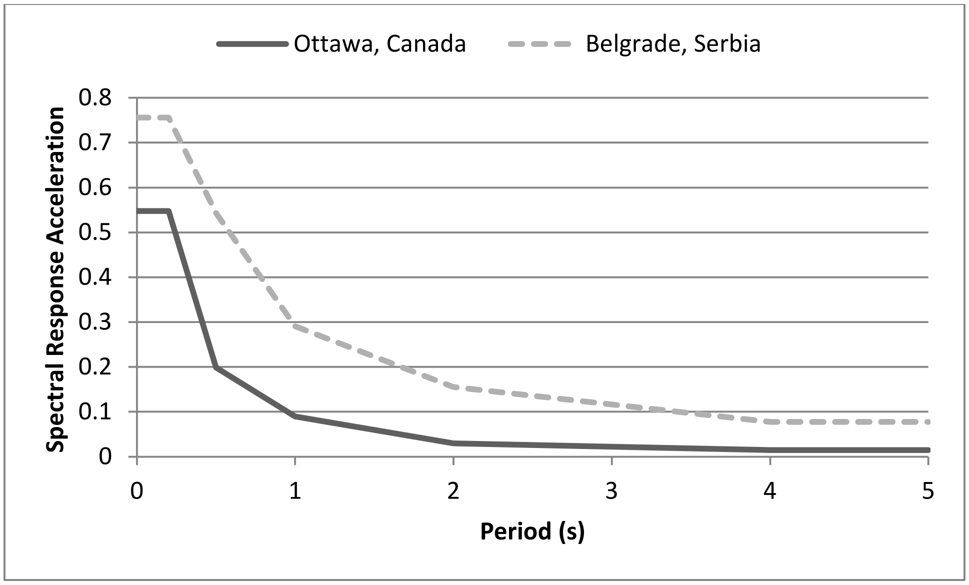
The literature states that several seismic upgrade solutions were evaluated but does not specifically identify what the other options were. However, the selected option, described above, was chosen because it was the least expensive option that would satisfy the code requirements. It was also noted that reinforced concrete was selected over structural steel because its material characteristics better matched those of the existing masonry wall structure. It should be noted that although the least expensive option was selected, the design attempted to minimize the level of intervention within existing functional spaces.

5.4.4 Performance targets

The Macedonia Parliament Building was strengthened to the level prescribed by the current technical regulations of Macedonia, the Rulebook on Construction of Structures in Seismically Prone Regions.

5.4.5 How forces compare to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below based on the numbers provided for Ottawa in the 2010 National Building Code of Canada climatic data and the numbers provided for Belgrade, Serbia, by the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad. Belgrade was used as a reasonable approximation to Skopje, Macedonia, as they are geographically close and no values for Skopje were available. All values are normalized to a site classification of B. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be approximately 1.4 times greater in Belgrade than in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.5 Victoria Memorial Museum Building[6]

The Victoria Memorial Museum Building (VMMB) is a Classified heritage structure and Canadian National Historic Site, located in Ottawa, Ontario. Constructed between 1905 and 1910, the building has served as home to several museums, and temporarily to the Canadian parliament, after the original Centre Block burned down in 1916. The four storey exterior walls are comprised of unreinforced stone and brick masonry with a rubble core, while the interior walls are primarily brick masonry. The floors were originally composed of terra cotta flat arches with a cementitious topping. As part of a major renovation program that began in 2000, the decision was made to seismically strengthen the building and construction was completed in 2010.

The VMMB is relevant to the Centre Block for a number of reasons. They are a similar age and construction type, and as they are in the same city, they have a similar seismic hazard. They are regarded with similar importance in Canada, and the VMMB provides an example of the Real Property Service's policy on seismic upgrades applied to an existing building.

5.5.1 <u>Seismic Retrofit</u>

Refer to Appendix A for a schematic rendering and photo of the seismic retrofit.

The seismic retrofit program for the VMMB primarily consisted of introducing a new “steel endoskeleton truss frame” on the inside of the exterior masonry walls. In addition to acting as a new lateral load resisting system for the building, the steel frames also restrained the masonry walls from buckling out-of-plane during an earthquake and provided a backup gravity load carrying system, in the event that the masonry walls failed during an earthquake.

The truss frame was designed to maximize stiffness, minimizing the differential stiffness between the steel frame and the masonry walls. The original floor topping was removed and replaced with a lightweight concrete structural slab diaphragm that was anchored to the existing masonry walls and new steel frames. New reinforced concrete foundations were also installed beneath the new steel frames and new concrete shear walls were also added around new elevator and stair cores.

At the front entryway, the masonry walls were reinforced with large grouted steel anchors that were drilled through the masonry from the top down. Steel frames were not used in this area in order to preserve all of the exposed heritage finishes.

Steel framing was also added at the roof level to seismically restrain the existing masonry parapets, and a wire mesh catcher system was installed beneath the terra cotta arches at the ground floor to reduce the risk of falling pieces of mortar and tile during an earthquake.

5.5.2 Type of Analysis Considered

A site-specific seismic design response spectrum was developed for the VMMB. This additional geotechnical study was completed in order to provide a more accurate design than would be possible with the generic site classifications of the National Building Code of Canada. It was intended that the additional cost associated with the geotechnical study would be outweighed by the value of the construction cost saved and by the value of the higher level of safety for the building contents.

The design of the new steel frames was then based on a dynamic 3D analysis that used the site specific response spectrum data that had been determined.

5.5.3 Retrofit Options Considered and Rationale

The first seismic retrofit option considered was to reinforce all of the masonry walls with drilled masonry anchors with grout sleeves for both in-plane and out-of-plane forces. The use of these anchors was quickly ruled out because it was estimated to be both the most of expensive and time consuming option. It was also felt to be the riskiest option because there was the highest probability of damage occurring to the walls during construction.

Friction dampers were briefly considered but were excluded because a system that relies on a significant degree of movement before engaging did not appear to be appropriate to protect a brittle structure and its contents. Base isolation was also ruled out because of the anticipated higher cost.

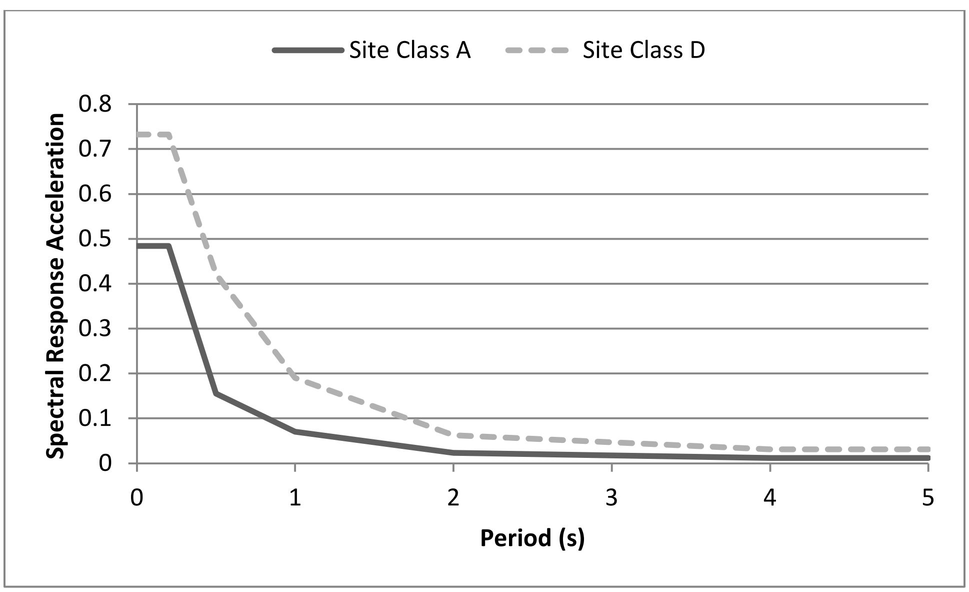
The second and third options, steel braced frames and concrete shear walls, were both considered to be feasible upgrade options from a seismic perspective. However, due to ongoing settlement challenges with the soft soil beneath the structure, the amount of weight added to the building needed to be minimized. Consequently, steel frames were selected as the predominant system for the building, with concrete shear walls present only at the new elevator and stair cores. The new steel endoskeleton frames and new floor diaphragms were located so as to minimize the heritage impact on the exterior stone masonry walls and the existing terra cotta arches.

5.5.4 Performance targets

In order to recognise the importance of the VMMB's heritage status and its collections, the seismic upgrade was designed to 150% of 1995 National Building Code of Canada, with the intent of exceeding the minimum life safety requirements. At the time of design, in 2002, the 2005 NBCC draft had been released for comments and the climatic data shown a significant increase in seismic loads to short period structures in Ottawa. The 50% increase in the design loads from the 1995 NBCC was also hoped to capture the anticipated increase in seismic loads.

5.5.5 How forces compare to Centre Block location

The VMMB is also located in Ottawa, Ontario, and was designed for similar spectral accelerations as the Centre Block would be, except that they are located on very different types of soils. The Centre Block, bearing directly on rock, is assumed to be Site Class A. The VMMB soil classification was determined to be Site Class D. Spectral accelerations for Ottawa for the two site classes from the 2010 National Building Code of Canada climatic data are provided below. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 1.5 times greater with Site Class D than with Site Class A, in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.6 San Francisco City Hall[7]

The San Francisco City Hall was designed in 1913 to replace the original city hall that was destroyed by the 1906 San Francisco earthquake. Listed in the National Register of Historic Places, the City Hall is recognised throughout the United States as one of the best examples of classical American architecture. The main building consists of 5 stories above grade, while the dome has a height of approximately 90 m above the ground. The gravity load system consists of reinforced concrete slabs supported by steel beams and columns. Lateral loads are primarily resisted by exterior integral stone and unreinforced brick masonry walls and interior hollow clay tile walls. The City Hall was significantly damaged in the 1989 Loma Prieta earthquake but did not collapse. As a part of the repair work, the building was seismically strengthened and reopened in 1999.

The San Francisco City Hall is relevant to the Centre Block because of the similarity of the construction materials and heritage importance of the two buildings.

5.6.1 Seismic Retrofit

Refer to Appendix A for a schematic layout and staging diagram of the seismic retrofit.

The San Francisco City Hall was seismically retrofitted primarily by the introduction of base isolators beneath the structure. The optimal height for the base isolation plane was determined to be immediately beneath the ground floor level columns and walls. In order to install the isolators, the existing foundations were partially modified and strengthened, and the existing columns and walls were shored. The new bearings could then be installed, followed by the new structural steel and composite steel deck to replace the ground floor structure, which acts as a rigid diaphragm to tie the isolators together.

Although the base isolation reduced the seismic demand on the structure, new reinforced concrete shear walls at the interior light wells were also required and longer shear walls were required at the ground floor to minimize uplift forces at the isolator locations. The existing foundations beneath the new isolators were tied together with concrete grade beams and a new “moat” was installed around the building with concrete retaining walls to separate the structure from the adjacent ground.

5.6.2 Type of Analysis Considered

A preliminary dynamic seismic analysis was performed to evaluate the existing structure. More sophisticated computer models were developed to analyse the seismic upgrade options, including extensive studies of the interaction between the dome and the rest of the building. Parametric studies were completed to assess the influence of materials properties on the results as well.

5.6.3 Retrofit Options Considered and Rationale

The options considered for the seismic strengthening of the City Hall were all strongly influenced by the requirement to minimize the visual impact of the work on both the heritage interior and exterior elements. Four options that met this criterion were considered: new concrete shear walls, base isolation, new braced frames, and the introduction of a flexible steel moment frame at the ground floor. The goal of each option was to bring the level of life safety up to current code standards for both the primary structural and non-structural elements, provide complete load-paths for seismic forces, protect heritage components from earthquake loads, and minimize the level of interruption caused by the retrofit program.

Only the concrete shear wall and base isolation options were selected for further development. The braced frame option was ruled out because although it would be theoretically possible to provide the braces with enough stiffness and strength, the level of disruption to existing steel frame and floor system would be extremely high and consequently this option would also be more expensive. Also, while adding the braces would prevent the collapse of the building, it provided no additional level of protection to the non-structural components of the building.

The flexible storey option was based on a similar notion to that of base isolation in that it tried to create a flexible but ductile link to separate the ground movements from the rigid structure above. This separation would be achieved by inserting a ductile steel moment frame into the structure at the ground floor. However, the new frames would require the complete temporary shoring of the structure above and would be extremely disruptive to the ground floor while only providing modest improvements to the performance of the existing structure above. For these reasons, this flexible storey option was also not developed further.

The concrete shear wall option was considered the most structurally efficient conventional seismic upgrade option. However, it would still require fairly extensive interruption to the existing public spaces and the new shear would stiffen the structure, which would increase seismic loads, and require more strengthening work (or complete replacement) of the existing dome. The new concrete shear walls also provided no additional level of protection to the non-structural components of the building.

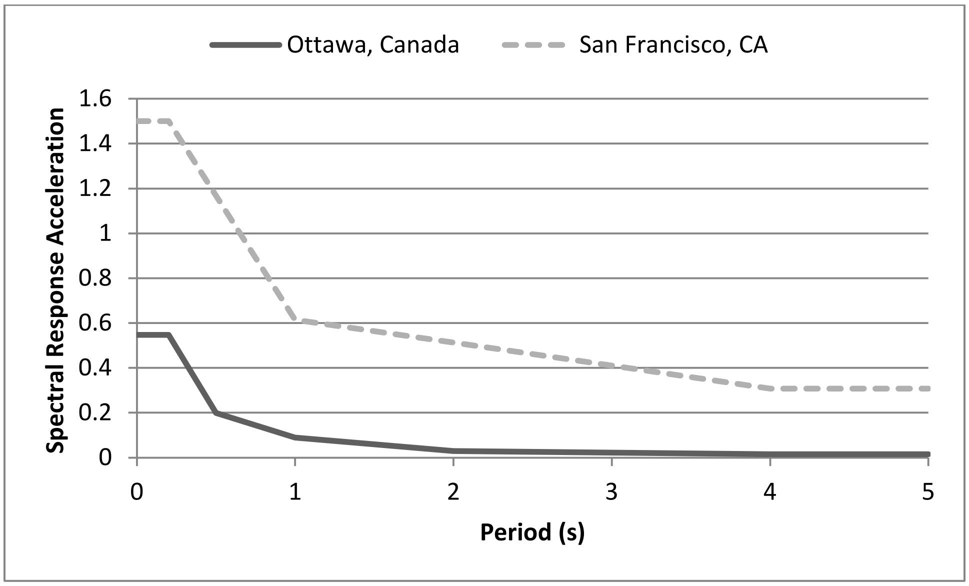
The base isolation method, as described above, was selected because it: significantly reduced the level of stress in the existing structure in the event of an earthquake, allowed for the minimum disruption of interior public spaces and historic features, required the minimum structural work to the dome and would provide the greatest level of protection to architectural features in the event of an earthquake. Furthermore, of the two options selected for further development, it was believed that the base isolation would be cheaper than the conventional shear walls.

5.6.4 Performance targets

The retrofit program was designed to 100 % of the requirements of the San Francisco Building Code for seismic loading.

5.6.5 How forces compare to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below based on the numbers provided for Ottawa in the 2010 National Building Code of Canada climatic data and the numbers provided for San Francisco, California, by the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad. All values are normalized to a site classification of B. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 2.7 times greater in San Francisco than in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.7 Sage and Gould Halls, Robert College[8]

Sage and Gould Halls are two historic, academic buildings located on the campus of Robert College in Istanbul, Turkey and were both constructed between 1910 and 1914. The four and five-storey buildings are both supported by similar types of construction, primarily unreinforced exterior concrete walls, unreinforced block masonry interior walls and steel columns. The exterior walls typically vary between 500 and 700 mm thick and are the primary lateral load resisting system. The floor structure consists of steel beams encased in plain concrete and reinforced concrete floors. The foundations consist of reinforced concrete strip footings bearing on rock. In 2007, a seismic assessment determined that both structures did not meet the current seismic code requirements and a retrofit plan was determined.

The Sage and Gould Halls are relevant to the Centre Block because they provide an example of how buildings of a similar age and construction type can be retrofitted with a relatively new material, fibre reinforced polymer fabrics, without significantly impacting the function of the building during or after construction.

5.7.1 Seismic Retrofit

Refer to Appendix A for sample details of the seismic retrofit method.

The planned seismic retrofit for both the Sage and Gould Halls involves the application and anchorage of fibre reinforced polymer (FRP) composite fabrics to the exposed concrete faces. Layers of FRP fabrics would be installed on both the interior and exterior faces of the walls and anchored to the walls with FRP anchors, epoxied into predrilled holes. The fibres within FRP fabric are unidirectional, so the fabric orientation has to suit the required strengthening. To increase bending capacity, the fabric is placed with the fibres orientated vertically, and to increase shear capacity, the fibres are oriented horizontally. Additionally, on the sides of the window openings, FRP layers would be installed with FRP anchors through the spandrel panels above and below. The material finish is selected to blend with the bare concrete wall surfaces, or hidden by finishes.

5.7.2 Types of Analysis Considered

Analysis of both the existing and upgraded structure was completed using a dynamic 3D computer model, with seismic demand entered in the form of a response spectrum.

5.7.3 Retrofit Options Considered and Rationale

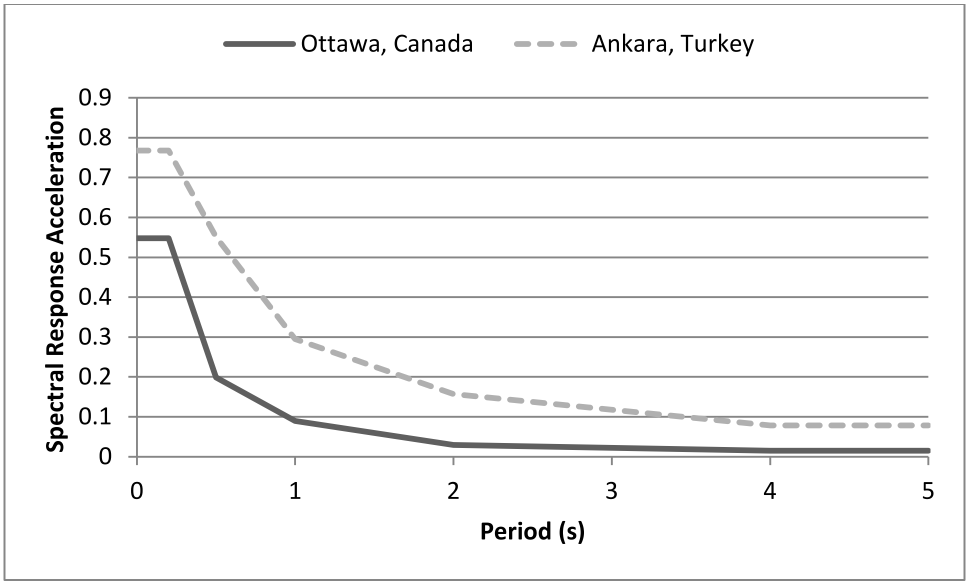
The literature states that different seismic upgrade solutions were evaluated but does not specifically identify what the other options were. However, the selected option, described above, was chosen because it was less invasive to the usable space of the buildings, less disruptive to the occupants of the buildings during construction and minimizes the amount of intervention to the buildings' historic fabric, particularly the interior components.

5.7.4 Performance targets

The retrofit program was designed to 100 % of the Life Safety requirements of the 2007 Turkish Seismic Design Code.

5.7.5 How forces compare to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below based on the numbers provided for Ottawa in the 2010 National Building Code of Canada climatic data and the numbers provided for Ankara, Turkey, by the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad. Ankara was used as a reasonable approximation to Istanbul, as they are geographically close and no values for Istanbul were available. All values are normalized to a site classification of B. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 1.4 times greater in Ankara than in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.8 Rikkyo University Chapel[9, 10]

The Rikkyo University Chapel is one of the most architecturally significant buildings of the Rikkyo University campus in Tokyo, Japan, and was constructed in 1920. The 3 storey building is primarily comprised of unreinforced masonry walls and piers with a timber. After the buildings walls were damaged in the 1923 Kanto Great Earthquake, a thin, reinforced concrete layer was added to the inside of the masonry walls to strengthen them. However, the building was again damaged in the 1995 Hyogoken Nanbu Earthquake and a more modern seismic retrofit was completed in 1999.

The Rikkyo Univerisity Chapel is relevant to the Centre Block because, for a similar type of construction, it demonstrates how even though the building had been conventionally upgraded with reinforced concrete, it was still significantly damaged in an earthquake. The building was then base isolated and in a subsequent earthquake, the building was not at all damaged.

5.8.1 Seismic Retrofit

Refer to Appendix A for sample details of the seismic retrofit method.

The seismic retrofit option for the chapel consisted of base isolating the existing building. The isolators on this project are different from those typically seen on the American projects in that they consist of rubber bearings and lead bars dampers, which are completely separate from each other (see photo in Appendix A). The existing ground floor slab was removed and the soil was excavated down to the bottom of the existing foundations. New foundation beams were installed around the existing footings, and then a second excavation occurred under the foundation to install the rubber bearings on new footings. A new system of reinforced concrete beams and slabs was installed at the ground floor to spread the loads to the new bearing system. Finally the lead bar dampers were installed and a retaining wall constructed around the perimeter of the building to maintain the required movement gap of the isolated structure.

The existing timber roof diaphragm was strengthened with steel bars, and the connections of the existing masonry wall to the roof and ground floor were strengthened. This building was the first base isolation retrofit of an existing masonry building in Japan.

5.8.2 Types of Analysis Considered

A non-linear seismic response analysis was completed for both the existing structure's capacity and the new base isolated structure. Five ground motion records were scaled to the level of demand for Tokyo and used for the time history analyses.

5.8.3 Retrofit Options Considered and Rationale

The literatures states that most buildings that were seismically upgraded in Tokyo after the 1995 earthquake were retrofitted with new concrete shear walls or steel braced frames. However, these methods would interfere with both the interior and exterior appearance of the existing building. Since the Chapel was regarded was one of the most architecturally important buildings on the campus, base isolation provided a greatly preferable option because it required minimal intervention to the building above the ground floor.

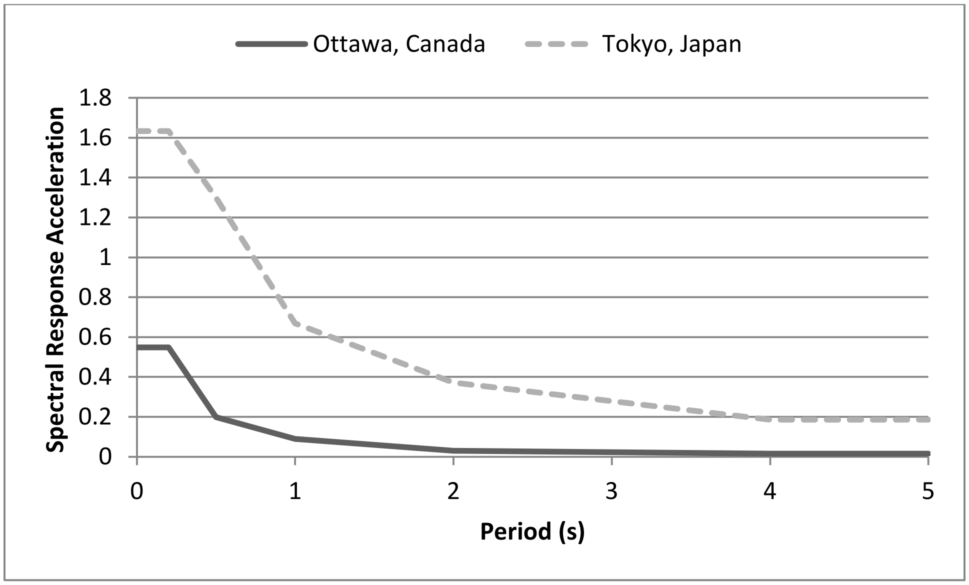
5.8.4 Performance targets

The time history records used in the analysis were scaled to the design response spectra specified by the Japanese code.

Accelerometers were also installed in the chapel and in an immediately adjacent masonry structure without base isolation. The 2011 Tohoku Earthquake caused measurable shaking in Tokyo and the responses of both buildings were recorded. No damage occurred to the Chapel and the accelerations in the non-base isolated masonry building were 6 to 8 times greater than those in the Chapel.

5.8.5 How forces compare to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below based on the numbers provided for Ottawa in the 2010 National Building Code of Canada climatic data and the numbers provided for Tokyo, Japan, by the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad. All values are normalized to a site classification of B. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 3 times greater in Tokyo than in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.9 Bucharest City Hall[11]

Bucharest City Hall in Romania serves at the main municipal office of the capital city of Bucharest and was constructed between 1906 and 1911. The main building has 4 stories with a total height of approximately 21 m above grade. The floors are made of reinforced concrete slabs, which are primarily supported on unreinforced brick masonry walls that act as the lateral load resisting system. Due to the age of the structure and four past earthquakes that affected the building, it was determined that the building should be seismically upgraded and two schemes were developed. Work was anticipated to begin in 2012 but no additional literature was found to confirm that date.

The Bucharest City Hall is relevant to the Centre Block because it demonstrates the application of base isolation in a region of similar seismic hazard to Ottawa. It also gives an example of a building where base isolation decreased the anticipated cost of the retrofit versus a conventional upgrade because the base isolation negated the need for any work about the ground floor.

5.9.1 Seismic Retrofit

Refer to Appendix A for a sample comparison of the existing structure to the proposed structure.

The recommended scheme for the seismic retrofit of the City Hall was to install lead rubber bearing isolators in the middle of the basement of the building. The isolators were to be installed on top of new reinforced concrete foundations. New reinforced concrete beams and slabs at the ground floor were to be installed to act as a rigid diaphragm and to transmit the gravity loads to the base isolators. No additional work was required above the ground floor.

5.9.2 Type of Analysis Considered

The type of analysis used to evaluate this structure was not stated in the literature.

5.9.3 Retrofit Options Considered and Rationale

The original seismic retrofit option consisted of installing many new concrete shear walls throughout the building. At the first and second floors, the brick walls would also need to be wrapped with a layer of reinforced concrete. Due to the poor soil conditions beneath the building and the additional weight of the concrete, the soil would need to be injected with a cementious material to stabilize it and the existing raft slab would need to be thickened and reinforced. Also, two expansion joints will need to be cut into the “U” shape of the building to divide it into 3 seismically separate pieces.

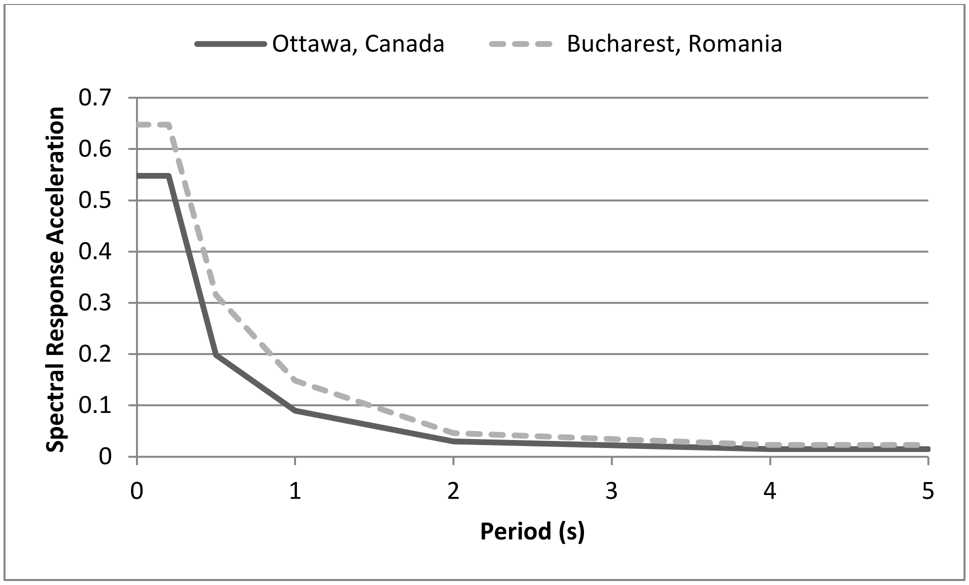
The base isolation option was deemed to be preferable because it required little intervention to the building above and much less extensive foundation work. Consequently, it was estimated that it will produce a savings of 5% in structural costs. Additionally, base isolation has the advantage of reducing repair costs (possibly to nothing) after an earthquake and increasing the chance of the building being fully operational after a major earthquake.

5.9.4 Performance targets

The seismic retrofit was designed to 100% of the Romanian building code requirements.

5.9.5 How forces compare to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below based on the numbers provided for Ottawa in the 2010 National Building Code of Canada climatic data and the numbers provided for Bucharest, Romania, by the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad. All values are normalized to a site classification of B. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 1.2 times greater in Bucharest than in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.10 California State Capitol West Wing[12]

The California State Capitol West Wing is the primary government building of the California State government in Sacramento, which was constructed in between 1861 and 1874. The building consists of 4 stories above grade and one below grade, with a central dome that extends above a rotunda to a height of approximately 65 m. The existing lateral load system consists of unreinforced brick masonry walls, which support a floor system of shallow brick arches that span to wrought-iron beams. The brick walls are supported on unreinforced concrete spread footings. In 1972, the structure was declared a seismic hazard and was largely vacated until a seismic retrofit was completed in 1982 and the building was reopened to the public.

The West Wing is relevant to the Centre Block because of the similarity of the construction materials and heritage importance of the two buildings.

5.10.1 Seismic Retrofit

Refer to Appendix A for plans and a section showing the seismic retrofit work.

The seismic retrofit of the West Wing primarily consisted of the demolition of most interior brick masonry walls and brick arch floors, and their replacement with 300 mm thick reinforced concrete shear walls and reinforced concrete floors.

At the exterior walls, and the walls of the rotunda beneath the dome, two wythes of the brick masonry were removed and replaced with a 300 mm thick reinforced concrete wall, with drilled anchors connecting the concrete to the remainder of the masonry wall. Mechanical splices were used extensively for large diameter reinforcing bars to reduce congestion and make construction feasible. At the fourth floor and roof level, new steel floor framing was also installed.

A new 900 mm thick reinforced concrete mat foundation was installed on top of the existing foundations, which supports the new concrete walls and is keyed into the existing masonry walls. Critically, the new load on the existing foundations was equal or less that before the retrofit.

At the dome structure, shotcrete, reinforced concrete and new structural steel framing were all installed to provide strength and ductility. Extensive shoring was required throughout the building for features that were not demolished during construction.

5.10.2 Type of Analysis Considered

A detailed analysis of soil conditions was completed to develop a site specific response spectrum at the foundation level of the building. Several dynamic analysis computer models were developed (likely because commercial programs as are used today were not available), and after studying the results, one was identified as provided the best representation for design purposes. The force distribution was also compared with that of the equivalent static procedure. The forces at the upper levels of the structure, particularly the irregular dome structure, were significantly higher from the dynamic analysis than the static analysis.

5.10.3 Retrofit Options Considered and Rationale

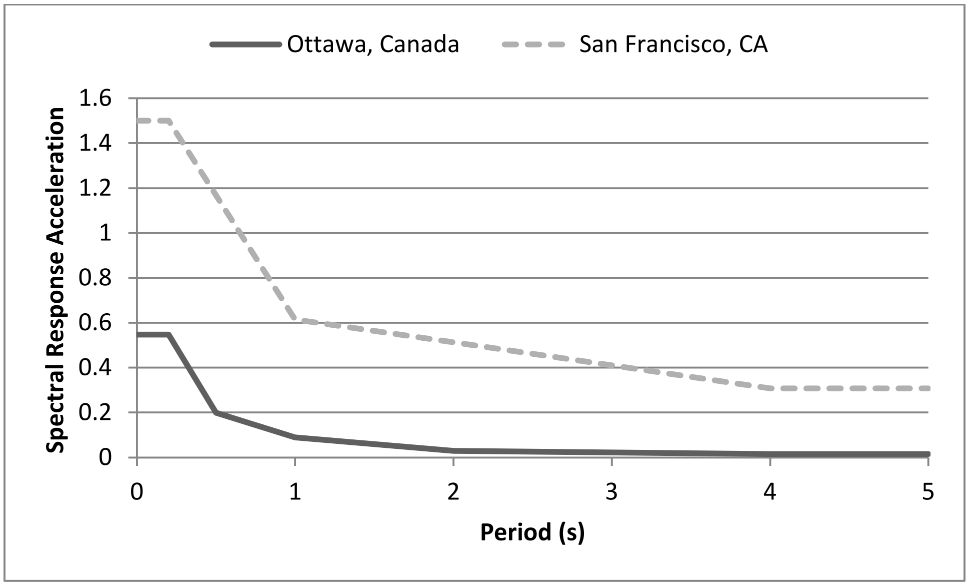
No other retrofit options were identified in the literature reviewed. The rationale for providing the concrete wall and floor system selected was that it would meet the strength and ductility requirements of the seismic provisions of the code at that time and allow the exterior appearance of the building to remain largely unchanged.

5.10.4 Performance targets

The seismic retrofit of the West Wing was designed to a level of forces that was determined specifically for this structure, its location and soil conditions. The literature does not explicitly state what these level of loads are or how they were determined.

5.10.5 How forces compare to Ottawa

An approximate comparison of the design spectral accelerations for given periods is provided below based on the numbers provided for Ottawa in the 2010 National Building Code of Canada climatic data and the numbers provided for San Francisco, California, by the Geological Survey of Canada in their 2008 report entitled Estimate Seismic Design Values for Canadian Missions Abroad. San Francisco was used as a reasonable approximation to Sacramento, as they are geographically close and no values for Sacramento were available. All values are normalized to a site classification of B. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 2.7 times greater in San Francisco than in Ottawa. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.11 Lord Strathcona Elementary School, Vancouver

The Vancouver School Board's Lord Strathcona Elementary School, located in downtown Vancouver, consists of three buildings with City heritage status. The 'Junior Building', a classroom building, was constructed in 1897 and had the attic restored after a fire in 1974. The three storey building plus basement has approximately 2200 m2 of floor area including the 'full height' attic. The basement perimeter walls are made of stone, while the upper floor perimeter and interior load bearing walls are made of multi-wythe brick. The floors and roof are constructed of timber construction. The school is planned for seismic upgrade, beginning in 2015. To date, the schematic design phase has been completed, the preferred retrofit option selected and detailed design is about to begin. At this time, it is understood that this will be the first base isolated building to be constructed in Canada.

The Lord Strathcona Elementary School is relevant to the Centre Block because it demonstrates an example of base isolation applied in Canada for an existing heritage building with similar construction materials.

5.11.1 Seismic Retrofit

Refer to Appendix A for a section showing the seismic retrofit work.

The selected seismic retrofit scheme for the school primarily involves the insertion of seismic isolators beneath a new concrete floor slab on top of new concrete columns and foundations. The location of the isolation plane was selected just below the first suspended floor (above ground), which was determined to be the most economical location. A new reinforced concrete slab will be constructed beneath the first floor to act as a rigid diaphragm to tie the isolators together. New reinforced concrete columns and foundations will be constructed to support the isolators within the interior of the existing basement. The basement will be fully reconfigured as part of the upgrade.

Often in base isolated buildings, the base isolators are located below grade. Isolators are very flexible by design, which requires large amounts of movement during an earthquake (up to approximately 0.5 m). To accommodate this movement, a “moat” must be constructed around the structure so that it does not hit the adjacent soil. For this school, because the isolation plane is above grade, a moat and retaining wall are not required. Some minor reinforcing of the timber elements in the attic is also required, along with the basement work.

5.11.2 Type of Analysis Considered

The schematic design of the seismic retrofit options to date has involved 3D computer analysis, including non-linear time history analysis. Further 3D non-linear time history analysis will be completed in the detailed design phase.

5.11.3 Retrofit Options Considered and Rationale

Two options were developed and considered for the seismic retrofit of the Lord Strathcona school, including the selected option described above. The intent of both upgrade options is to have no change to the exterior appearance of the building.

The second option considered adding concrete shear walls on the inside of existing masonry walls, complete with new reinforced concrete foundations and soil anchors. All of the diaphragms would need to be reinforced and substantial strengthening of the attic timber framing is required. Additionally, the multiple wythes of the brick walls will need to be “stitched” together with drilled anchors and connected to interior back up structure for out-of-plane restraint.

The shear wall option is intended only to provide life safety performance in the event of an earthquake. The base isolation option provides both post-disaster performance and heritage preservation performance.

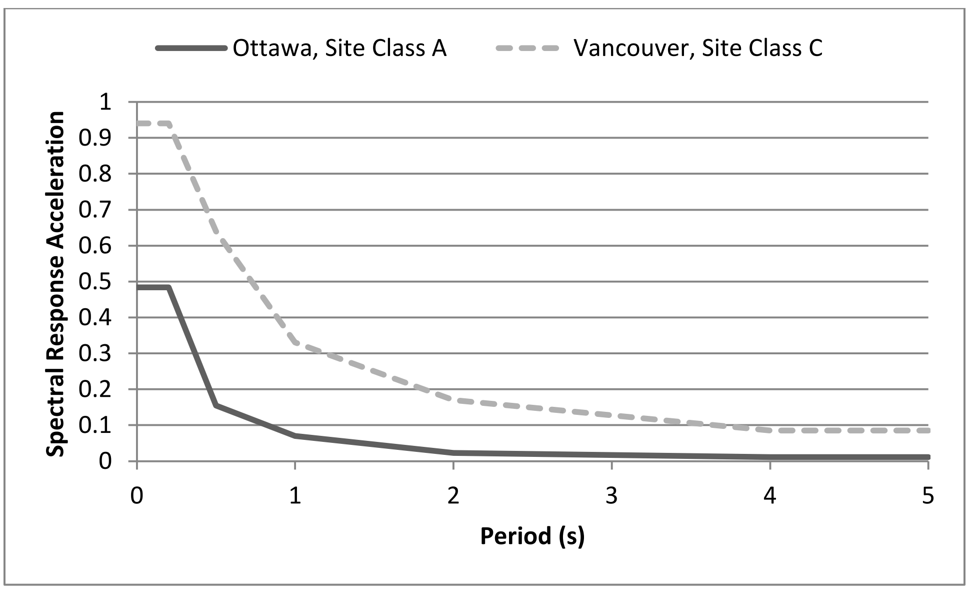
The costs of both options were found to be similar and consequently base isolation was selected because it provides a higher level of building performance and heritage preservation.

5.11.4 Performance targets

The seismic upgrade is being designed to 100% of the NBCC, however, as previously noted the selected option of base isolating the building provides a performance level comparable to a post-disaster level, including heritage preservation.

5.11.5 How forces compare to Ottawa

The Lord Strathcona is located in Vancouver, British Columbia, and the seismic retrofit was designed for the spectral accelerations specified in the 2010 National Building Code of Canada for Site Class C. The Centre Block, bearing directly on rock, is assumed to be Site Class A. Spectral accelerations for Ottawa and Vancouver from the 2010 National Building Code of Canada climatic data are provided below, for their respective site classifications. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 1.9 times greater for Vancouver, Site Class C, than for Ottawa, Site Class A. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.12 Holy Rosary Cathedral, Vancouver

The Roman Catholic Archdiocese of Vancouver's Holy Rosary Cathedral, located in downtown Vancouver, was constructed in 1899 and is a recognised heritage building. The cathedral is approximately 30 m by 50 m in plan, and has two towers the highest of which is approximately 37 m in height. The 3-wythe stone walls, ranging in thickness from 0.6 m to 1 m thick, consist of ashlar sandstone, rubble infill, and mortared random sandstone. The floors and roof are of timber construction. There is a partial basement, with a low crawl space beneath the majority of the ground floor slab. A voluntary seismic retrofit is proposed to commence in 2015. The work may be completed in combination with an extensive excavation beneath the cathedral to create a new full height basement.

The Holy Rosary Cathedral is relevant to the Centre Block because it demonstrates an example of base isolation applied in Canada for an existing heritage building with similar construction materials.

5.12.1 Seismic Retrofit

Refer to Appendix A for a section and a plan showing the two seismic retrofit options.

The preferred seismic upgrade option provides both post-disaster performance and heritage preservation performance to 100% of the NBCC, through the introduction of base isolators. The retrofit scheme introduces concrete shear walls in several locations of the cathedral on the inside face of the stone walls, including both towers. New concrete foundations are required beneath the walls. All of the main columns of the cathedral will be reinforced through their cores.

The roof is to be strengthened with external steel bracing, as will be the steeples and their connections to the towers. The vaulted ceilings will also be reinforced from within the attic space. The three wythes of the exterior stone walls need to be “stitched” together with drilled anchors.

Base isolators would be introduced in the space immediately beneath the ground floor and the gravity loads from the existing walls and columns would need to be transferred to the new base isolators from the existing foundations. The ground floor would be replaced with a reinforced concrete slab to act as a rigid diaphragm.

The isolators would sit atop new concrete columns and foundations, and there would be a system of steel bracing to laterally restrain the tops of the columns. A moat is required around the structure to allow for the deformations of the structure above the isolators.

5.12.2 Type of Analysis Considered

The design to date has involved a 3D computer analysis, including response spectra analysis for a conventional upgrade option, and non-linear time history analysis for a base isolation option.

5.12.3 Retrofit Options Considered and Rationale

In addition the base isolation option, a “conventional” seismic upgrade option was designed, which considered only the life safety performance of the occupants to 60% of the NBCC.

The conventional option includes almost all of the same work as the base isolation option, but to a much greater extent and size. This would mean more and larger concrete shear walls, as well as more reinforcing throughout the building. The base isolators, their new foundations and the moat would not be required.

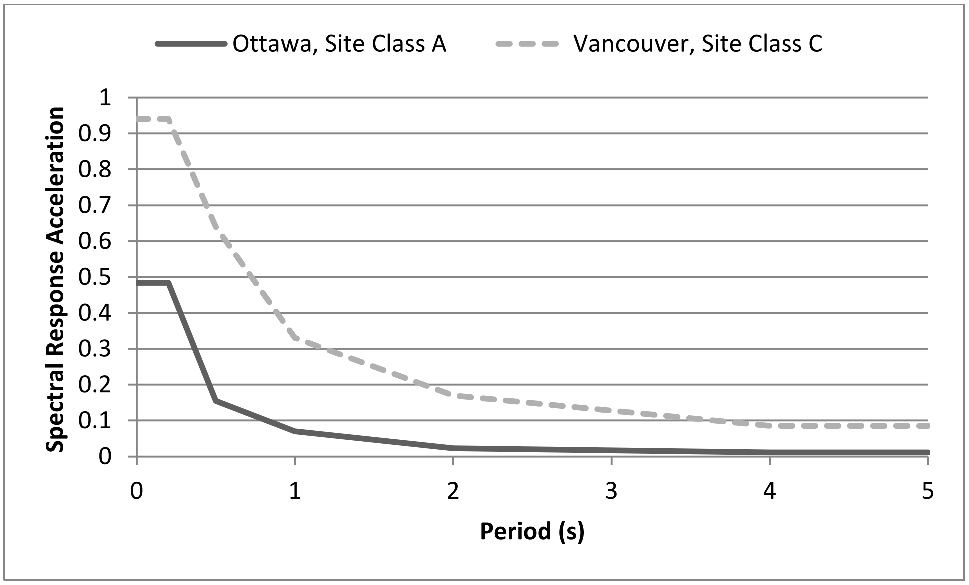
With the excavation already required for the new basement, it is anticipated that the base isolation scheme will be approximately 10 – 15% more expensive than the conventional life safety scheme. However, the base isolation is still the preferred option because it is designed to provide a greater level of performance than the conventional upgrade and to resist 100% of the NBCC loads.

5.12.4 Performance targets

The conventional seismic upgrade is being designed to 60% of the NBCC for life safety only. The base isolation option is being designed to 100% of the NBCC and provides a performance level comparable to a post-disaster level, including heritage preservation.

5.12.5 How forces compare to Ottawa

The Holy Rosary Cathedral is located in Vancouver, British Columbia, and the seismic retrofit designs used the spectral accelerations specified in the 2010 National Building Code of Canada for Site Class C. The Centre Block, bearing directly on rock, is assumed to be Site Class A. Spectral accelerations for Ottawa and Vancouver from the 2010 National Building Code of Canada climatic data are provided below, for their respective site classifications. This graph indicates that for a short period structure, similar to the Centre Block, the demand would be 1.9 times greater for Vancouver, Site Class C, than for Ottawa, Site Class A. For a discussion of relative seismic hazards, please refer to section 5.1.1.



5.13 Lessons Learned

Throughout the literature and experiences of team members who worked on some of these projects, there are a number of lessons learned that can be related to the Centre Block seismic upgrade project. These thoughts for all of the projects researched have been compiled here.

5.13.1 Lessons Learned – New Zealand Parliament House

A lesson learned on the New Zealand Parliament House was the consideration of how much the base isolated building would permanently move in earthquakes that are less severe than the design earthquake, and the impact this would have on architectural, mechanical and electrical services. Designing the non-structural components to the same return period as the structural components was determined to be impractical as the movements could be as high as 400 mm. In the case of the Parliament House, the designers selected a 25 year return period which corresponded to 50 mm of movement at the moat separating the ground from the base isolated structure. All of the non-structural details would need to accommodate this amount of movement (in any horizontal direction) without damage. A similar decision on an appropriate return period would have to be assessed and made if base isolation were selected for the Centre Block.

The New Zealand Parliament House was one of the first seismically isolated retrofits to be done using a formed concrete floor diaphragm above the isolators. The isolators were placed on the tops of large rectangular concrete columns in the basement of the building. The construction complexity of placing the (lead-rubber) isolation bearings was diminished by suspending the bearings from the diaphragm framing, and then casting the columns up underneath the isolators. This scheme has subsequently been used on other base isolation retrofit projects.

5.13.2 Lessons Learned – Salt Lake City and County Building

The first lesson learned is that it is very important to complete a non-linear analysis to assess how tall, slender components of the building will perform with base isolation. The second lesson is that it is important for the structural engineers to work with local contractors to overcome some of the unique challenges that can occur with base isolation. More technical details of these lessons follow.

On the Salt Lake City and County Building, the simplified linear analyses techniques did not well approximate the forces in tall appendages with frequencies of vibration that are higher than the base building (which would be very similar to the Peace Tower, or ventilation and water towers). The yield of the lead core (similar to the stick-slip behavior of a friction pendulum system) excited the higher modes and increased the lateral shears in the tower appendages (spires) in ways that were not captured by the spectral approximations initially used for design. As a result, it was discovered that the tower elements needed bracing. This bracing was much less than would have been required by a conventional retrofit but it was a late discovery and an added expense.

On the constructability side, a construction sequence was developed that allowed the building to be fully supported on its own bearing walls throughout construction. The openings that were established were checked to verify the remaining walls could support the entire weight of the building. The structural engineer worked with local contractors, who had connections with the local mining industry, and they beneficially used non-impactive (rotary) mining equipment to make the large masonry cuts to allow the placement of the reinforcement and rebar cages in the resulting cavities in the piers and walls. This equipment included diamond-encrusted cables and special, large diameter diamond saws. Rock-drilling was also done in order to install horizontal post-tensioned thread bars. It was decided that for any such retrofit the structural engineer would need to develop a detailed proposed construction sequence and load transfer-related vertical movement criteria to ensure the structure would not be compromised. This was the first use of epoxy-injected flat jacks to pre-load the isolators in order to prevent shortening of the rubber units upon load transfer.

5.13.3 Lessons Learned – Victoria Memorial Museum Building

On the Victoria Memorial Museum Building, it became apparent during demolition of the ceiling finishes for the seismic retrofit that the terra cotta flat arch floors could pose a significant risk from falling pieces of mortar and tile. To mitigate the risk, a wire mesh catcher system was suspended from the steel beams. The Centre Block floors are constructed with a nearly identical system of terra cotta flat arches and so the same risk would be present.

Another lesson learned on the VMMB pertained to the installation of the grouted steel anchors in the main entrance. To install the anchors within the walls, holes had to be drilled down the height of the walls. Typically this is done with water added in the hole to minimize the amount of dust and improve the drilling process but this was not possible as the water would have seeped through the masonry and damaged the heritage finishes that they were trying to protect. Consequently, the drilling process was extremely time-consuming and costly, and produced a very large amount of dust all around the site and neighbouring properties during the drilling process. While installing anchors to reinforce the walls of Centre Block could be a way to avoid having to remove any architectural finishes on the surface of the masonry walls, it clearly poses some extreme construction difficulties, especially if imagined on a large scale.

5.13.4 Lessons Learned – San Francisco City Hall

The following are a number of lessons learned on the seismic retrofit of the San Francisco City Hall, which included base isolation:

- A highly refined sequence was required for the cutting of columns and isolator installation, so as not to leave the building too vulnerable to a moderate or larger earthquake during the prolonged construction period.
- Temporary lateral bracing of the building was also required in the basement during the sequence of cutting columns and isolator installation; this was done by way of a design-build specification, so that the contractor would be able to influence the design in order not to present undue obstructions to the basement construction.
- Construction-phase column dead loads (self-weight) needed to be estimated closely to verify the proper jacking effort, so that the isolator installation subcontractor would know when to cut the columns, prior to isolator placement, and would be able to correctly estimate jacking force for the flat jacks to avoid vertical movement of the isolated columns (up OR down).
- Analytically and empirically, an innovative test and modeling process was developed to accurately quantify the shear stiffness and cracking strain in the composite masonry infill; this information was used to inform the optimal selection of isolator properties and also served as a basis for design of supplemental lateral bracing in the office tower (or dome, in the case of SF City Hall), to prevent significant cracking and loss of lateral stiffness (change of calculated modes) in the building. Good use of empirically measured dynamic periods of the building was made in order to “calibrate” or prove out the validity of the analytical model.
- There could be isolator shear distortions caused by production testing in the rubber isolator units with larger cores. These misalignments between top and bottom plates needed to be corrected as a step in the installation process. The alignment has improved over the years, but is never perfect. Consequently it is beneficial to allow some tolerance in the bolted attachments to the isolators or to otherwise allow for possible small offsets in the construction details.

5.13.5 Lessons Learned – General Lessons

Past seismic retrofit projects in the United States have developed a relatively economical installation technique that positions new footings and isolators off-grid from the original columns. The economy of this approach results from allowing the original structure to support the entire weight of the structure until the isolators were pre-loaded. The load transfer to, and pre-loading of, the isolators is accomplished by post-tensioning the concrete transfer elements. This technique of relocating all of the primary support points may be especially advantageous to the Centre Block if there is a desire to modify the basement space.

An important lesson learned for conventional seismic upgrade projects is that is that gaps in information available can have significant impacts on the design of the upgrade. Despite the large number of drawings available for Centre Block, there are still significant gaps of knowledge in the material properties, structural assemblies and particularly in the strength of non-structural components. In order to produce a reasonable design, designers must either make conservative assumptions or do more intrusive testing, both of which can damage the heritage fabric of the building. However, with base isolation, substantially reduced accelerations and stresses will be imposed on the existing building and its components. As a result, even conservative assumptions of material properties and connections may not necessitate intrusive work to the existing building.

6. POTENTIAL SEISMIC UPGRADE METHODS

6.1 Conventional Seismic Retrofit Techniques

Many masonry buildings, both nationally and internationally, have been successfully upgraded to modern design codes for anticipated seismic loads. The most common methods involve the installation of reinforced concrete shear walls or steel braced frames, depending on the soil conditions, building layouts and structural demand. These methods have been proven to be cost effective, and are well understood within both the design and construction communities. These methods have been effectively applied within the Ottawa region on many seismic retrofit projects to increase the performance of the building for life safety.

However, these methods generally require a large degree of interference to the existing structure as they take up physical space within the building. They are typically either added into the footprint of the building or replace existing masonry walls, which are removed. In order to attach diaphragms to the new frames or shear walls, and to brace existing walls for out-of-plane bending, many openings in the existing finishes are generally required. Given the large mass of the Centre Block, the irregular openings, and the limited ability to strengthen the diaphragm without completely replacing the floors, it is likely that many walls or frames would need to be added throughout the building.

Balancing the need to introduce a large number of new structural elements against the desire to limit the interruption of the heritage components will present a significant challenge for the Centre Block project. However, these structural elements may also provide an opportunity to run new architectural (elevators, stairs), mechanical and electrical elements in and around them.

In addition, new concrete shear walls or steel braced frames can generally only provide for the safety of the building and its contents. They provide little additional protection from damage for non-structural elements and there is generally the expectation that the building may not be fully operable after a major earthquake due to damage to architectural, mechanical or electrical systems. Seismic restraint can prevent non-structural and secondary structural elements, such as unreinforced masonry chimneys and vaulted ceilings, in Centre Block from collapsing on the occupants during a seismic event but they may still be considerably damaged.

6.2 State of the Art Analysis and Retrofit Techniques

6.2.1 State of the Art Analysis Techniques

In a linear modal response analysis, the response of the structure is determined from the combination of the mode shapes, with a response from a design spectrum of accelerations applied to each mode based on its frequency. The forces are then applied to a model with linear elastic properties and non-linear effects are accounted for by a code specified reduction factor. This method is commonly used in Canada because of its relative ease of use for the designer.

Non-linear time history analysis uses earthquake ground motion records applied to a computer model that incorporates the non-linear, inelastic properties of the structure. This analysis technique has been used in seismic engineering for over 25 years and is still widely recognised as the analysis technique providing the most probable seismic response of building structures. It is required for base isolated design, but also used for conventional seismic designs when improved analysis techniques are desired.

6.2.2 Base Isolation Seismic Retrofit

Based on the literature review, incorporating base isolation into a seismic retrofit appears, by a significant margin, to be the most common "non-conventional" system of improving the building's performance. Starting in the 1970's, many buildings throughout the world have successfully installed base isolation and several have been successfully tested in earthquake events. The first base isolation projects in Canada are currently underway in Vancouver but no base isolation projects have yet been completed elsewhere in Canada to our knowledge. There is a significant trend towards base isolation becoming a commonplace solution to addressing seismic risk as owners become better aware of the benefits it offers and designers become more familiar with the technology.

The main benefits of base isolation are that it: minimizes interference with the existing architectural space and heritage finishes, reduces damage to non-structural elements in the event of an earthquake, and prevents the failure of brittle structural elements (such as unreinforced masonry) under high deformations imposed by seismic forces. All of these benefits would be applicable to a seismic retrofit project at the Centre Block.

In most seismic retrofit projects that have included base isolation, there is still a need for a reduced amount of interference in the building, such as strengthening the floor and roof diaphragms or introducing a limited scope of steel bracing or shear walls. It is not possible to say without a more detailed analysis whether additional work would be required for the Centre Block if the building were base isolated.

In most base isolation projects, a rigid floor diaphragm is required immediately above the base isolators; for the Centre Block, this could involve slab thickening beneath the main concrete floor with or without new concrete beams. This new floor structure may provide an opportunity to relocate the existing structural elements (piers and walls) within the basement space.

6.2.3 Friction and Viscous Damper Seismic Retrofit

Friction and viscous dampers are another more recent element of seismic force resisting systems that has been used to seismically retrofit existing buildings. However, no examples of heritage masonry structure retrofits were found that used friction or viscous dampers. In addition, the literature suggests that friction or viscous dampers are most effectively applied to existing steel or concrete moment frame buildings due to practical constraints on how and where the dampers are installed. The large degree of movement required to engage the dampers is also a concern for brittle masonry structures that may not be able to resist large deformations without collapse. Friction and viscous dampers are generally not an efficient seismic upgrade option for the Centre Block's or Peace Tower's type of construction.

6.2.4 Drilled Steel Reinforcing Seismic Retrofit

Drilling vertical steel reinforcing anchors into existing walls is another technique that has been used to seismically retrofit both concrete and masonry walls by adding tensile capacity to the walls. This technique has the advantage of requiring very little interference to the existing finishes or final appearance of the building. However, the inability to add water when drilling the holes if existing finishes are left in place significantly complicates the construction and produces a large amount of dust all around the site. It also provides no additional protection for non-structural components of the building. This seismic retrofit option may be a possible option for both the Centre Block and Peace Tower, but construction complications and impact on the adjacent buildings that will still be operable during construction will need to be very carefully considered.

6.2.5 Fibre Reinforced Polymer Seismic Retrofit

Fibre reinforced polymer (FRP) fabrics work similarly to steel anchors by adding tensile capacity to the masonry walls. However, as an overall retrofit scheme, they would require a very large degree of disruption to the heritage fabric of the Centre Block if applied, and in the final state, may be required to be visible on the exterior face of the stone masonry, altering the appearance of the building. No additional protection for non-structural components would be added. However, FRP may be used selectively in the building to address connection or load path deficiencies, such as diaphragm connections, in combination with another retrofit scheme.

7. CONCLUSIONS

The Centre Block building was constructed at a time when there was little or no understanding of the impact of earthquakes on the design of building structures. The typical stone and brick masonry walls and terra cotta arch floors that were common to that era are particularly sensitive to damage or collapse during a seismic event. Due to past experiences and a better understanding of the seismic risk, many owners of historic buildings throughout the world have made the decision to complete seismic upgrades.

Conventional seismic upgrades of both heritage and non-heritage structures in the past have typically involved the use of concrete shear walls and/or steel braced frames. These methods have been proven to be cost effective, and are well understood within both the design and construction communities. These methods have been effectively applied within the Ottawa region on many seismic retrofit projects. However, these methods generally require a large degree of interference to the existing building and heritage fabric as they take up physical space within the building and must be anchored to the existing structure.

Base isolation is becoming increasingly common as a seismic retrofit method as owners and engineers better understand the higher performance level it offers, and its ability to reduce the intervention in the building in the upgrade process. Experience in the implementation of base isolation has also improved designs and decreased the cost of installation for many projects. In most seismic retrofit projects that have included base isolation, there is still a need for a reduced amount of interference in the building, such as strengthening the floor and roof diaphragms, introducing a limited scope of steel bracing or shear walls, or improving connections and load path.

Friction and viscous dampers were reviewed and found to not typically suit very stiff brittle masonry wall structures. Adding tensile and shear capacity to the unreinforced masonry has been achieved on limited projects with drilled steel anchors and fibre reinforced polymer (FRP) fabrics, but have been generally limited in their usefulness, especially for heritage structures. They can both be effectively used to address selective issues of connection or load path deficiencies.

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APPENDIX A: Examples of Seismic Upgrade Details

New Zealand Parliament House[2]

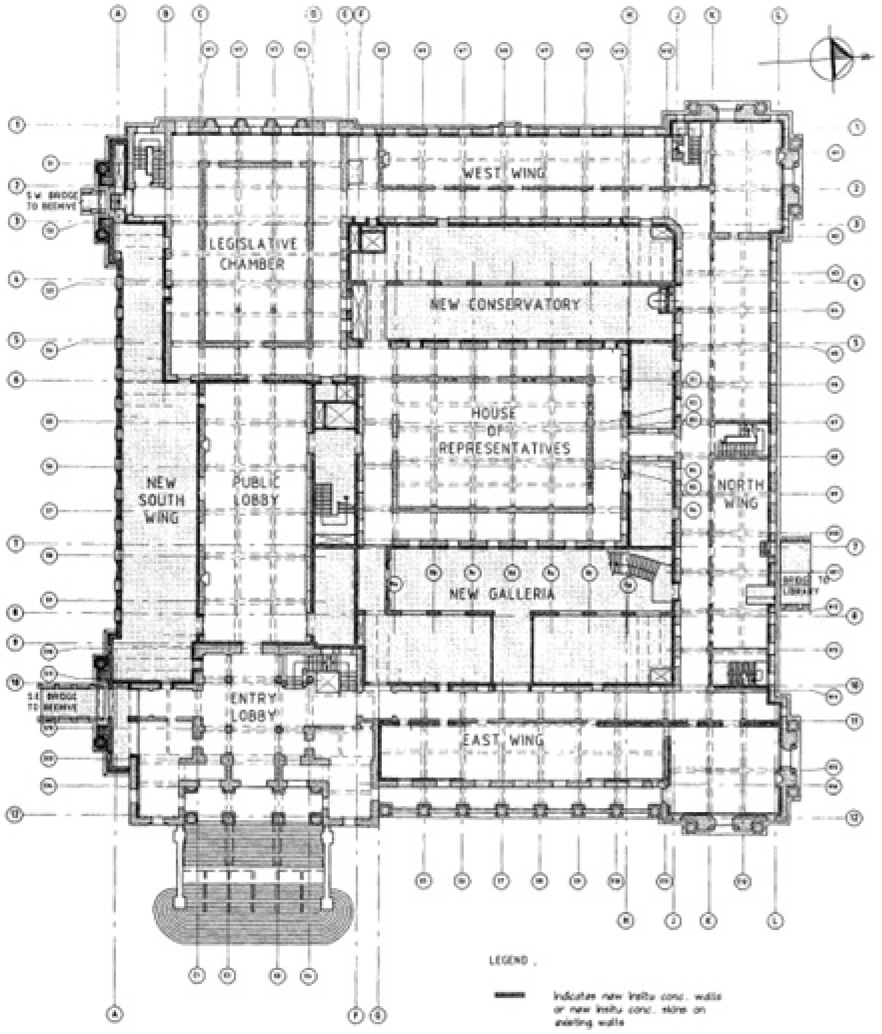


Figure 2 Parliament House, Level 1 Floor Plan.

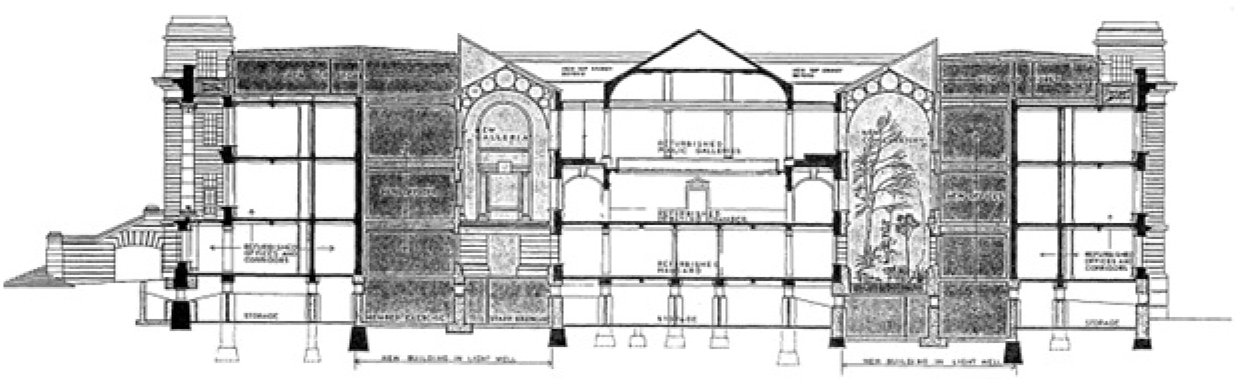


Figure 3 Section through Parliament Building showing proposed strengthening and refurbishment.

New Zealand Parliament House[2]

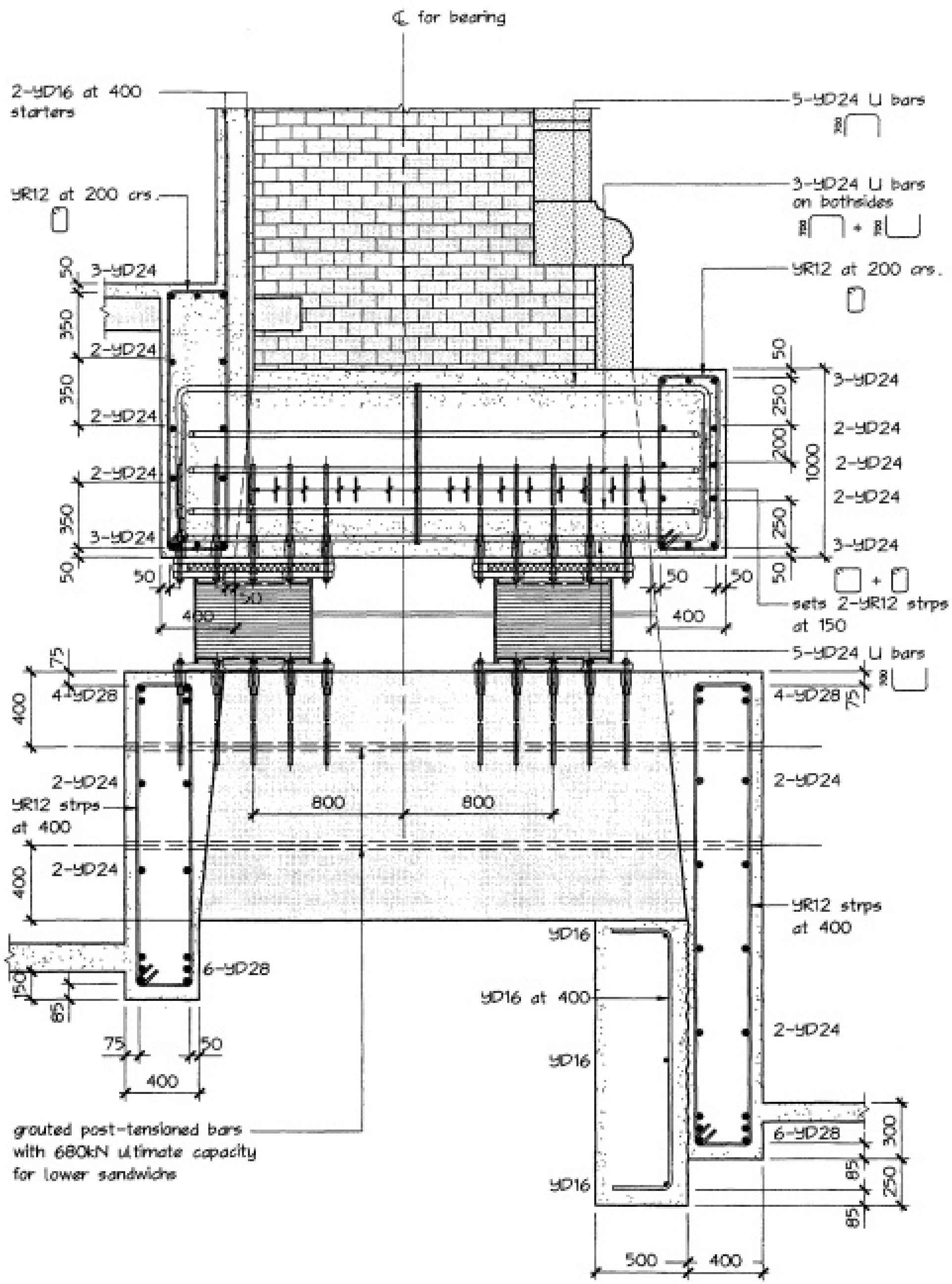


Figure 5 Section through exterior north wall at bearings.

New Zealand Parliament House[2]

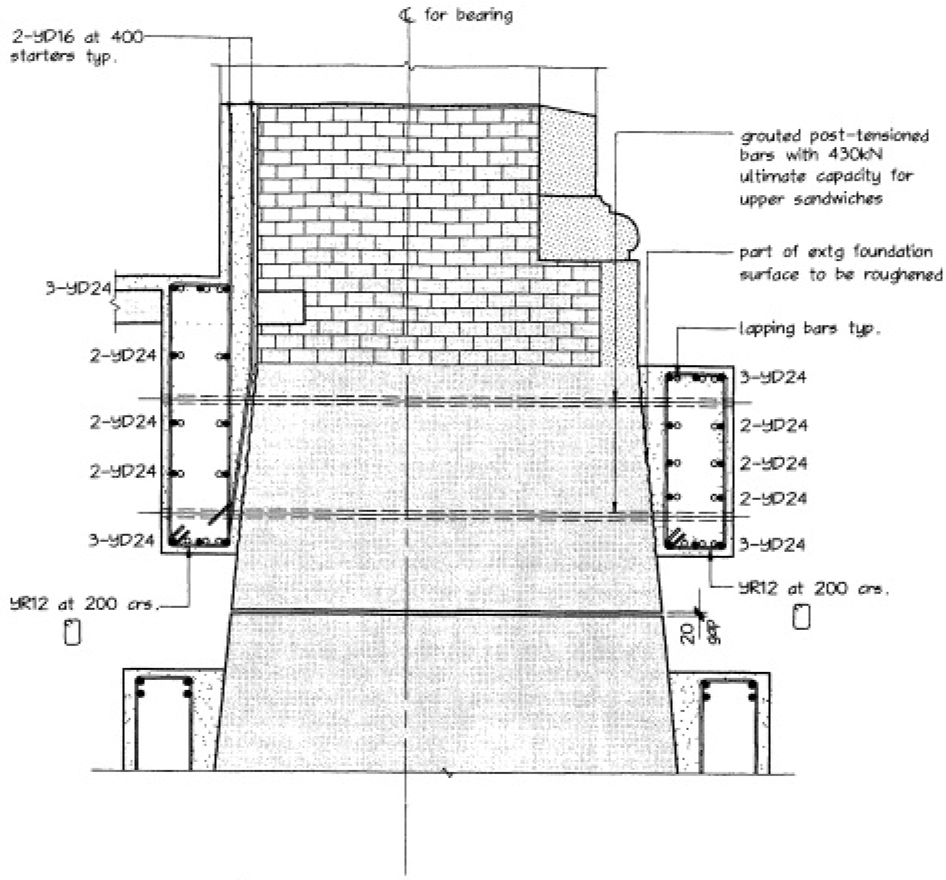
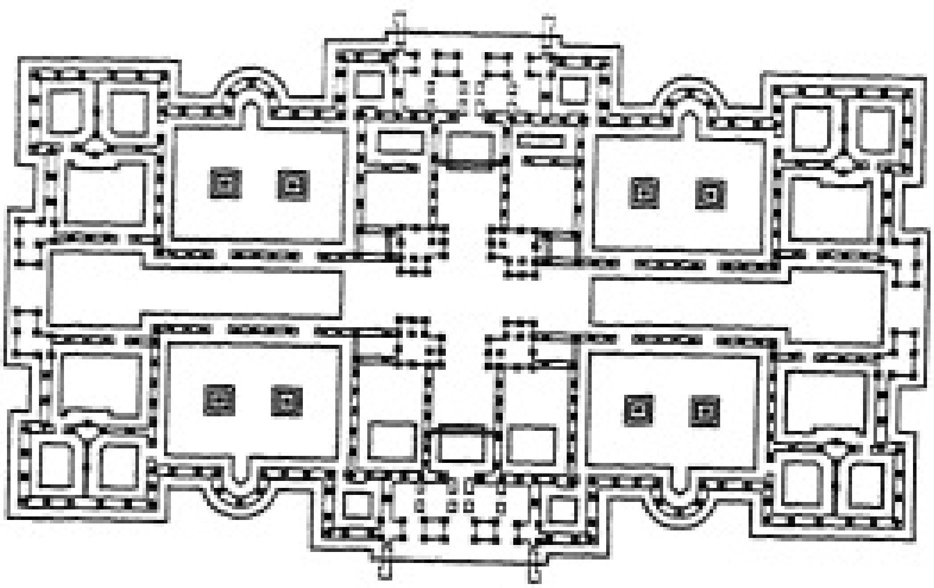
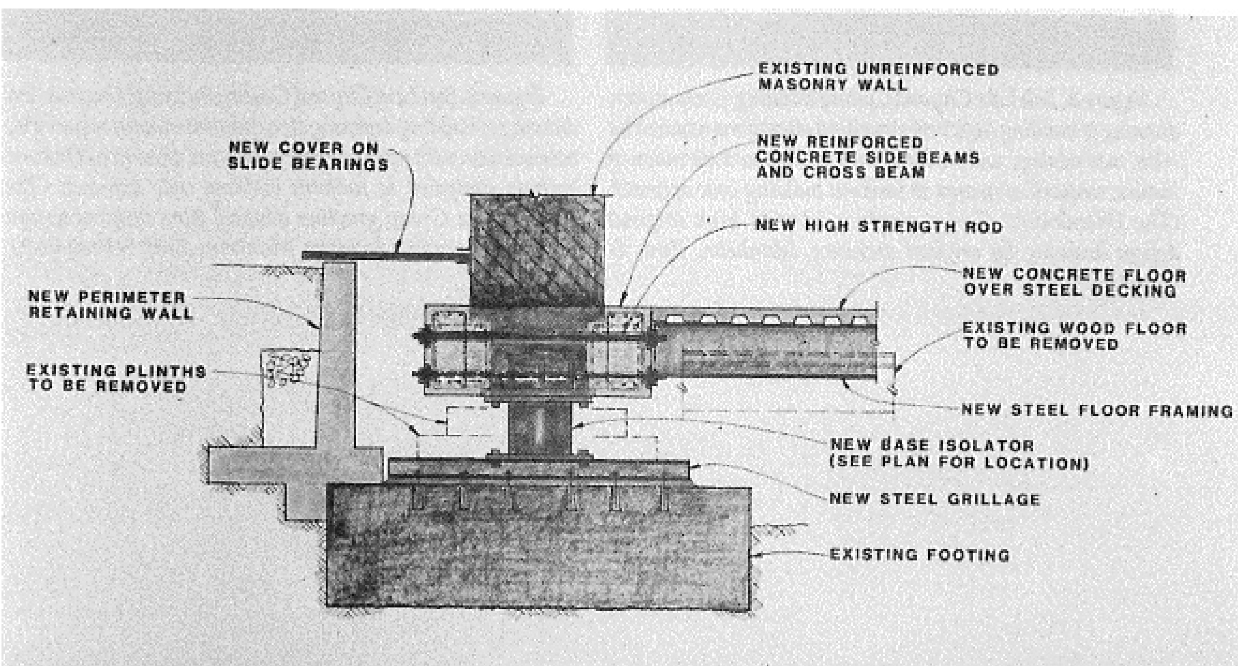


Figure 6 Section through exterior north wall between bearings.

Salt Lake City and County Building[3, 4]

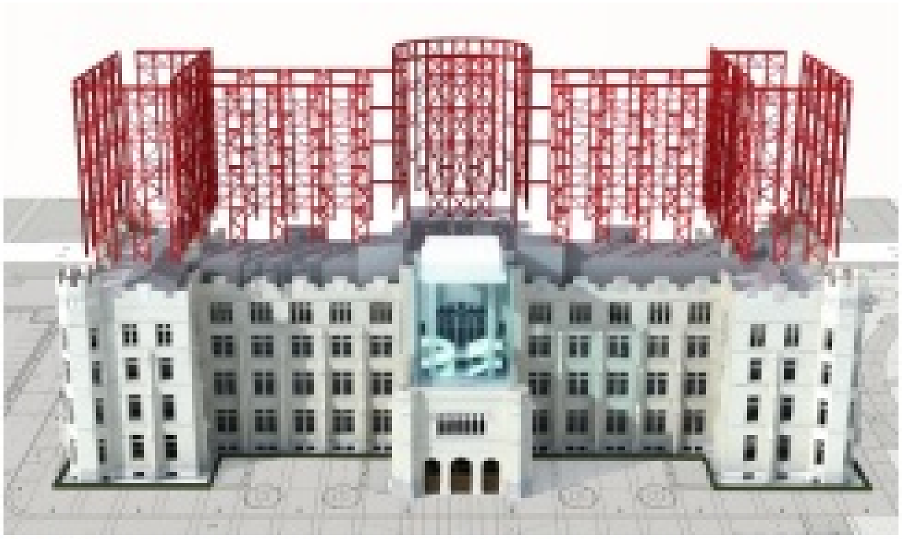


SCHEMATIC BASE ISOLATOR LOCATIONS

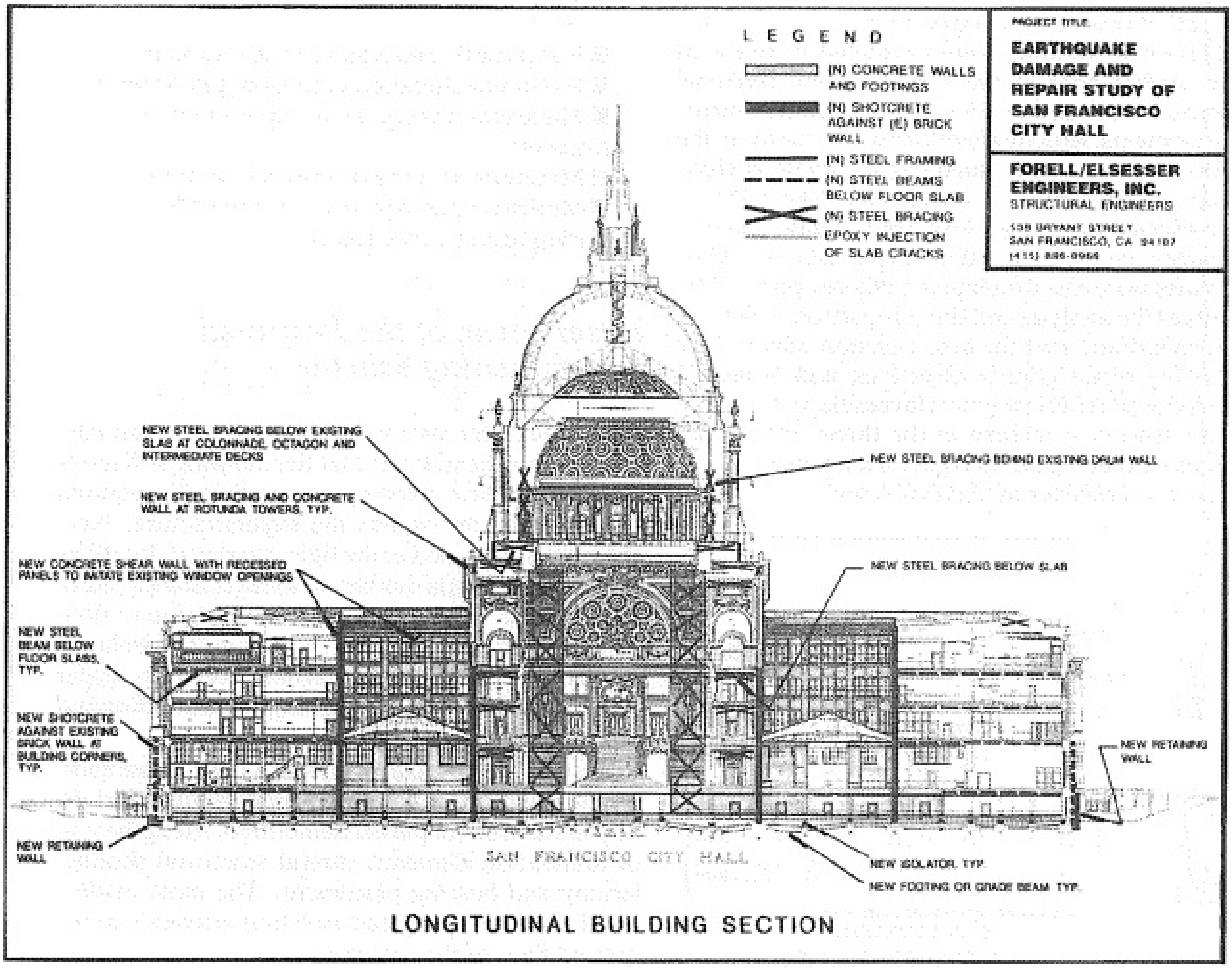


TYPICAL ISOLATOR AT EXTERIOR WALL

Victoria Memorial Museum Building[6]



San Francisco City Hall[7]



San Francisco City Hall[7]

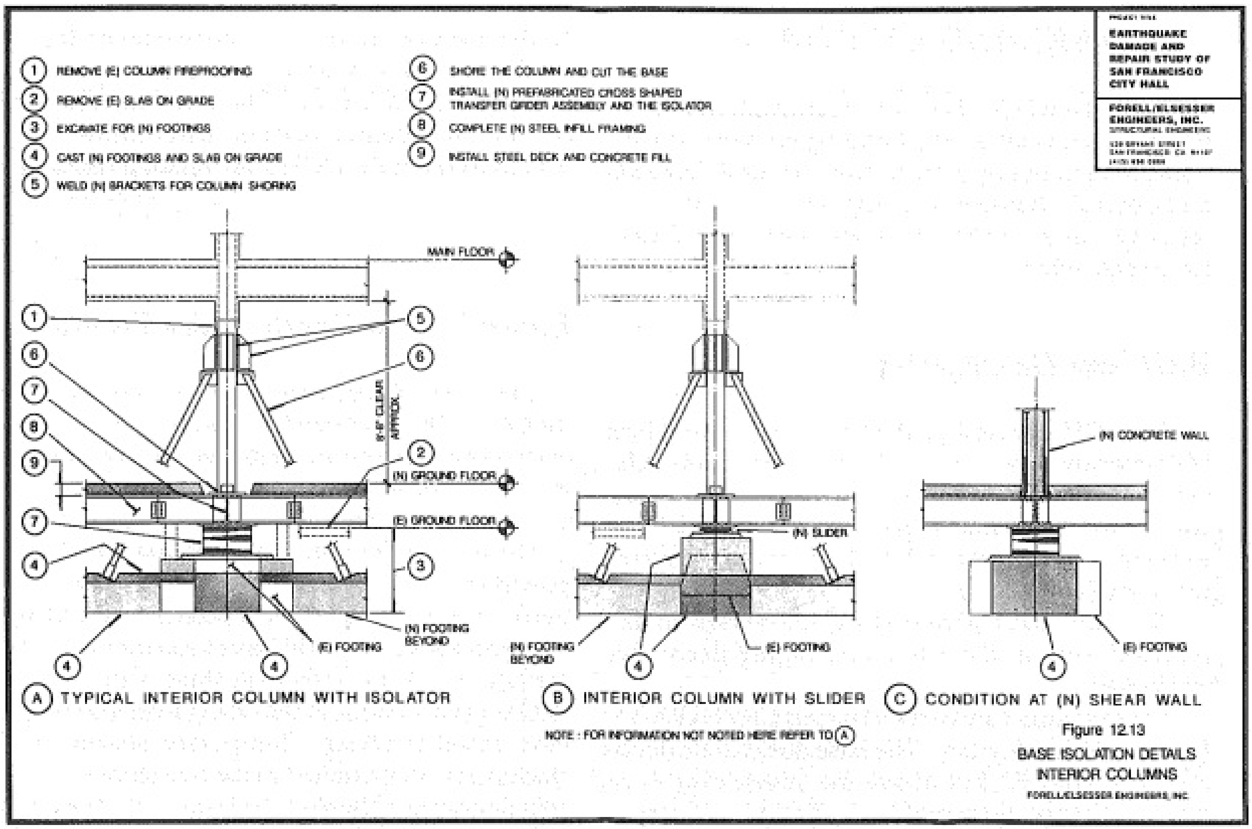
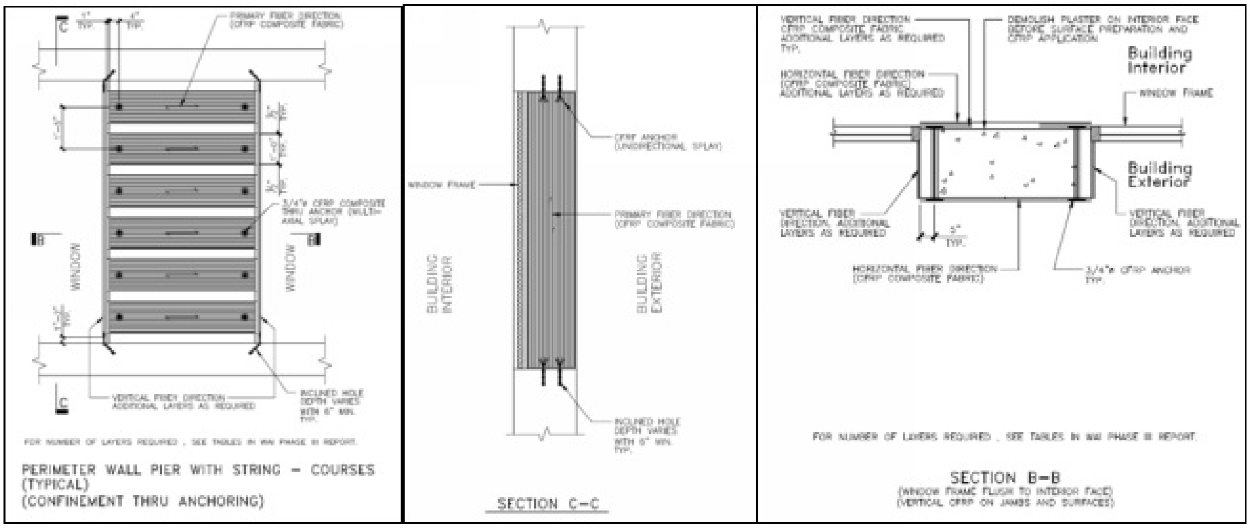
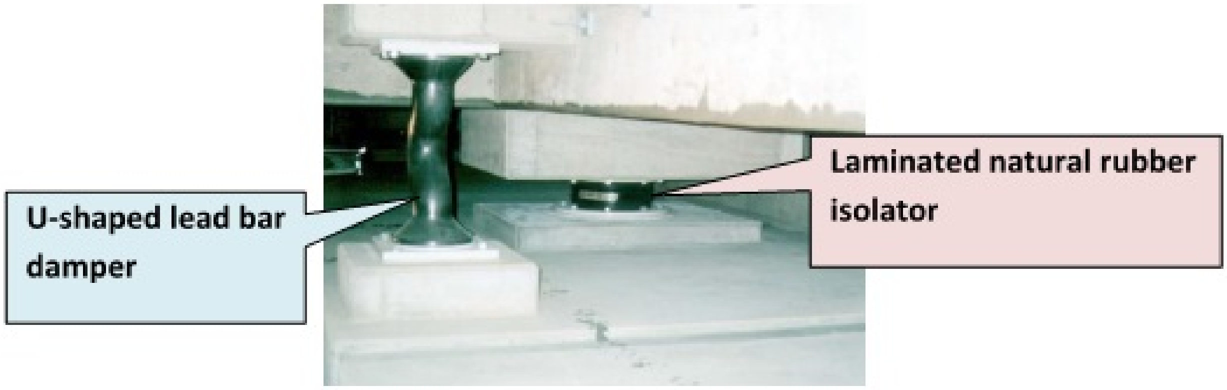


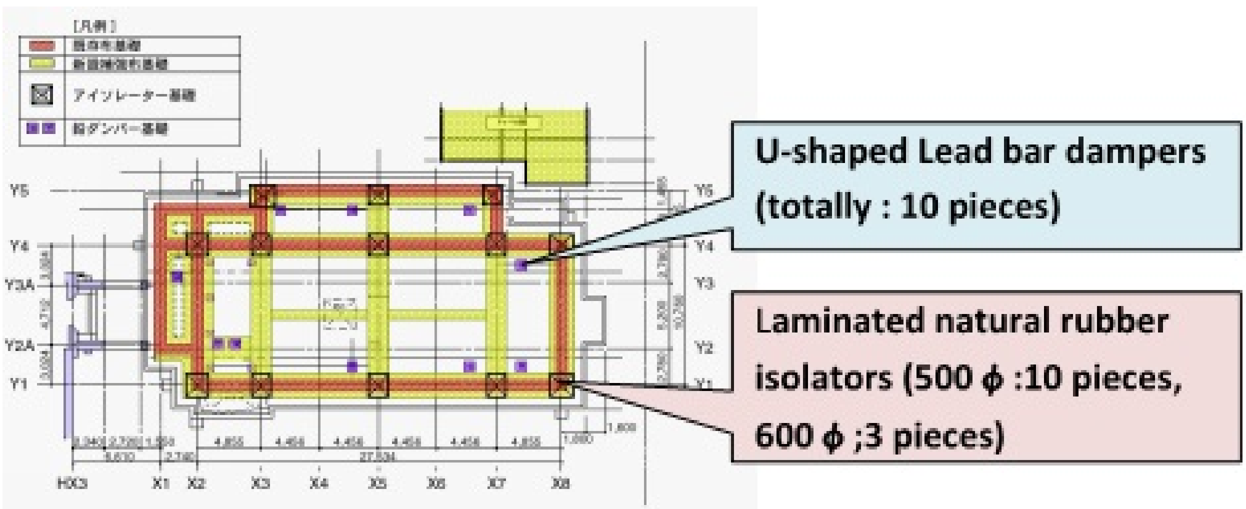
Figure 12.13
BASE ISOLATION DETAILS
INTERIOR COLUMNS

Sage Hall and Gould Hall[8]

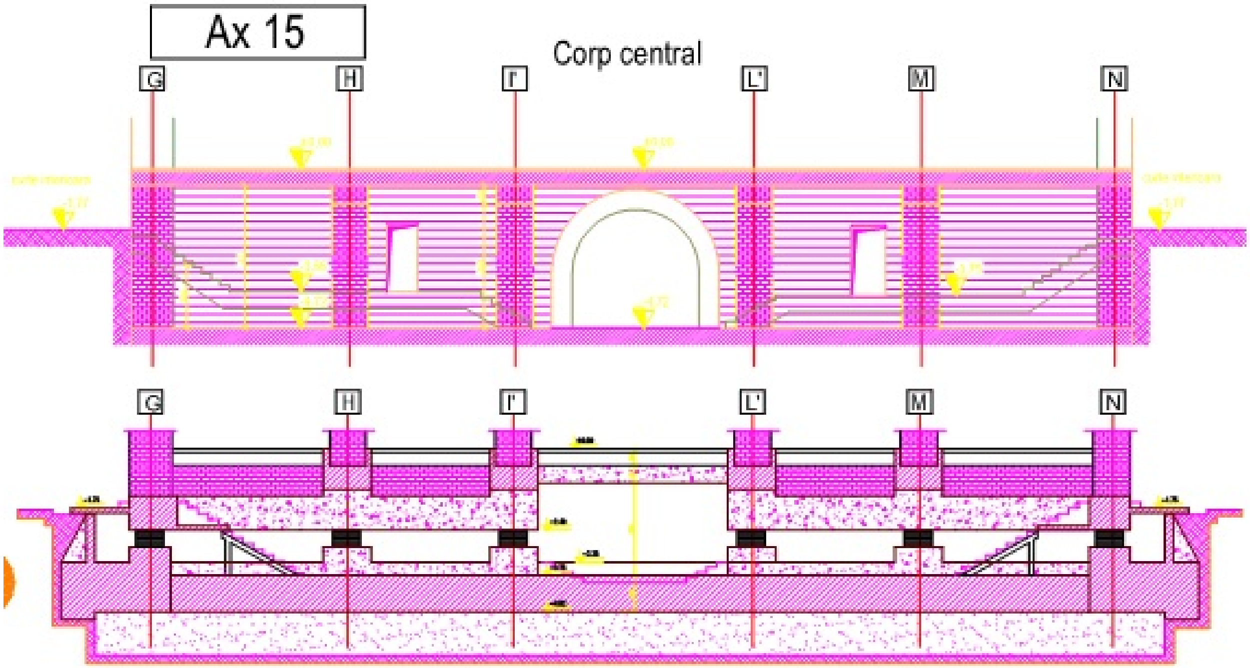


Rikkyo University Chapel[10]





Bucharest City Hall[11]



California State Capitol[12]

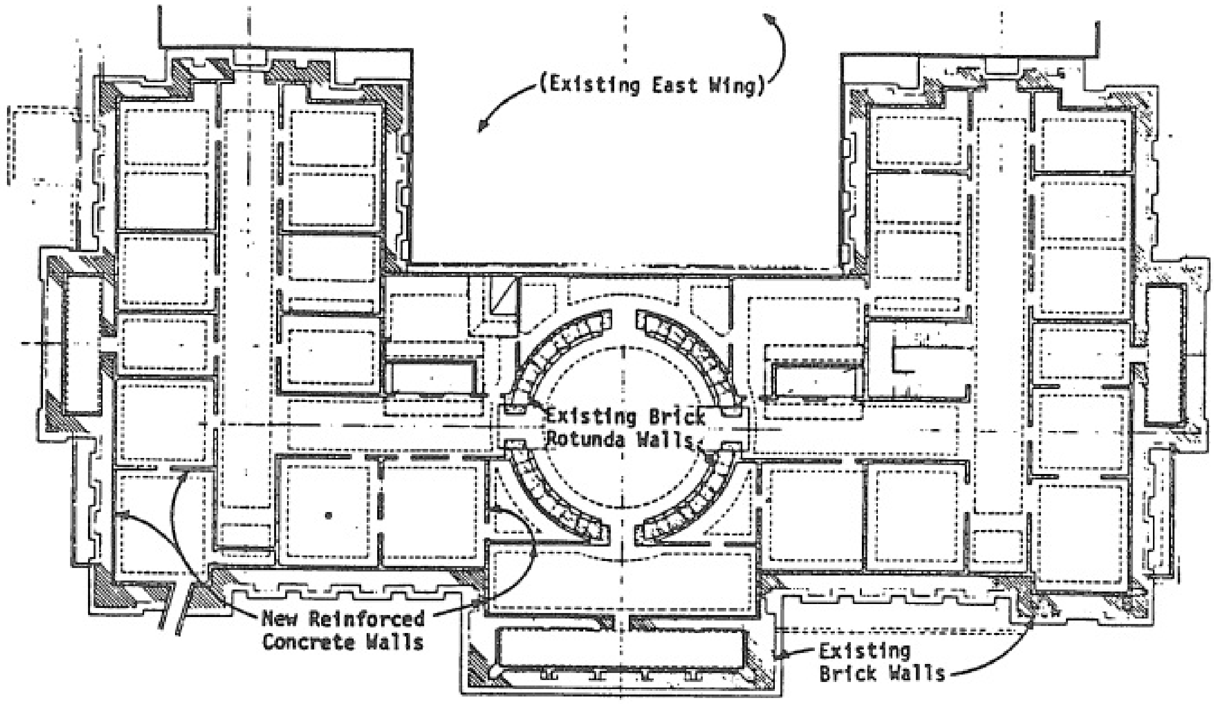


Figure 3. Foundation and basement plan.

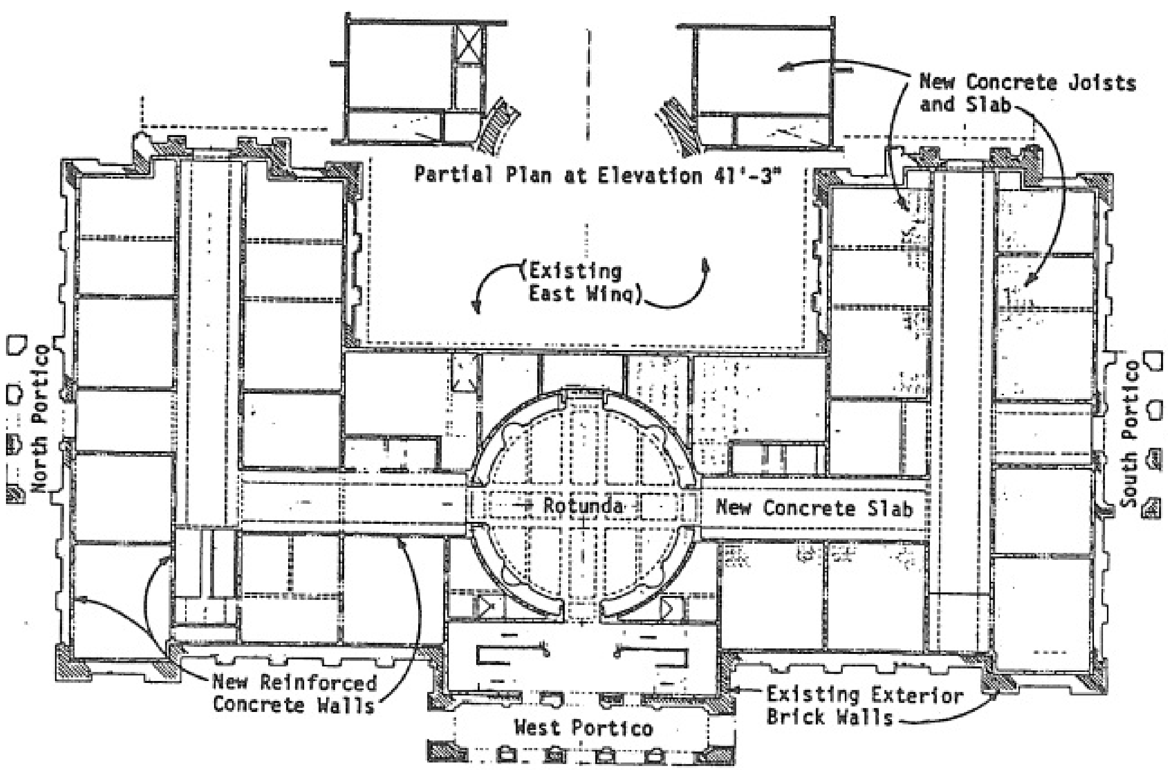


Figure 4. First floor framing plan.

California State Capitol[12]

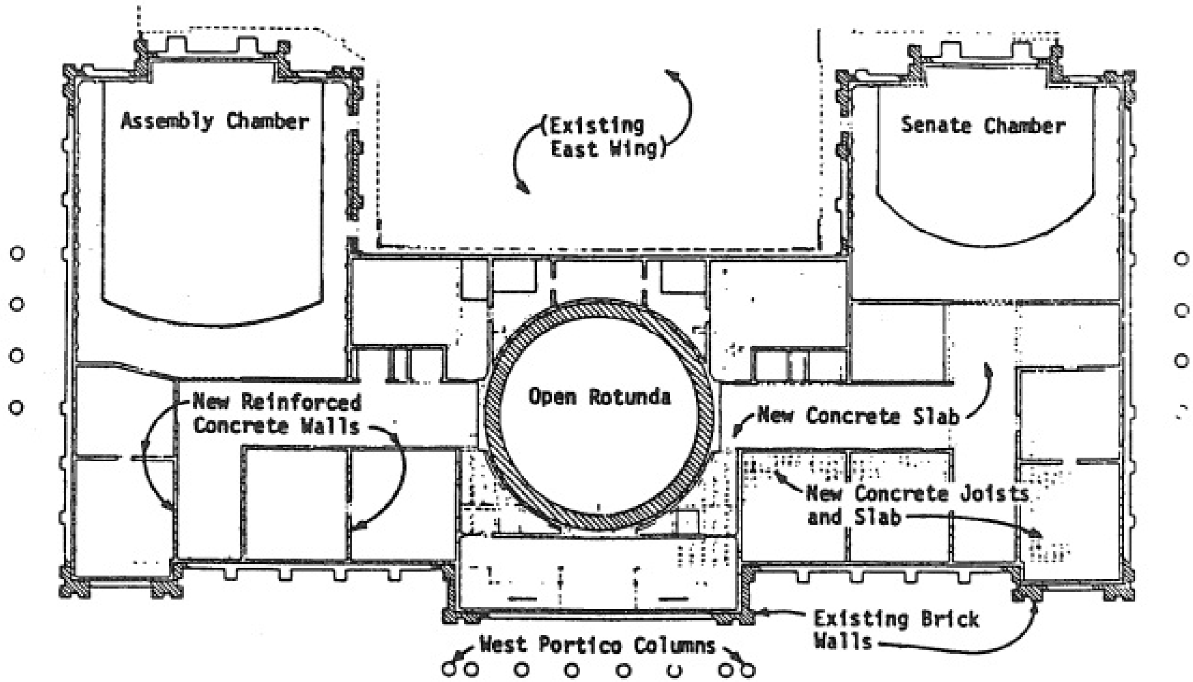


Figure 5. Third floor framing plan.

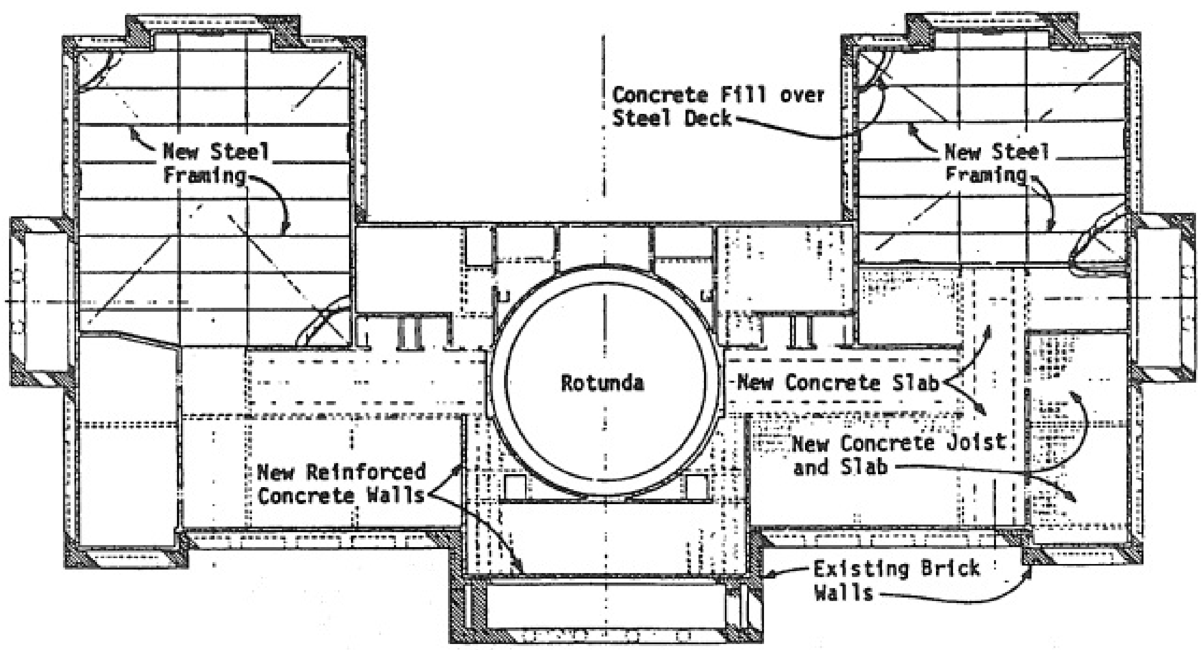


Figure 6. Fourth floor framing plan.

California State Capitol[12]

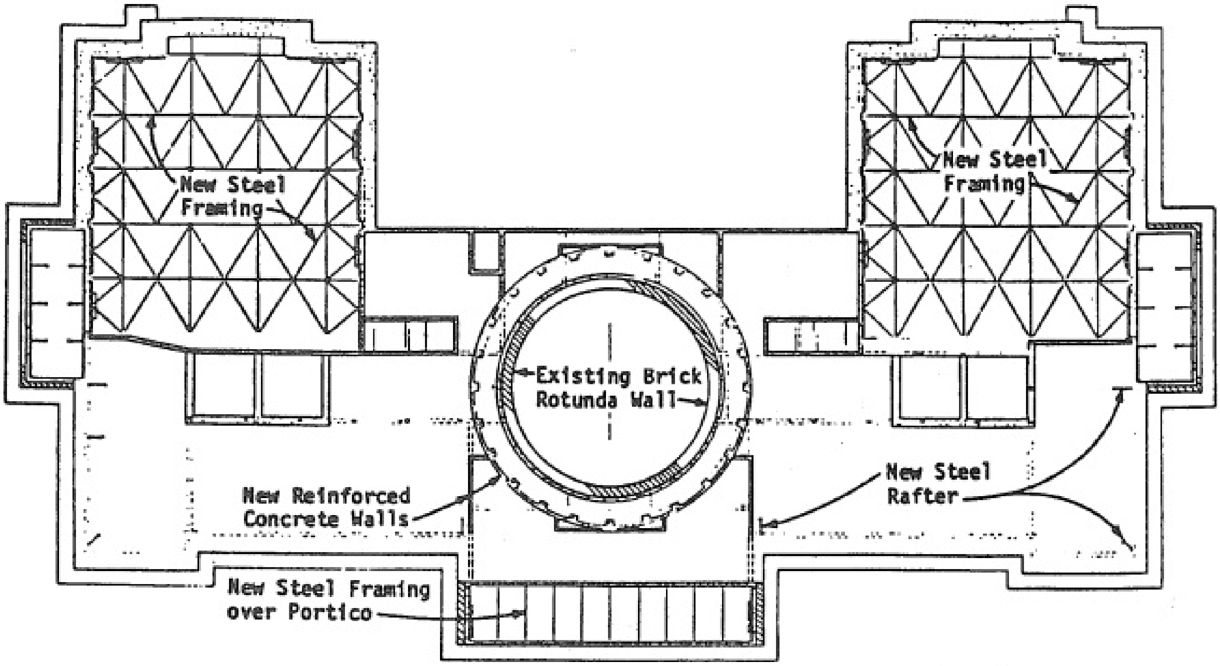


Figure 7. Fifth floor framing plan.

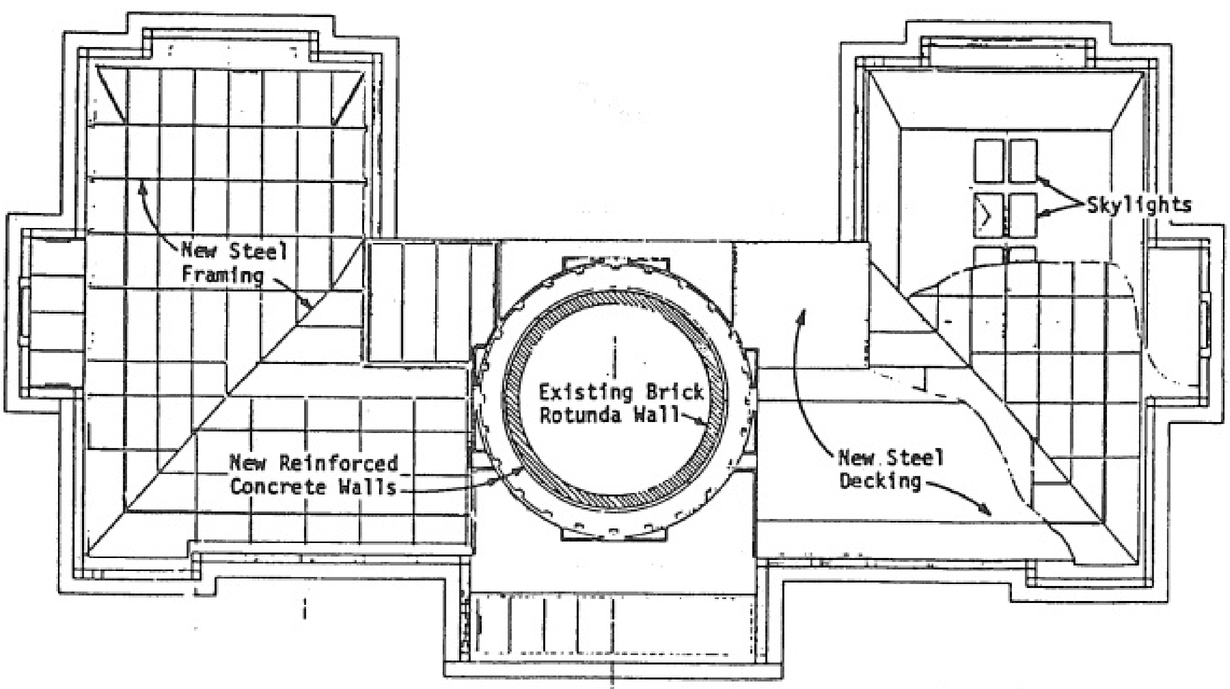


Figure 8. Roof framing plan.

California State Capitol[12]

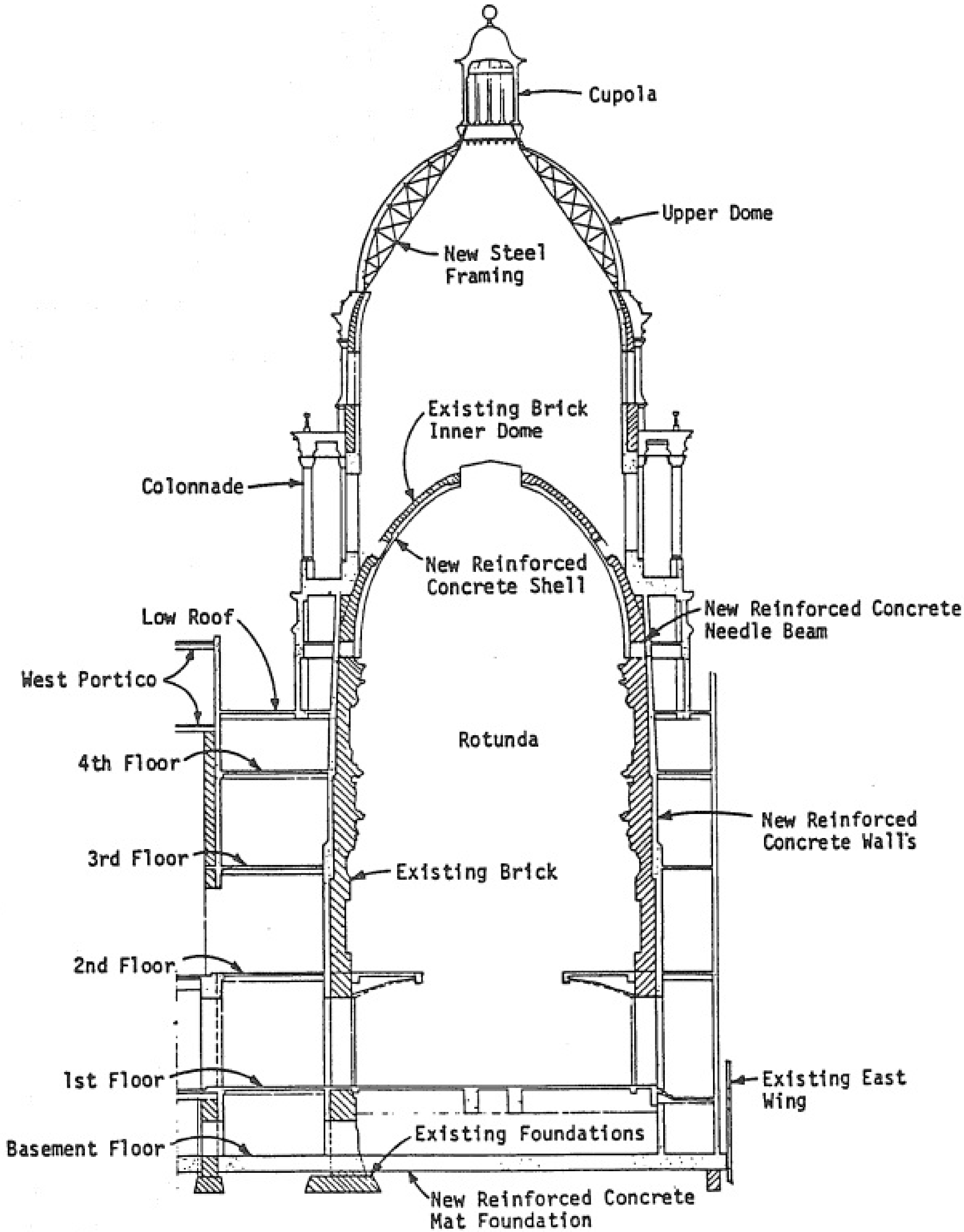
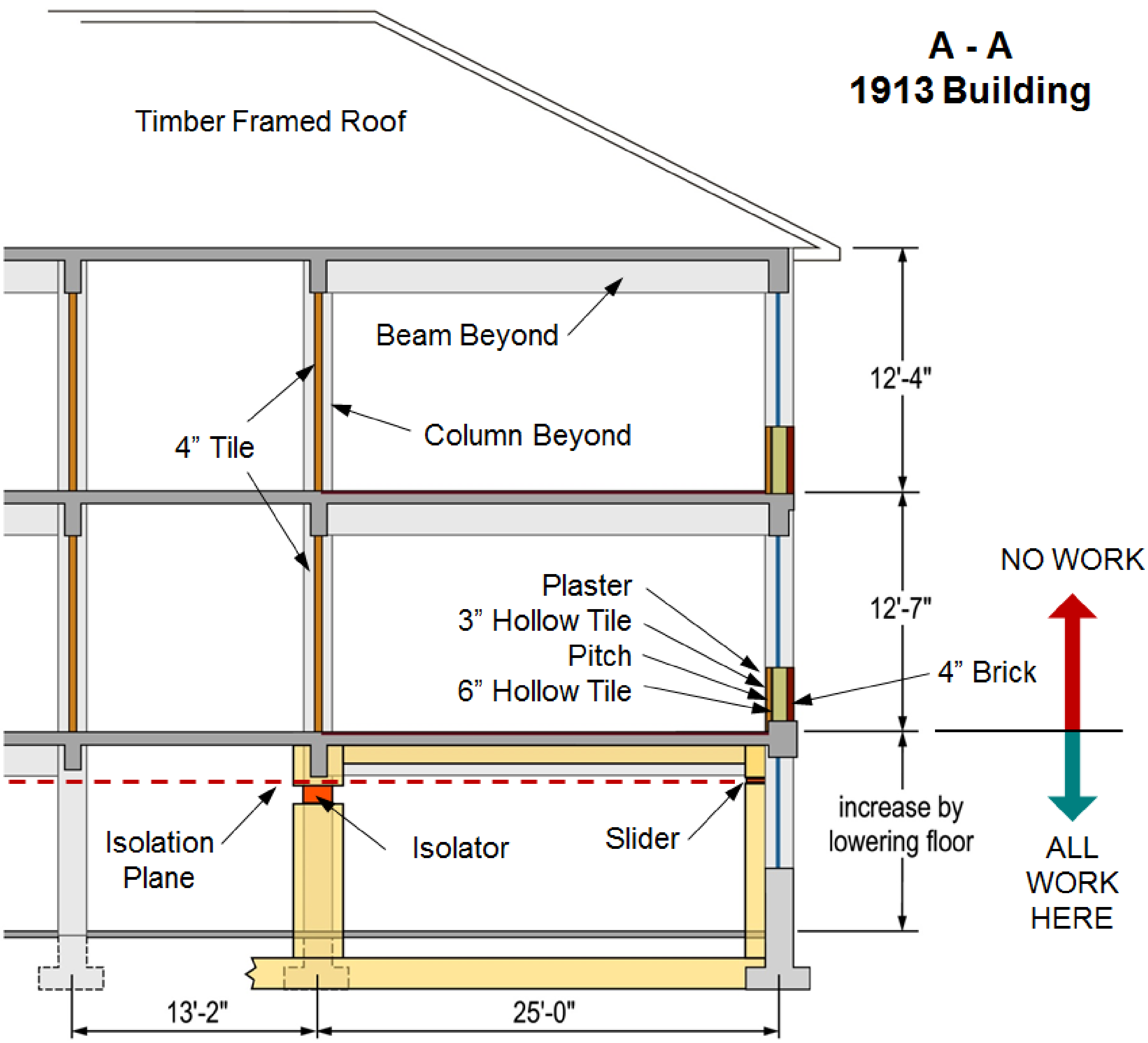
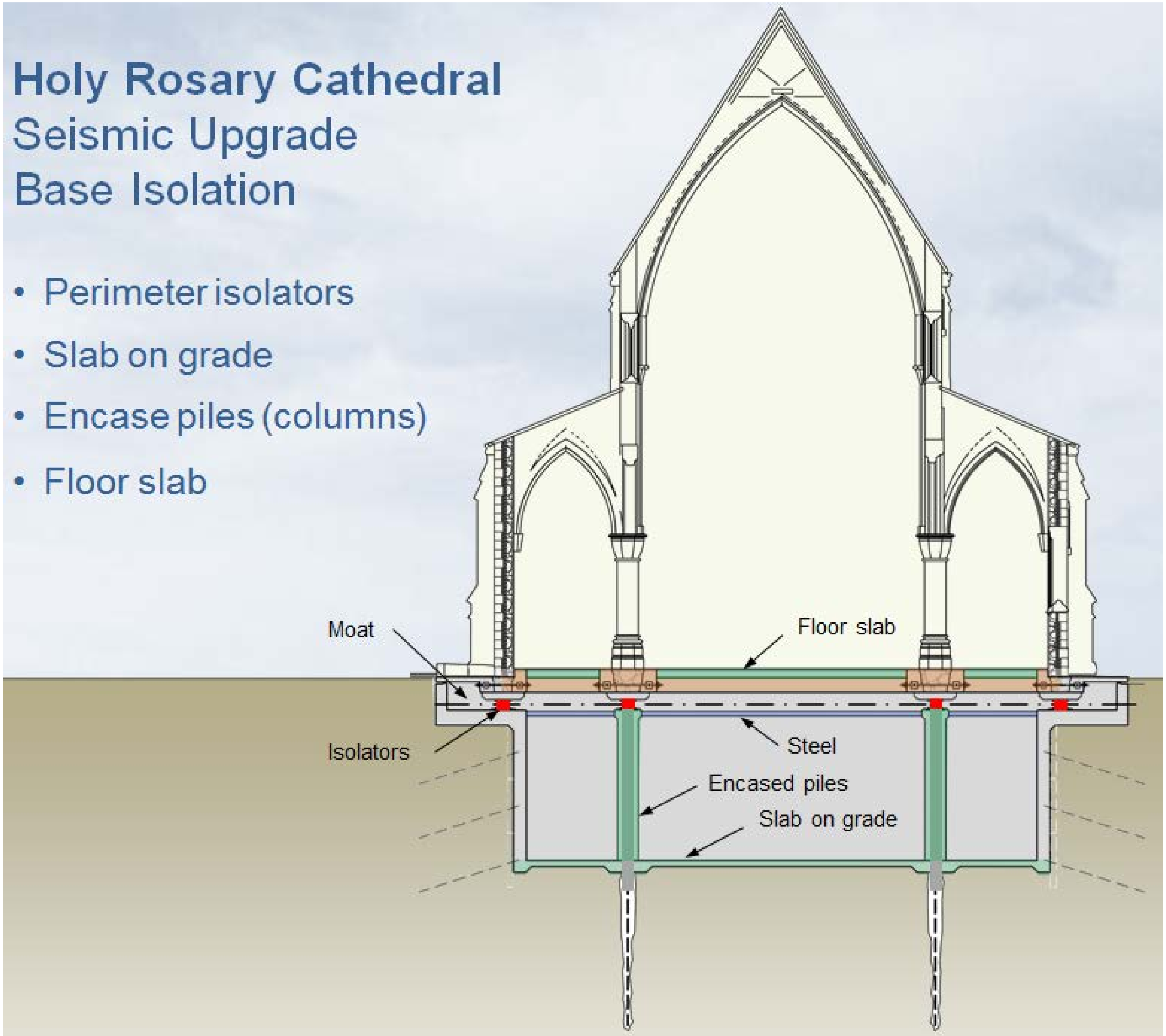


FIGURE 10 TYPICAL SECTION THROUGH ROTUNDA AND DOMES

Lord Strathcona Elementary School



Holy Rosary Cathedral



Holy Rosary Cathedral

New Shear Walls