Technical guide for evaluation of seismic force resisting systems and their force modification factors for use in the National Building Code of Canada with concepts illustrated using a cantilevered wood CLT shear wall example

DeVall, Ron; Popovski, Marjan; McFadden, Jasmine B. W.

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Technical Guide for Evaluation of Seismic Force Resisting Systems and Their Force Modification Factors for Use in the National Building Code of Canada with Concepts Illustrated Using a Cantilevered Wood CLT Shear Wall Example

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To the memory of Dr. Ron DeVall.

Authors:

Ron DeVall (deceased), Ph.D., P.Eng., Read Jones Christoffersen Ltd, Canada

Marjan Popovski, Ph.D., P.Eng., FPInnovations, Canada

Jasmine B.W. McFadden, Ph.D., P.Eng., National Research Council Canada, Canada

Reviewers:

Andy Buchanan, Ph.D., University of Canterbury (Professor Emeritus), PTL Structural Consultants, New Zealand

John W. van de Lindt, Ph.D., F.ASCE, Colorado State University, U.S.A

Tobias Smith, Ph.D., PTL Structural Consultants, New Zealand

Tony Yang, Ph.D., P.Eng., University of British Columbia, Canada

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1. Introduction and objectives

The objective of this guideline is to provide a simple, systematic, and sufficient procedure for evaluating the performance of Seismic Force Resisting Systems (SFRSs) and to determine the appropriate ductility-related (R_d) and over-strength related (R_o) force modification factors for implementation in the National Building Code of Canada (NBC). The procedure relies on the application of non-linear dynamic analysis for quantification of the seismic performance of the SFRS. Note that the procedure is also suitable for assessing force modification factors (R_dR_o values) of systems already implemented in the NBC.

The audience for this guideline are those (called the "project study team" in this document) who submit proposals for new SFRSs with defined R_dR_o values to the NBC for inclusion in Subsection 4.1.8., Earthquake Loads and Effects, of Division B of the NBC. This guideline can also be used by a team performing an alternative design solution for a specific project and seeking acceptance from authority having jurisdiction. In such cases, not all aspects of this guideline (e.g., having different archetypes) will be needed.

The procedure requires the study of a variety of combination of parameters that may affect the seismic performance of a structure. These include:

- the total building height,
- the number of floors,
- storey height,
- the number of bays of the SFRS,
- the aspect ratios of the SFRS,
- the location and types of the energy-dissipative elements used, and the cyclic non-linear inelastic behavior of those energy-dissipative elements,
- the different locations or regions in which the building is located, and
- different earthquake intensities, etc.

This is, in effect, a large sensitivity study of the above parameters to determine how they affect performance when the structure is designed using the proposed R_dR_o values.

A knowledgeable and experienced peer review panel is integral to the procedure and must be involved from the initial concept stages to the determination of the conclusions and recommendations.

The roles of the project study team and the peer review panel are as follows:

- **Project study team**. This team proposes the SFRS and is responsible for:
  - implementing and documenting the work outlined in this guideline,
  - producing a summary study report containing their conclusions and recommendations, and
  - resolving all peer review panel comments to the satisfaction of the panel.

- **Peer review panel**. This is an independent group comprised of at least three individuals with expertise on the topic being studied. Members of the panel must be qualified to critically evaluate the system proposed by the project study team, including testing, design and analysis. The involvement of the peer review panel starts at the conceptual stage and extends throughout the process and must include a sufficient number of recorded meetings to allow the panel to produce a letter containing their opinion of the work, their findings and the recommendations of the project study team.

The results of the study are to be recommendations that satisfy the life safety and performance expectations within the NBC for the range of variables considered and include:

- recommendations for R_dR_o values and any required limitations on building height, irregularities, and seismic zone intensities for inclusion in the NBC,
- recommendations for calculating the fundamental period of the system for the NBC and how they compare to existing NBC values,
- recommendations for CSA material standards (e.g., CSA O86,"Engineering design in wood" for timber SFRS), such as the detailing of energy-dissipative elements, designing for higher mode shear effects, the definition of over-strength demands, limits on SFRS aspect ratios, restrictions on the behavior of energy-dissipative elements, and other information the study has shown to be important to achieving the desired performance.

The proposed procedure in this document is a simplified version of the methodology described in FEMA P-695 (2009), "Quantification of Building Seismic Performance Factors."

The focus of the guideline is for systems with the goal of including them in the NBC and CSA standards framework. It may also be used to show that a proprietary SFRS qualifies for an existing R_dR_o value in Table 4.1.8.9., SFRS Ductility-Related Force Modification Factors, R_d, Overstrength-Related Force Modification Factors, R_o, and General Restrictions, of Division B of the NBC, as permitted in Sentence 4.1.8.9.(5), SFRS Force Reduction Factors, System Overstrength Factors, and General Restrictions, of Division B of the NBC, in which case the new SFRS would need to meet all of the same requirements and constraints in Article 4.1.8.9. of Division B of the NBC as the existing R_dR_o system. However, if not named in the NBC, the proposed SFRS would have to be accepted by each authority having jurisdiction at the location in which it is intended to be used.

This guideline is general in nature and can be used to develop R_dR_o values for a range of systems and materials. However, to illustrate its concepts and application only, a timber SFRS example is used to develop the appropriate technical information required for implementation of the procedure for such an SFRS in the NBC and CSA O86. The actual detailed design and non-linear analysis is beyond the scope of this guideline.

A simple example outlining and illustrating the ideas and concepts using a cantilever balloon-type cross-laminated-timber (CLT) shear wall is included in Sections 2, 3 and 5 of this document.

2. Development of suite of 2D archetypes and defining energy-dissipative elements and/or energy-dissipating mechanisms

The structural archetypes chosen should reflect a broad range of configurations and parameters that capture most practical cases and that are representative enough of the design space that the peer review panel is willing to accept and join the project study team in their recommendation of the proposed R_dR_o values and design rules to the NBC and the appropriate CSA design standard.

Examples of configurations or parameters are:

- different total heights of the system,
- different storey heights,
- different SFRS aspect ratios,
- different vertical gravity loads on the SFRS compared to the total lateral "weight" assigned to the SFRS,
- different compression stresses in the SFRS,
- different earthquake hazard regions,
- different soil conditions, and
- irregularities[(1)].

(1) R_dR_o values are intended for use in buildings that use a static or response spectrum analysis and are not meant to apply to all building configurations. The NBC has limits on various types of irregularities, but some systems may satisfy these requirements and still need additional restrictions applied according to the respective CSA standard. Examples might include limiting the size of openings or their locations in the walls or requiring strong, stiff, continuous columns to spread yielding in SFRSs prone to single storey sway failures at high R_d values (such as braced frames).

The importance of these parameters may vary with the system chosen, for example:

- For cantilever flexural wall systems, different floor heights may not be important, while different vertical gravity loads resisting different amounts of the overturning demand may be.
- For braced frames different compression stresses in the braces may not be important, while the compression stresses due to overturning in shear walls may be, in which case crushing in the compression zone may need to be limited.

The configurations noted above will be influenced by the choice of SFRS, the energy-dissipative elements and energy-dissipating mechanisms, and the seismic zone. As the work progresses, the project study team and the peer review panel may need to add more configurations or variables. Conversely, not all variables may need to be reviewed in detail. For example, in some cases, it will quickly become apparent that the system is robust and the chosen R_dR_o values easily satisfy the performance criteria. The project study team, as well as the peer review panel, can then focus on paths that lead to critical behavior and optimum results.

Examples of energy-dissipative elements or energy-dissipating mechanisms are:

- flexural yielding of steel or concrete beams,
- axial yielding of tension-only steel diagonal braces,
- shear panel yielding in steel eccentric braced frames,
- yielding of the vertical reinforcement in plastic hinge regions in a concrete shear wall structure,
- yielding of fasteners in connections in wood structures, and
- proprietary yielding devices for a variety of structures.

For timber SFRSs, the energy-dissipative elements will be located in the connections with the mass timber elements designed to remain "elastic." Examples of energy-dissipative elements in connections include various dowel-type fasteners such as:

- regular, spiral or ring nails,
- regular or proprietary self-tapping screws,
- bolts or dowels,
- lag screws, and
- various proprietary inelastic devices that can act as fuses.

Examples of energy-dissipative connections in various timber SFRSs include:

- the connections at both ends of a diagonal brace,
- the tension (uplift) connections at both ends at the base of a flexural mass timber shear wall,
- moment-resisting connections between the beams and the columns in moment-resisting frames, and
- the distributed connection around the edges of a light wood-frame panel shear wall.

2.1 Example: Cantilevered mass timber balloon-type shear walls

The example chosen to illustrate the requirements in this section is a mass timber (MT) balloon-type shear wall cantilevering off its base, with yielding confined to the uplift connections at the base. The suite of archetypes and the energy-dissipating elements are described below:

- The SFRS archetypes are a suite of cantilever walls made with CLT or other MT panels.
- The walls are cantilever (balloon-type) construction.
- The building heights considered are 10-m-, 20-m-, 30-m-, and 36-m- (12-storeys)-high.
- Several wall lengths and wall aspect ratios are considered:
    - a length of 3 m for the 10-m-high wall,
    - a length of 4 m for 20-m- and 30-m-high walls, and

  - a length of 6 m for the 20-m-, 30-m- and 36-m-high walls.

- Gravity loads on the walls are 10% and 90% of the lateral tributary weight that is used to calculate the seismic force.
- Different earthquake hazard regions are considered.
- Different soil conditions are considered (refer to Section 4 of this Technical Guide).
- The energy-dissipative elements are the connections (hold-downs) at both ends of the wall located at the base of the wall. They will be modeled using non-linear hysteretic models that reflect test data and include cyclic degradation when present. The project study team and peer review panel must both agree on the hysteresis model and the shape of the backbone curve that will be used to constrain the hysteresis loops (depending upon the quality of the data determined in accordance with Section 3).
- Connections that connect two MT panels (one on the top of another) that are used to extend the MT panels at a certain height of the building (if present) will be capacity protected in uplift and shear and will incur minimal deformation so that the two panels behave as a single panel unless the design of the SFRS requires otherwise. Horizontal shear connections at the base of the wall will be detailed to accommodate vertical displacement demands while exhibiting minimal deformation under horizontal shear loading.
- In cases where two or more MT panels are needed to form a single continuous wall to increase the length of the wall, the connections connecting these panels along the height of the wall will be capacity protected and will be able to transfer all forces needed with minimal deformations so that the two panels act as a single wall (which is the wall type for this example).
- A different cantilever wall case is one in which two or more side-by-side MT wall panels are used to form a coupled wall with connections between the panels along the height of the wall that are designed to dissipate energy. Such connections would be tested to determine their properties under reverse loads and would be modeled using non-linear hysteretic models that reflect the test data and include cyclic degradation when present. Note that this coupled wall would be a different system with its own R_dR_o values and would require a separate study.
- MT panel elements are modeled to account for flexural compression strains and deformations.
- The non-linear connections are modeled using the properties determined from testing.
- Gap elements are used where required by the anticipated behavior of connections and/or panels.
- Wall thickness is determined by the shear demands and flexural compression demands due to earthquake where the compression length of the wall is taken as 15% and 30% of the wall length in order to investigate the sensitivity of the wall ductility to the compression strength of the wall.
- The wall design is based on the NBC with the assumed R_dR_o values and CSA O86 requirements for the non-energy-dissipative portions and test data for the energy-dissipative elements. The non-energy-dissipative elements are modeled as linear elastic in the analysis. They are reviewed after completion of the analysis to determine what over-strength requirements are required in the design procedure to keep them elastic.

- Higher mode shear effects will be addressed with design rules where required.
- During the study, it may become clear that some variables are important and affect the wall performance while others do not, and it may be possible for the project study team (in conjunction with the peer review panel) to reduce the number of analyses.

Additional information illustrating the archetype is provided in Figure 6 in Section 3.

3. Determining the properties of the energy-dissipative elements

Analytical modeling alone cannot lead to adequate prediction of the seismic response of various structural systems, particularly those that have not been subjected to past earthquakes. A comprehensive experimental investigation program is therefore necessary to establish the material properties, determine the structural component properties, calibrate and validate the component models, and calibrate the numerical analyses for a proposed SFRS. The combination of experimental and analytical data should be sufficient to achieve the main objectives of the numerical analyses which are:

(a) to adequately predict the seismic response of the selected SFRS when subjected to the various seismic demands (including those that are nearing a collapse state), and
(b) to adequately assess the ductility-related force modification factor R_d for the selected system.

Consequently, the main objective of this section is to describe the main considerations that the project study team should take into account during the experimental evaluation of the load-deformation properties of the parts of the SFRS that have significant influence on its seismic response. Additional information that can benefit the reader in the area of wood-based buildings can be found in the current and subsequent editions of FPInnovations' Special report No. SP-55E (2014), "Technical guide for the design and construction of tall wood buildings in Canada."

3.1 General requirements

Components of the SFRS that have the most significant influence on its seismic response need to be identified. Such components are usually the energy-dissipative elements such as steel braces in braced frames, the beams in concrete or steel moment-resisting frames, or the yielding connections in wood-based SFRSs. Evaluation and testing of these energy-dissipative elements that display non-linear inelastic behavior will be necessary in most cases unless a significant amount of testing and research information is already available. Attention should also be paid to elements that are designed to be capacity protected, as those should have adequate strength and stiffness to activate the dissipative elements of the SFRS during the seismic response. The testing program should consist of a coordinated approach of material, structural components and assembly testing. Material test data should serve as reliable reference values for prediction of the strength, stiffness, and deformation properties of the structural components under earthquake loading. Component test data are needed to develop and calibrate the design criteria and the hysteretic load-deformation models of the components that form the essential part of the SFRS. Assembly test data is needed to quantify the interactions between the structural components that cannot be adequately predicted in an analytical way.

Development of the testing program needed for evaluation of a certain SFRS is a complex matter. Such a program will be affected by the type of SFRS under evaluation and the material of which the SFRS is made.

For example, the non-linear material properties of the steel used in the beams and the columns in a steel ductile moment-resisting frame (MRF) are very important, while those properties may be of lesser importance in wood-based MRFs since all non-linearity in the latter case is located in the connections. Determination of the non-linear (crushing) properties of the material however, may be needed in case of wood-based balloon-type (cantilevered) walls.

It is beyond the scope of this section to specify all the testing requirements needed for all possible SFRSs made from all construction materials. Such detailed plans should be made by the project study team in collaboration with the peer review panel. Consequently, the rest of this section includes the main requirements only, with emphasis placed on wood-based SFRSs.

3.2 Testing requirements and instrumentation

Testing requirements and instrumentation are described below.

Test data

If test data is not available, appropriate tests on the energy-dissipative elements (i.e., connections in the case of wood-based SFRSs, or assemblies of several elements (if deemed necessary)), need to be conducted to determine their properties. In most cases, reverse cyclic tests will be sufficient to determine the load-deformation properties. Pseudo-dynamic tests may be warranted in cases of large and more complex structural systems where the applied cyclic displacement history may not cover the range of displacements that the structure would undergo under dynamic action.

Testing laboratory

Testing laboratories used to conduct an experimental investigation program should generally comply with national or international accreditation criteria, such as ISO/IEC 17025:2017, "General requirements for the competence of testing and calibration laboratories." Testing laboratories that are not accredited may be used for the experimental investigation program, provided that the peer review panel verifies the acceptability of the laboratory.

Fabrication of the component or assembly specimens

The energy-dissipative connections tested shall consist of materials representative of those to be used in the actual SFRS of the building. The size of the tested specimens should be as close as possible to the real size that will be used in the SFRS. In cases where this is not possible nor practical due to various limitations (e.g., available space, capabilities of the testing apparatus, or the high cost of testing), use of smaller (reduced scale) specimens is allowed, provided that it can be shown by theory or experiment that testing of reduced-scale specimens will not significantly affect the conclusions with regard to the system behavior and resulting performance.

Construction of the component or assembly specimens should match the construction practices in the field. Special construction techniques or quality control measures should not be employed unless they are part of the design requirements.

Test setup

The test setup should be designed in a way that allows for the tested component or assembly to exhibit movements and deformations during the testing that are close to the ones expected during the earthquake motion when the specimen is part of the SFRS. The boundary conditions of tested specimens should be representative of the constraints that the specimen would experience in the SFRS. In cases in which R_d factors are being investigated for a given SFRS, the boundary conditions should be sufficiently general for the results to be applied to a number of system configurations used in practice. In any case, the boundary conditions should not impose any beneficial effects on seismic behavior that would not exist in the SFRS configurations in practice.

Loads

Loads should be applied to test specimens in a manner that replicates the transfer of load to the component as it would occur in common system configurations. The tests should generally be conducted using displacement control unless the component under investigation requires load-control testing. For components that resist gravity and overturning loads along with the lateral loads, the test configuration should include the gravity loads unless it can be shown that they do not significantly influence the component performance or have beneficial effects on the component performance.

Instrumentation

The instrumentation used should provide accurate and reliable measurement and interpretation of the main parameters, such as strength and deformation properties. All instrumentation should be calibrated before use and records of calibration should be kept.

Test results

If any test results are used that were not part of the study, attention should be paid to all considerations and limitations that were in place when the testing was conducted and their influence on the test results. This is especially important in the case of proprietary products, in which case it will be prudent to ask for the manufacturer's original test results.

3.3 Cyclic tests and data analysis

The cyclic load tests should be conducted using a displacement-based test protocol that is developed in terms of well-defined quantity (e.g., displacement, story drift, or rotation) and should consist of symmetric deformation cycles of step-wise increasing amplitude. Cycles of smaller amplitudes between cycles of increasing amplitudes (trailing cycles) should only be included if they negatively affect the cyclic response of the component. Examples of testing protocols include those found in ASTM E2126-11, "Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings" (see Figure 1).

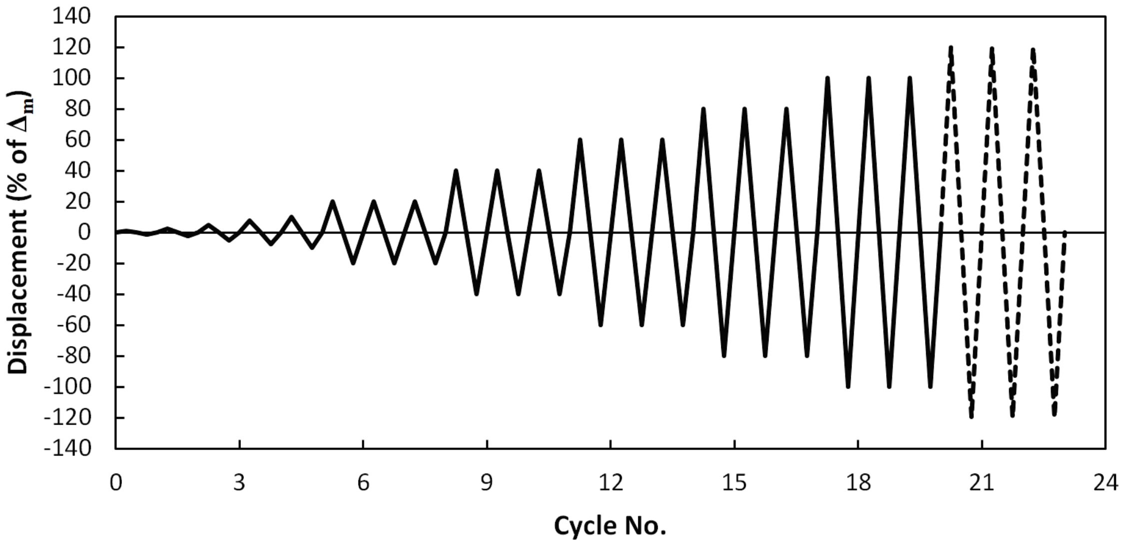


Figure 1: Cyclic testing protocol Method B from ASTM E2126

Most cyclic testing protocols are developed based on a percentage of a reference displacement level that is determined either as an ultimate deformation (Δ or Δ_m) obtained from a monotonic test on the same component, or as an estimate of that value. The value of Δ or Δ_m should not exceed 0.025 times the storey height (2.5% drift limit). The number of cycles should be sufficient to obtain the degradation of strength, stiffness, and energy dissipation capacity of the tested component under repeated cycles of loading. The specimens should be tested to deformations large enough to achieve a "near-collapse" state of the component or assembly (which may be well beyond 2.5% drift), so that not only can the degradation in strength and stiffness be observed, but also the possibility of brittle failure modes. The maximum deformation used in the testing should always be larger than the deformation where there is at least 20% reduction in applied load, after the maximum load has been reached, and can therefore reach the ultimate deformation, Δ_U, in at least one direction of loading. For wood connections and assemblies, the cyclic testing protocol Method B or C from ASTM E2126 are recommended.

When testing connections, a minimum of three (3) replicates should be tested under the monotonic loading to determine the reference displacement level and the number of tests for the cyclic tests. The number of replicates to be tested under cyclic loading should be determined based on the coefficient of variation (COV) of the strength obtained from the monotonic tests by using the approach specified in ASTM D2915-17, "Standard Practice for Sampling and Data-Analysis for Structural Wood and Wood-Based Products." When testing assemblies, a minimum of two (2) and preferably three (3) assemblies should be tested. Three tests would be necessary if the strength varies by more than 10% between the two tests, or if the ultimate deformation Δ_U, varies by more than 20% between the two tests (FEMA P-795 (2011), "Quantification of Building System Performance and Response Parameters — Component Equivalency Methodology.") In cases where a rapid and unpredicted deterioration of the strength occurs that is typical for connections or assemblies experiencing a brittle failure mode, the tested specimens should be redesigned to avoid this type of failure modes and then tested again.

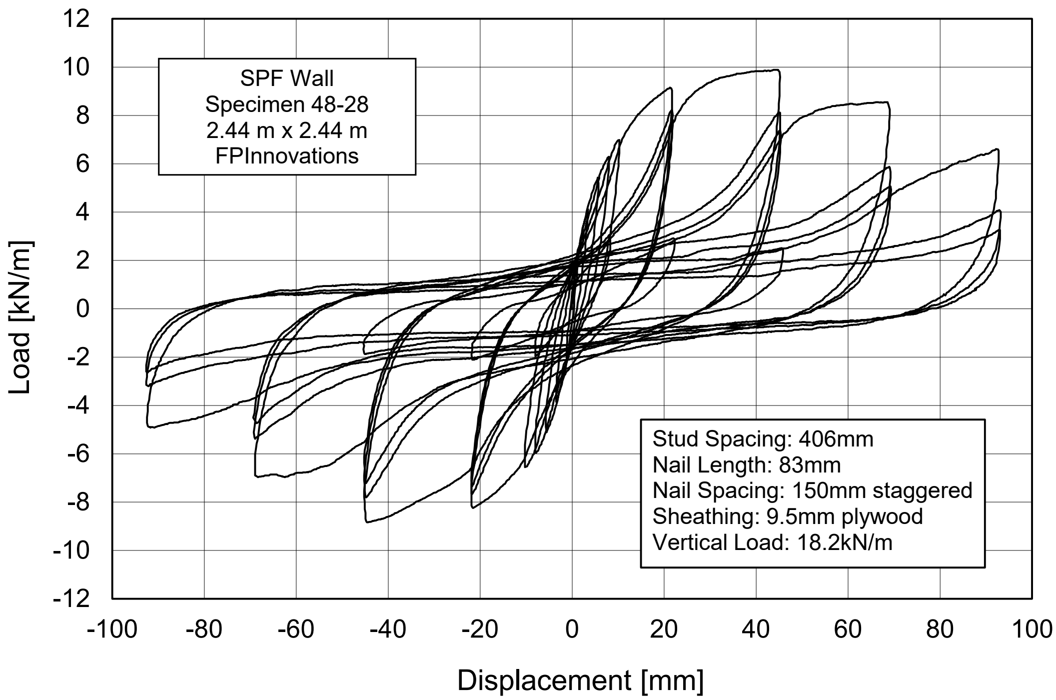
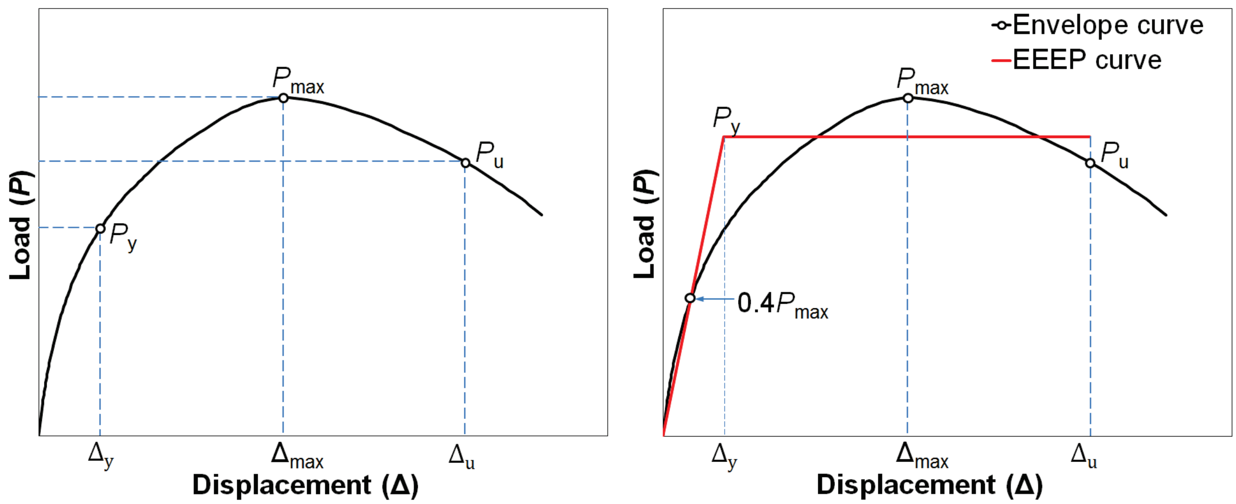


Figure 2: Typical hysteretic response of a wood component obtained from reversed cyclic testing.

The cyclic envelope curves should be constructed based on the maximum load at each level of deformation from the cyclic hysteretic curves. Example of typical hysteresis curves obtained from cyclic tests on a wood-based component are given in Figure 2. The following parameters should be determined using the cyclic envelope curve from the cyclic-load test data (Figure 3):

- Maximum load, P_{max}. This value is called "the ultimate load" in CSA O86.
- Ultimate deformation, Δ_U is defined as deformation at $0.8P_{max}$, after P_{max} was reached.
- P_U is defined as a load level at the ultimate deformation Δ_U.
- Initial stiffness, K_1, for wood components is calculated differently according to different standards.
- Yield deformation, Δ_Y, for wood components is calculated differently according to different standards $\Delta_Y = P_{max}/ K_1$.
- Ductility capacity, μ, is calculated as $\mu = \Delta_U / \Delta_Y$.

While the initial stiffness for steel and concrete structures may be clearly defined, for wood-based components the initial stiffness can be determined in different ways according to different testing standards. According to ASTM E2126, the stiffness can be determined as the slope of the line connecting the origin to the $0.4P_{max}$ point on the envelope curve, while most European and ISO standards calculate the stiffness as the slope between the points of 0.1 P_{max} and $0.4P_{max}$ on the envelope curve. ASTM E2126 defines the yield displacement using the ideal equivalent energy elastic-plastic (EEEP) curve (Figure 3) circumscribing an area equal to the area enclosed by the envelope curve between the origin, the ultimate displacement, and the displacement axis. On the other hand, ISO and European standards define the yielding point Δ_Y as an intersection between the initial stiffness (K_1) line and the line that has 1/6 of the initial stiffness and are tangent to the envelope curve.



a) b)

Figure 3. Definitions of various ductility parameters (a) Typical load-displacement curve; (b) Equivalent energy elastic-plastic (EEEP) curve

The method for computing the average performance parameters (ultimate deformation, initial stiffness, yield load, and ductility) that characterize both positive and negative envelopes can make a difference when the specimen shows an asymmetrical response. The responses from most wood-based component tests are asymmetrical to some degree as the damage created with an initial positive excursion tends to weaken the response from the subsequent negative excursion within the same cycle. Determining the average response parameters for a specimen with slightly dissimilar positive and negative envelope curves by analyzing each envelope, the positive and the negative individually, and then averaging the results obtained from each envelope can result in non-conservative estimates of the performance. In these cases, it is suggested that the average envelope curve is determined first (Figure 4) and then that the parameters be determined based on that average envelope curve.

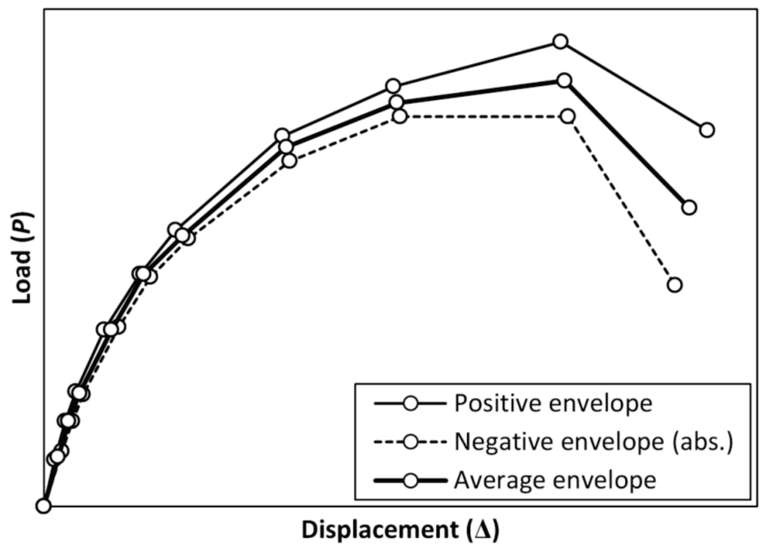


Figure 4. Example of an average envelope curve obtained from a Test Method C of ASTM E2126

For components with reasonably symmetric behavior, values of ultimate deformation, initial stiffness, yielding load, and the ductility, can be calculated as the average of their respective values determined from the positive and the negative portions of the envelope curve. The average of the yielding displacement can then be back calculated using the average yield load and the average initial stiffness. For components with significant asymmetric behavior, positive and negative values of these parameters should be calculated and evaluated separately for each loading direction. For more information, please refer to ASTM E2126.

3.4 Example: Cantilevered (balloon-type) mass timber (MT) wall

MT buildings are typically constructed using either the platform-type or balloon-type method, as shown in Figure 5. In platform-type CLT construction, the floor platform at each storey is used as a base for erection of the CLT walls of the next storey. In balloon-type construction, the walls are continuous for the entire height of the building and the floor panels are attached to the walls at each storey. This solution avoids the compression perpendicular-to-grain stress from accumulating in the CLT floor panels, and also takes advantage of CLT panels being manufactured with a length up to 20 m. Several buildings of this type have already been built in Canada. Examples include the 13-storey Origine building in Quebec City, QC, the eight-storey buildings of the Arbora complex in Montreal, QC, and the 30-m-high wood Innovation and Design Centre in Prince George, BC.

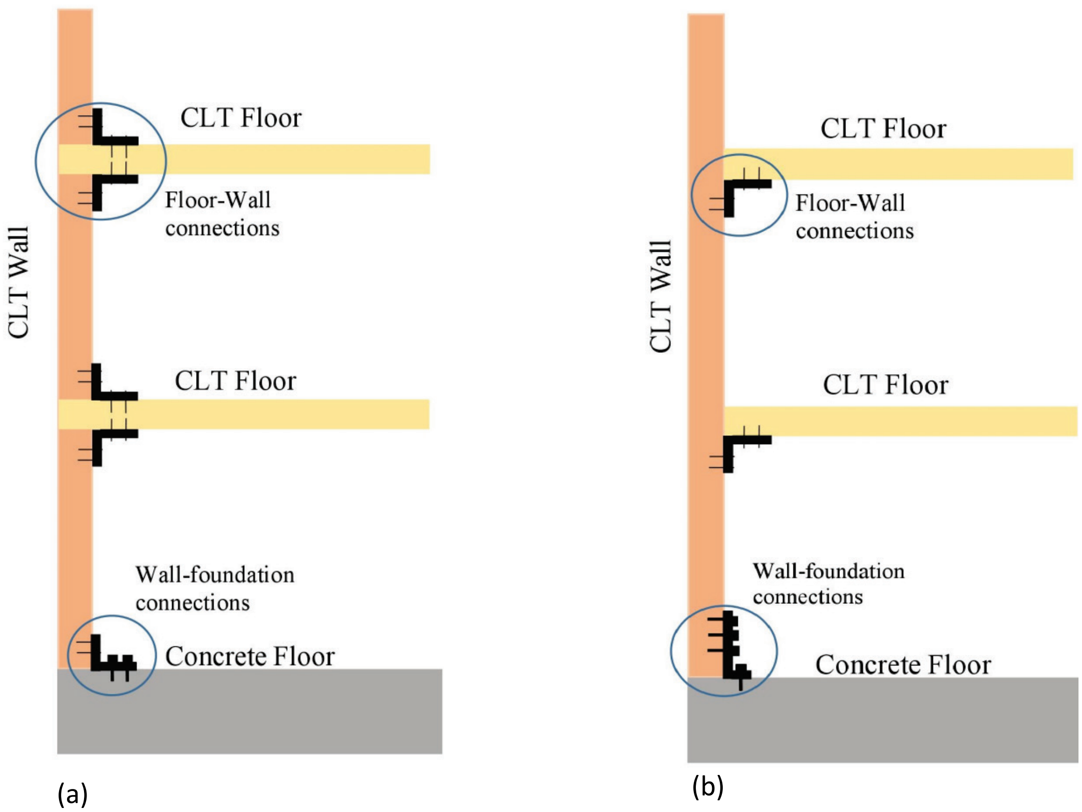


Figure 5. Simplified schematic of (a) platform-type and (b) balloon-type CLT construction

Examples of balloon-type MT wall configurations are given in Figure 6 (a) and (b). The seismic response of this type of SFRS is influenced by the hold-down connections at the bottom of the wall, as well as the connections in the vertical joints (Figure 6b) if the wall consists of more than one panel and the connections are designed to dissipate energy. In the latter case, the walls are usually referred to as "coupled shear walls."

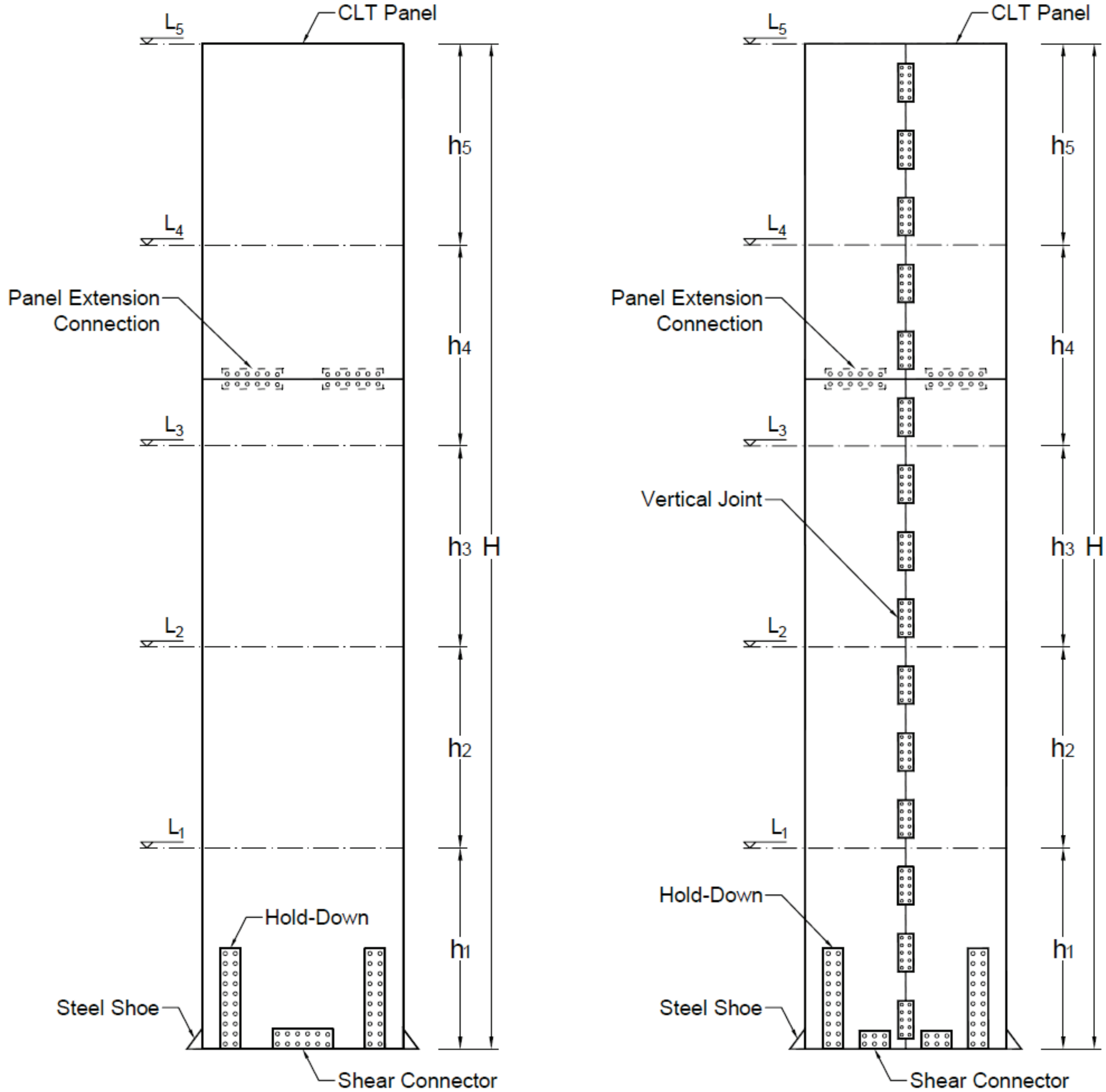


Figure 6. Simplified schematics of (a) single panel CLT balloon-type wall and (b) coupled balloon-type CLT wall

The horizontal shear connections at the base of the wall should be detailed to accommodate the vertical displacement demands while exhibiting minimal deformation (sliding) under horizontal shear loading. The hold-down connections are part of the energy dissipative part of the system and should be tested as specified in this section of the document. In this case, the cyclic loads during the testing should only be applied in one direction (tension) as the compression is resisted by the MT panel. The compression properties of the MT material should also be tested to determine the potential deformation during the seismic response. Depending on the type of the shear and hold-down connections used, these types of walls may sometimes experience out-of-plane movement (walking) of the panel during the seismic response. To eliminate such movement, steel toes or shoes should be used at both ends of the panel as shown in Figure 6. Connections in the vertical joints should be tested under reversed cyclic loads as they exhibit such motions in reality. The obtained hysteresis loops from tests on both connections can be used in developing the numerical models of the walls. Connections that are used to extend the MT panels at certain heights of the building should be capacity protected in uplift and shear and should incur minimal deformations. If strength and stiffness properties of these connections are not known, they have to be tested to determine their properties.

In cases where two or more MT panels are needed to form a single wall (to increase the length of the wall), the connections connecting these panels along the height of the wall should be capacity protected and should be able to transfer all the forces needed with minimal deformations so that the two panels act as a single wall.

4. Choosing and scaling earthquake ground motions

Comments in FEMA P-695 indicate that in general, design procedures developed for regions of high seismicity work in regions of low seismicity. However, the requirements needed for acceptable performance in high seismicity zones may be unnecessarily restrictive in low seismic zones. Therefore, several seismic zones should be chosen to develop appropriate requirements that are efficient for different parts of the country and not overly conservative.

It is important that the choice of ground motions for the non-linear analyses is made by and peer reviewed by those with extensive experience in choosing records and who are familiar with the seismicity of the particular region. It is desirable that this work is performed by the project study team and the peer review panel. However, this is a very specialized topic and may require an outside consultant as well as a separate peer review.

When evaluating and determining R_dR_o values for a suite of archetypes, FEMA P-695 suggests using ground motions developed for Site Class D. This is proposed for the procedure described in this document, as well.

4.1 Considered locations for seismicity

The locations for seismicity that can be considered for the non-linear analyses are:

- **Tofino on Site Class D**. This is a location of very high seismicity with a small population and few tall buildings. The main seismic hazard comes from subduction earthquakes with long durations. The requirements developed for this location may be so rigorous that the best way to treat them may be as special cases with extra requirements when compared to Victoria or Vancouver. This was done for some cases in CSA S16-14, “Design of steel structures,” where design forces higher than the NBC values are mandated for some situations.
- **Victoria on Site Class D**. This is also a high seismic zone governed by subduction earthquakes and may need to be treated as a special case as discussed above.
- **Vancouver on Site Class D**. The Vancouver region seismic hazard is governed by subduction earthquakes in the structures’ long period range. Crustal and sub-crustal earthquakes govern the hazard in the short period range. However, the intent here is to have a standard code solution that fits in the NBC R_dR_o tables that works for both period ranges.

- **Montreal on Site Class D**.
- **Toronto on Site Class D**.

During the study, the participants may feel that it would be worthwhile to consider additional regions or site classes for study.

Since structures designed for high seismic regions typically perform well in lower seismic regions, it may become clear early in the process when considering lower seismic regions such as Toronto, Montreal, or the Maritimes that the design requirements will be governed by those needed for Vancouver and Victoria. If this is the case, the project study team and peer review panel may conclude and document that a full set of analyses is not needed in these low seismic regions.

However, the project study team may also feel that it may be desirable to relax the requirements for lower seismic regions and have a second set of less stringent requirements for these regions. If this is the case, the study must be completed in full for the lower seismic regions using the less rigorous requirements.

4.2 Choice of earthquake ground motions and scaling

Earthquake ground motions chosen for the analyses shall satisfy the requirements in NBC Commentary J and its Appendix A in Structural Commentaries, User's Guide: NBC 2015 (Canadian Commission on Building and Fire Codes, Structural Commentaries, User's Guide, 2015), with the exception described below.

Subduction earthquakes typically have long durations. When compared to short duration earthquakes, long duration earthquakes with many cycles can have detrimental effects on non-linear degrading elements greater than expected based on spectral intensity alone. This can lead to un-conservative results in the short period range for subduction ground motions at the "cross-over" point for locations like Vancouver where short duration earthquakes that dominate the short period range are scaled to the uniform hazard spectrum (UHS) in the short period range and the long duration subduction earthquakes that dominate in the long period range are scaled to the UHS in the long period range. A second concern arises with scaling long duration subduction motions to the long period range only of the UHS and that is the loss of control of the subduction motions in the short period range. This can lead to overly conservative or un-conservative results for subduction earthquakes in the short period range.

Methods to deal with this for a single building exist and typically involve developing a spectrum tied to the UHS at the building period along with associated probabilities for the spectrum. However, when a suite of buildings with a wide variety of periods is to be investigated, it is desirable to have a simple, appropriately robust target spectrum that covers the period range from 0.0 to 10.0 seconds.

To address these issues for Victoria and Vancouver, three target spectra that span the period range are developed. They are based on the 2% in 50-year UHS for each source zone of crustal, sub-crustal and subduction source zone, each multiplied by 1.3 but with no value taken greater than the NBC UHS. Note that this scaling factor is based on NBC 2015 seismic data and it may not be applicable to NBC 2020 seismic data. Acceptance of the analysis results is based on the governing source zone.

An example is not provided in this section as the section itself serves as an example with detailed recommendations for regions across Canada that are to be considered. However, additional information is given in Appendix A for spectra for Victoria and Vancouver, and in Structural Commentaries, User's

Guide: NBC 2015 (Canadian Commission on Building and Fire Codes, Structural Commentaries, User's Guide, 2015) on selecting ground motions and using them in non-linear analysis.

5. Developing target R_d and R_o factors

5.1 Over strength-related force modification factor R_o

The R_o factor can be developed using the results from the test data, engineering judgment and the procedure in the paper by Mitchell et al. in the *Canadian Journal of Civil Engineering*, Volume 30, Number 2, April 2003. This paper contains much useful information and describes everything needed to understand R_o for the various structural materials and lateral load resisting systems.

However, a very brief summary follows:

The R_o factor is an estimate of the dependable (or minimum) over-strength of a lateral system and takes the form of:

$$R_o = R_{size} \cdot R_{\phi} \cdot R_{yield} \cdot R_{sh} \cdot R_{mech} \qquad \text{(Eq.1)}$$

Where

R_{size} is a factor representing how "tight" the design can be, or whether it may be governed by minimum requirements or by choices between arbitrary sizes.

R_{ϕ} gives a nominal strength and is $1/\phi$ where ϕ is the material strength reduction factor.

R_{yield} estimates the actual or expected yield strength of a material above the nominal value.

R_{sh} estimates the effect of any strain hardening.

R_{mech} estimates the over-strength from mobilizing the full capacity of the system at a collapse mechanism.

The total R_d R_o is the result to be determined from the non-linear analysis. R_d is then determined from the total and knowing R_o.

5.1.1 Cantilevered (balloon-type) CLT shear wall

If we assume that the yielding connection at the base of the cantilever (balloon-type) flexure wall is to be a steel plate connection nailed to the CLT wall, with non-liner inelastic behavior provided by the nails, then the proposed values of the various overstrength factors based on the concepts in Mitchell et al. (2003):

R_{size} = 1.1. (it is 1.15 in Mitchell et al. (2003), for wood frame shear walls)

R_{ϕ} = 1/0.8=1.25 (it is 1.43 in Mitchell et al. (2003))

R_{yield} = 1.0

R_{sh} = 1.05

R_{mech} = 1.0

$R_o = 1.1 \cdot 1.25 \cdot 1.05 \cdot 1.0 = 1.51$ and can be taken as 1.5.

Note that this is different than the value of R_o = 1.7 given for wood-frame shear walls in Mitchell et al. (2003) and in NBC.

R_{size} has been reduced in comparison to the wood-frame shear walls since this connection will be better "tuned" to the demand and not governed by minimum numbers or steps in choosing the nail sizes.

R_ϕ has been reduced compared to the value for wood-frame shear walls as ϕ for nailed connections has a value of 0.8 in CSA O86, which is different than the value of 0.7 that was previously included for wood-frame shear walls.

The R_o factor can be calculated in a similar way for connections with other fasteners using the design information provided in CSA O86 and the available test data.

If a connection such as a buckling restrained brace (BRB) is being considered instead of a nailed plate, the R_o would be 1.2 (as shown in CSA S16) to reflect the tight control of a manufactured proprietary product. The non-linear hysteresis loop for a BRB is readily available for use in the non-linear inelastic analysis.

All of the above would be finalized with discussions between the project team and the peer review panel.

5.2 Ductility-related force modification factor R_d

Method 1

A simple method to determine Rd factor is to estimate a value for the R_d factor based on the test data of the critical energy-dissipative elements in the structural system and their location, and then test it using the procedure outlined in section 6 for a location such as Vancouver and a representative configuration. Note that this will require judgment and understanding of the structure as the R_d value is a system force modification (reduction) factor (often thought of as "system ductility factor") and for many systems, the energy-dissipative element ductility demand is several times larger than the system R_d values.

Method 2

For systems dominated by the first mode behavior, the results from the non-linear static analysis (pushover) may be utilized to provide an estimate of the R_d value as a starting point. It may require some iteration before an acceptable value of R_d is determined. This value can then be tested out using the full procedure outlined in Section 6 and iterated if required.

Method 3

A more refined initial estimate of an R_d factor for a timber SFRS may be developed using the following steps:

1. Assume an R_d value and design the 2D prototype of the structural system for strength using an NBC-equivalent static procedure or linear dynamic procedure.
2. Determine the stiffness of the elements in the prototype and create a stiffness model of the prototype by modeling all the elements including the connections.

3. If some connections (such as the base tie-down connection of a shear wall) have a different stiffness in tension than in compression, use Raleigh's method to determine a first period. Otherwise, simply determine the first period using the analysis program.
4. Calculate the period using the NBC empirical equations for equivalent static procedure and adjust the strength and stiffness of the prototype accordingly to satisfy any NBC requirements for limits on period, drift, and design base shear.
5. Perform a response spectrum analysis (RSA) for R_dR_o=1.0 and determine the elastic response top displacement.
6. Knowing the R_d value (the system "ductility" factor) and R_o value, estimate the non-linear demand on the energy-dissipative element (the element "ductility" factor). Adjust the system ductility factor R_d estimate and iterate if required until the estimated energy-dissipative element ductility demand is acceptable (i.e., less than 50% of the non-linear displacement between the "yield point" and "capping point" displacement, based on the elements non-linear capacities).
7. Use this value of R_d as the proposed value to test using the full procedure outlined in Section 6.

Similar steps can be followed for other SFRSs, except that the modeling of energy-dissipative elements would be different.

6. Non-linear analysis approach

6.1 Suggested approach

In simple terms, a summary of the suggested approach is:

1. Design a 2D archetype structure from the suite using the proposed R_d and R_o values to the NBC using the equivalent static force procedure (ESFP) or the linear dynamic analysis procedure (LDAP) incorporating all of the requirements and constraints except height limits as the height limits are part of this study.
2. Perform a non-linear inelastic analysis on this prototype as outlined in NBC Commentary J and its Appendix A with ground motions scaled to 100% of the UHS and as outlined in Section 4. The acceptance criteria are as Commentary J and its Appendix A.
3. Perform a second non-linear inelastic analysis with the ground motions' intensity doubled (200% of the UHS). If more than 50% of the ground motions result in an unacceptable response (basically collapse) as described in NBC Commentary J and its Appendix A, the system is considered to have failed. This is similar to the collapse margin ratio approach described in FEMA P-695. Unacceptable response is further discussed in Section 6.2.
4. Repeat the same procedure for the entire suite of different structural archetypes and parameters.

A flowchart is presented in Figure 7 for the suggested approach.

The above procedure is strongly influenced by FEMA P-695. Note that this is not a full FEMA P-695 analysis as that analysis requires using uncertainties in parameters, a full incremental non-linear analysis, calculating a collapse margin ratio and not setting it as in the suggested procedure. A full FEMA P-695 approach requires an enormous amount of analysis. However, the proposed methodology described above, while much simpler, should give a robust assessment of the parameters. It is also more rigorous than methods commonly used in the past to develop R_dR_o values.

In evaluating a seismic system, the intent of the procedure is to work through all the archetypes and variables in Section 2 and assess what variables are important and need to be addressed in the NBC and corresponding material design standard. Note that it is difficult to plan in a fixed path in advance through this process as the results along the path may dictate changes to the approach.

For instance:

- The R_dR_o value chosen may work for all the archetypes in all seismic zones and that may end the process.
- As discussed in section 4.1, it may become clear that the R_dR_o value is considered too conservative for some variables or zones and that a larger R_dR_o value can be shown to be appropriate for these cases, or it may be possible to retain the R_dR_o value and reduce the number of analyses necessary to complete the study.
- There may be unacceptable results for some seismic zones or variables, but this may only require some constraints in the NBC or corresponding material design standard.
- There may be enough unacceptable results encompassing several seismic zones and/or variables and configurations that would require a complete revision of the proposed R_dR_o value.

6.2 Analysis results

6.2.1 Unacceptable response

Examples of unacceptable responses are dynamic instability, non-convergent analysis, and force or deformation demand on an element that exceeds the force or deformation capacity of that element.

For all responses of motions that are scaled to 100% of UHS, the inter-storey drift limits per the NBC shall also be respected. For responses of ground motions that are scaled to 200% of UHS, the absolute value of the maximum inter-storey drift from the suite of analyses shall not exceed 4.5%. Non-linear time history analysis beyond this drift limit is considered unreliable using current available analysis tools (Pacific Earthquake Engineering Research Centre Guidelines for Performance-Based Seismic Design of Tall Buildings (“PEER Guidelines”).

For motions that are scaled to 100% of UHS, in accordance with NBC Commentary J and its Appendix A, unacceptable responses shall not be allowed, except under the following conditions where one outlier response is permitted:

- the suite includes a minimum of 11 ground motions,
- additional evaluations indicate that the predicted response is not indicative of unacceptable structural performance, and
- spectral matching techniques are not used.

For motions that are scaled to 200% UHS, if more than 50% of the ground motions in each suite result in an unacceptable response (basically collapse), the system is considered to have failed.

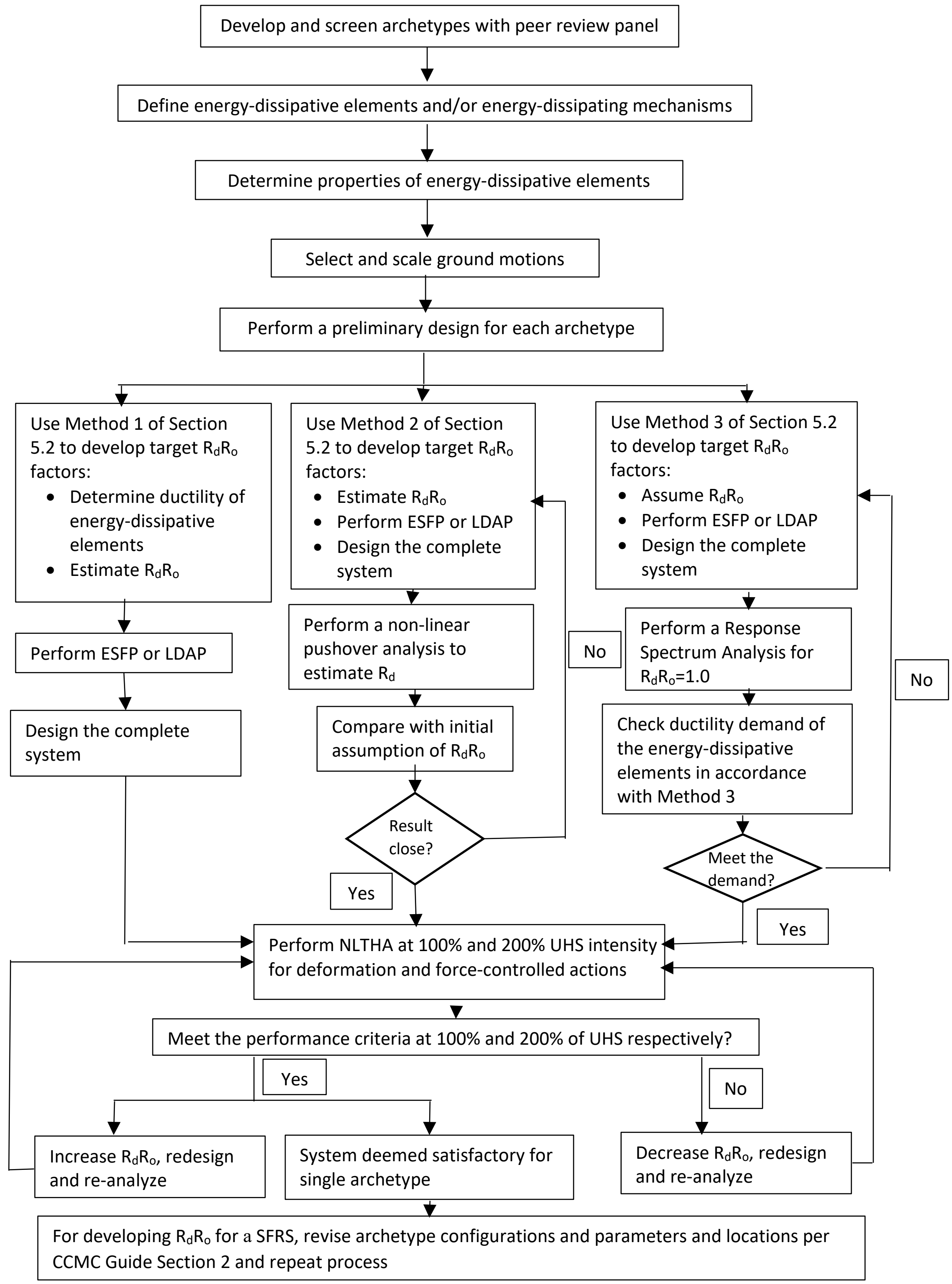


Figure 7. Flowchart of approach for developing R_d factors

7. Summary of study by the project study team

At the end of the study, the project study team provides a brief, succinct summary report containing:

- a description of archetypes used,
- a sample of the energy-dissipative elements used along with test data where applicable, hysteresis plots, backbone curves, and modeled curve results,
- the name of the analysis program used,
- a table of the ground motions and scale factors, method of scaling used and plots of the scaled ground motion spectra with the average of the suite and the UHS,
- a representative sample of results,
- a brief discussion of results and conclusions,
- recommendations for R_d and R_o for the NBC with any limits determined in the study,
- recommendations for any additional and/or revised clauses in the NBC and appropriate CSA structural design standard, and
- a letter from the peer review panel stating their acceptance of the study.

This summary report is in addition to the documented work kept on file.

8. References

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Appendix A – Target spectra for Victoria and Vancouver

This appendix develops target spectra for Victoria and Vancouver, BC for three different source zones:

- inslab
- crustal
- subduction

For Site Class D:

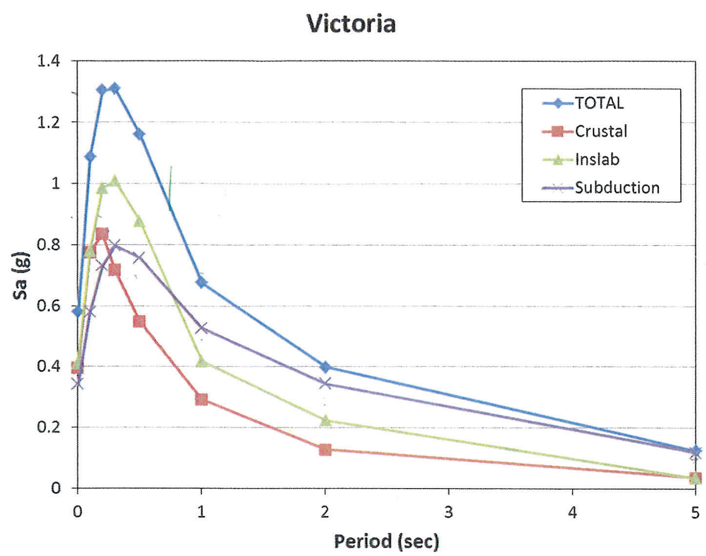
Steps:

(1) Take the Site Class C spectral values from Figure A1(a) in this appendix for Victoria for the 2% in 50-year response spectra for the NBC 2015 UHS and the three 2% in 50-year UHS response spectra for the three source zones.
(2) Develop site factors to take the Site Class C spectral values to Site Class D spectral values based on each PGA (Sa(0.0)) for each of the NBC UHS and the three source zones.
(3) Calculate Site Class D spectral values for the NBC UHS and the three separate source zones using the Site Class Factors in (2) and the Spectral Values in (1).
(4) Develop three target spectra, one for each source zone of inslab, crustal, and subduction earthquakes.

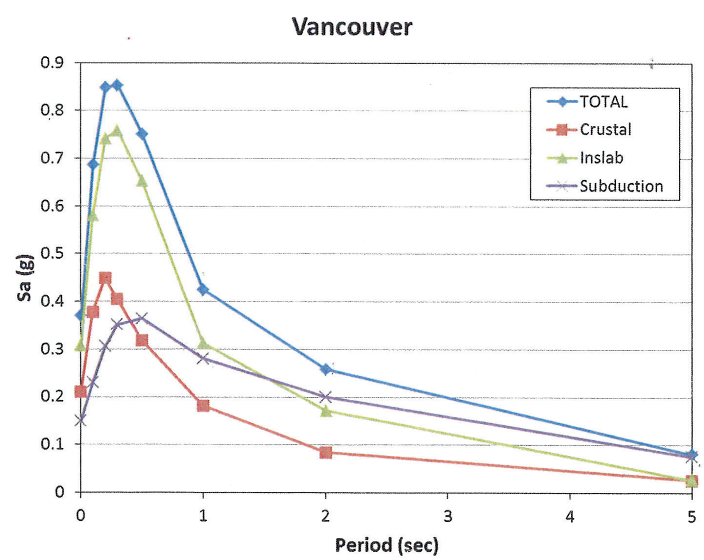
Each target spectra = source spectra × 1.3 but not greater than the Site Class D NBC UHS spectra. These are tabulated on page 29 and illustrated on page 30 (see Figure A2).

Use each target spectra to assess the suite of archetypes in Victoria with the worst case governing.

Repeat the process for Vancouver starting with Figure A1(b). The tables and figures are on pages 31 and 32 (see Figure A3).



Victoria 2% in 50 year UHS by source, Site Class C = 450m/s



Vancouver 2% in 50 year UHS by source, Site Class C = 450m/s

Figures A1(a) 2% in 50 year UHS by source for Site Class C in Victoria; and A1(b) 2% in 50 year UHS by source for Site Class C in Vancouver

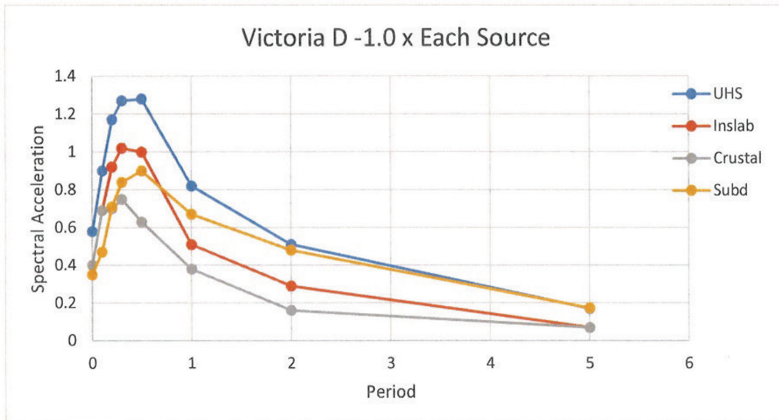
Victoria – Site C spectra									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.58	1.08	1.3	1.3	1.16	0.676	0.4	0.125	0.044
Inslab	0.4	0.78	0.98	1.0	0.88	0.41	0.22	0.05	0.0
Crustal	0.4	0.78	0.84	0.74	0.55	0.3	0.125	0.05	0.0
Subdtn	0.36	0.58	0.73	0.8	0.77	0.52	0.36	0.125	0.044

Factors – Site Classes C to D – general – from NBC 2015 – based on PGA in left column									
Period / **PGA**	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
0.1	1.0	1.21	1.24	1.34	1.47	1.55	1.57	1.58	1.49
0.2	1.0	1.04	1.09	1.17	1.3	1.39	1.44	1.48	1.41
0.3	1.0	0.94	1.0	1.08	1.2	1.31	1.36	1.41	1.32
0.4	1.0	0.89	0.94	1.02	1.14	1.25	1.31	1.37	1.34
0.5	1.0	0.84	0.9	0.96	1.1	1.21	1.27	1.34	1.31

Factors – Victoria – C to D – for NBC UHS and source UHS – (PGA in left column 0.0 period)									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.58	0.84	0.9	0.98	1.1	1.21	1.27	1.34	1.31
Inslab	0.4	0.89	0.94	1.02	1.14	1.25	1.31	1.37	1.34
Crustal	0.4	0.89	0.94	1.02	1.14	1.25	1.31	1.37	1.34
Subdtn	0.35	0.81	0.97	1.05	1.17	1.28	1.34	1.39	1.36

Victoria – Site D spectra for NBC UHS and each source									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.58	0.9	1.17	1.27	1.28	0.82	0.51	0.17	0.058
Inslab	0.4	0.69	0.92	1.02	1.0	0,51	0.29	0.07	0.0
Crustal	0.4	0.69	0.79	0.75	0.63	0.38	0.16	0.07	0.0
Subdtn	0.35	0.47	0.71	0.84	0.9	0.67	0.48	0.173	0.058

Victoria – Target spectra – Site Class D source values × 1.3 but less than NBC UHS									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.58	0.9	1.17	1.27	1.28	0.82	0.51	0.17	0.058
Inslab	0.4	0.9	1.17	1.27	1.28	0.66	0.38	0.09	0.0
Crustal	0.4	0.9	1.03	0.98	0.82	0.49	0.21	0.09	0.0
Subdtn	0.35	0.61	0.923	1.09	1.17	0.82	0.51	0.17	0.056



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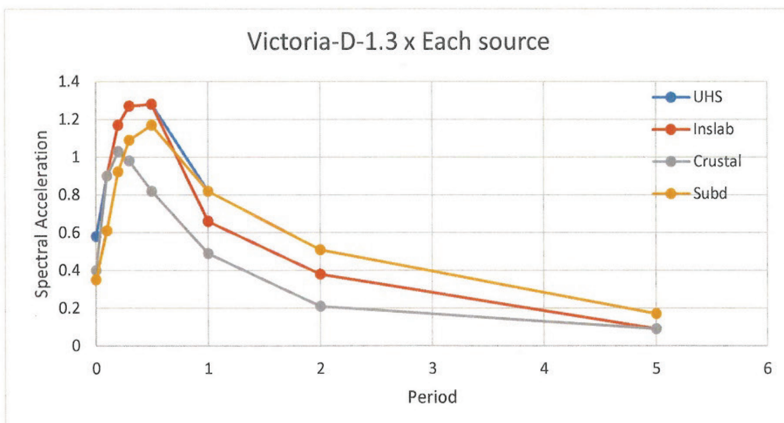


Figure A2(a) 2% in 50 year UHS by source for Site Class D in Victoria; and A2(b) Target spectra for Site Class D in Victoria

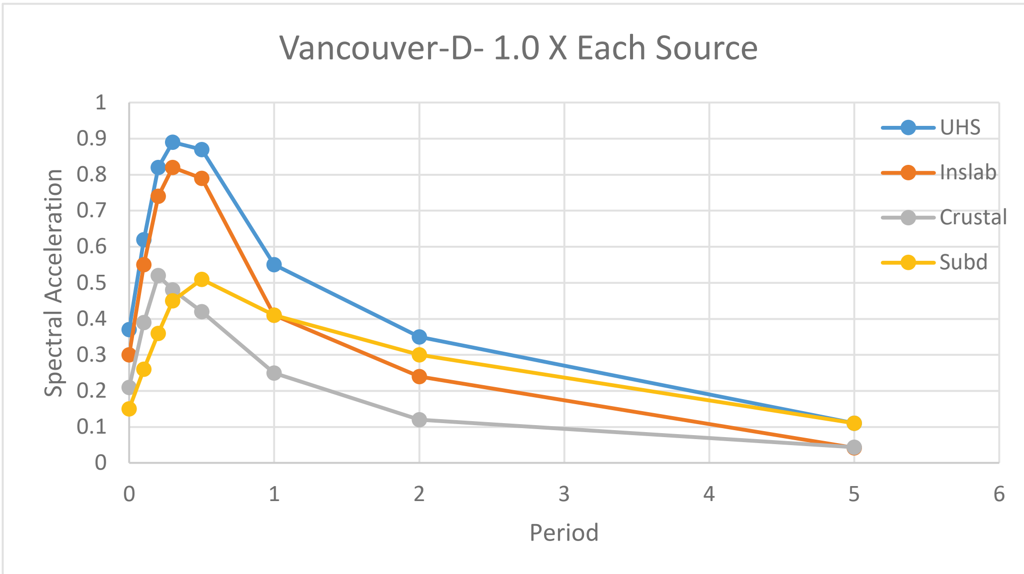
Vancouver City Hall – Site C spectra									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.37	0.69	0.85	0.86	0.75	0.43	0.26	0.08	0.029
Inslab	0.30	0.58	0.74	0.76	0.66	0.31	0.18	0.03	0.0
Crustal	0.21	0.38	0.46	0.41	0.32	0.18	0.08	0.03	0.0
Subdtn	0.15	0.23	0.31	0.36	0.37	0.28	0.20	0.08	0.029

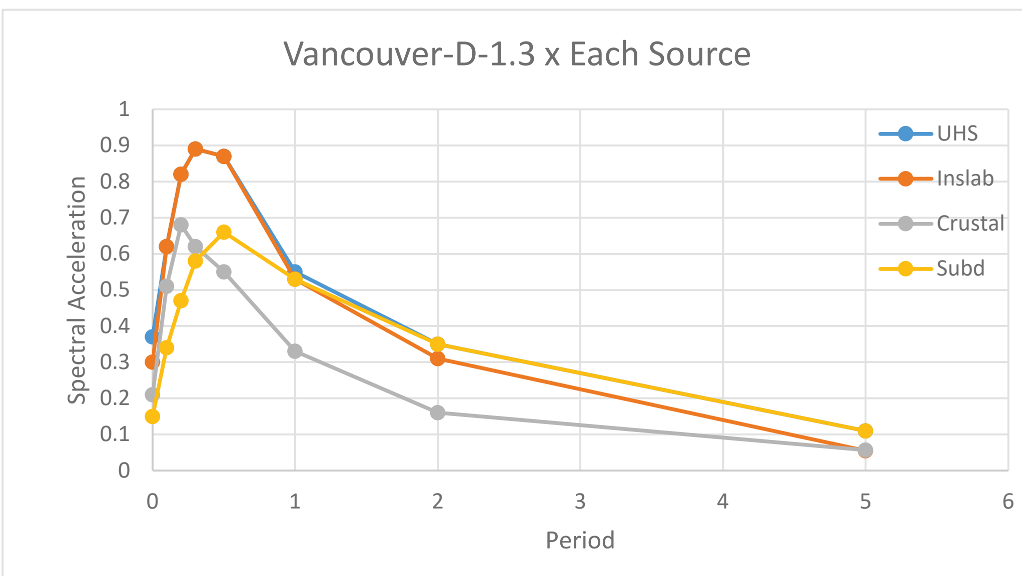
Factors – Site Classes C to D – general – from NBC 2015 – based on PGA in left column									
Period / **PGA**	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
0.1	1.0	1.21	1.24	1.34	1.47	1.55	1.57	1.58	1.49
0.2	1.0	1.04	1.09	1.17	1.3	1.39	1.44	1.48	1.41
0.3	1.0	0.94	1.0	1.08	1.2	1.31	1.36	1.41	1.32
0.4	1.0	0.89	0.94	1.02	1.14	1.25	1.31	1.37	1.34
0.5	1.0	0.84	0.9	0.96	1.1	1.21	1.27	1.34	1.31

Factors - Vancouver – C to D – for NBC UHS and source UHS – (PGA in left column 0.0 period)									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.37	0.9	0.96	1.04	1.16	1.27	1.33	1.38	1.35
Inslab	0.3	0.94	1.0	1.08	1.2	1.31	1.36	1.41	1.34
Crustal	0.2	1.04	1.09	1.17	1.3	1.39	1.44	1.48	1.41
Subdtn	0.15	1.13	1.17	1.26	1.39	1.47	1.5	1.53	1.45

Vancouver City Hall – Site D spectra for NBC UHS and for each source									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.37	0.62	0.82	0.89	0.87	0.55	0.35	0.11	0.039
Inslab	0.3	0.55	0.74	0.82	0.79	0.41	0.24	0.042	0.0
Crustal	0.2	0.39	0.52	0.48	0.42	0.25	0.12	0.44	0.0
Subdtn	0.15	0.26	0.36	0.45	0.51	0.41	0.30.	0.11	0.039

Vancouver City Hall –Target spectra – Site Class D source values × 1.3 but less than NBC UHS									
Period	0.0	0.1	0.2	0.3	0.5	1.0	2.0	5.0	10.0
NBC	0.37	0.62	0.82	0.89	0.87	0.55	0.35	0.11	0.039
Inslab	0.3	0.62	0.82	0.89	0.87	0.53	0.31	0.055	0.0
Crustal	0.2	0.51	0.68	0.62	0.55	0.33	0.16	0.057	0.0
Subdtn	0.15	0.34	0.47	0.58	0.66	0.53	0.35	0.11	0.039





Figures A3(a) 2% in 50 year UHS by source for Site Class D in Vancouver; and A3(b) Target spectra for Site Class D in Vancouver