BRIDGING GUIDELINES FOR THE PERFORMANCE-BASED SEISMIC RETROFIT OF BRITISH COLUMBIA LOW-RISE SCHOOL BUILDINGS

COMMENTARY

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BRITISH COLUMBIA
Ministry of Education

PART A EXPLANATORY NOTES ON SECOND EDITION BRIDGING GUIDELINES

A1.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR LOW-RISE SCHOOL BUILDINGS – GENERAL REQUIREMENTS

A1.1 Introduction

1) These guidelines are for both risk assessment and retrofit design, the two primary steps in the seismic upgrading of low-rise British Columbia school buildings.

2) These guidelines are for low-rise buildings only. Low-rise buildings are defined as buildings with three storeys or less above the basement or crawl space.

3) These guidelines are divided into seven related parts:

   (a) General requirements that are applicable to all buildings (Section 1)

   (b) Toolbox method for combining the strengths of different lateral load resisting systems (LDRSs - Section 2) in a building that has concrete diaphragms and moderate plan eccentricities

   (c) Guidelines for calculating the factored lateral resistance contribution for the different LDRSs (Sections 3-7)

   (d) Guidelines for LDRSs that rock before they reach their maximum lateral resistance (Section 8)

   (e) Guidelines for heavy non load-bearing (partition) walls including specific guidelines for vertical bracing elements that provide out-of-plane support

   (f) Guidelines for diaphragms and connections (Sections 10-11)

   (g) Commentary on each sentence of the guidelines and technical background information that documents the development of this performance-based methodology

4) These guidelines have been drafted to complement the 2005 edition of the National Building Code of Canada (NBCC05). NBCC05 states performance-based design objectives but lacks the technical details required by engineers to implement the performance-based objectives.

The purpose of these guidelines is to provide the engineer with the necessary technical tools to implement a comprehensive performance-based seismic upgrading of low-rise school buildings in British Columbia.

The performance-based methodology adopted in the development of the Bridging Guidelines offers a rational method for implementing a life-safe, cost-effective strategy for the risk assessment and seismic upgrade of low-rise buildings.

6) All school buildings in British Columbia are to be seismically upgraded using the provisions of the Bridging Guidelines. If a school building lies outside the scope of these guidelines, the building is to be assessed and retrofitted exclusively on the provisions of NBCC05. No mixing of Bridging Guidelines and NBCC05 provisions is permitted.

7) The primary life safety objective of the Bridging Guidelines is collapse prevention. The Bridging Guidelines methodology quantifies collapse prevention by limiting inelastic drift for the design ground motion that has a probability of exceedance of 2% in 50 years (50 percentile values).

   The probability of collapse is significant for inelastic drifts that exceed the specified limits. Collapse is not an automatic consequence of excessive building drift. The greater the drift exceeds the drift limit, the greater the probability of collapse. The risk of building collapse and the resulting protection of life safety are considered acceptable for drift levels that do not exceed the drift limits.

8) All building elements must be capable of accommodating the maximum building drift without loss of vertical support (loss of vertical support is a precursor to collapse). These vertical building elements include both members of the LDRSs (e.g.: shearwalls) and individual load-bearing elements that are not part of any LDRS. Heavy non load-bearing (partition) walls must also be stable during shaking and deformation of the building.

9) As noted in (8) above, heavy partition walls are included in the scope of these guidelines to provide a cost-effective, comprehensive life safety upgrade of seismically deficient buildings.

A1.2 Seismicity

1) The probability of exceedance of 2% in 50 years is an integral feature of the Uniform Hazard Spectra (UHS) approach. Heidebrecht (1999) suggests that the 2% in 50 years probability level represents an approximate structural failure rate deemed acceptable.

As noted in A1.1(7), structural failure is not an automatic consequence of a building being subjected to ground motion stronger than 2% in 50 years.

2) The building code tabulates the seismic hazard data for British Columbia by community (building code Appendix C, Table C2). For ease of use of this seismic hazard data in the Bridging Guidelines, the province has been geographically divided into six seismic zones as given in Table A.1-1 based on the maximum seismic spectral response acceleration value for a period of 1.0 seconds (Sa(1.0)).

Refer to Section B5.2 for a listing of the assigned seismic zone for all municipalities and regional districts in British Columbia. For the analysis given in the Bridging Guidelines, the seismicity of each seismic zone is modelled by the seismic hazard data for the representative community given in Table A.1-1.

5) Until specific guidelines are developed for Seismic Zone 6, UBC will prepare, upon request from the Ministry, custom seismic design criteria for the seismic assessment and retrofit design of any high risk school building in Seismic Zone 6 on a school by school basis.

Table A.1-1 Seismic Zones for British Columbia

Seismic Zone	Representative Community	Range in Sa(1.0)
1	Kelowna	$Sa(1.0) \leq 0.1$
2	Princeton	$0.1 < Sa(1.0) \leq 0.25$
3	Chilliwack	$0.25 < Sa(1.0) \leq 0.3$
4	Vancouver	$0.3 < Sa(1.0) \leq 0.35$
5	Victoria	$0.35 < Sa(1.0) \leq 0.4$
6	Tofino	$Sa(1.0) > 0.4$

A1.3 Definitions

1) This section clarifies the definition of all terminology used in these guidelines.

A1.4 Notations

1) The list given in this section defines every notation used in these guidelines.

A1.5 List of LDRSs

1) Table 1.1 at the end of this section lists all LDRSs included in these guidelines. Several features of this list are as follows:

(a) These guidelines include 17 LDRS prototypes and 6 diaphragm prototypes.

(b) Prototype number (eg: W-1) is the principal method of prototype identification in these guidelines

(c) The Instability Drift Limit (ISDL) values specify the maximum drift permitted for each LDRS prototype

(d) The Diaphragm Inelastic Strain Limit (DISL) values specify the maximum lateral inelastic strain permitted for each diaphragm prototype

(e) The Overstrength related force modification factor R_o is included for reference (refer to Section C2.5 for details)

A1.6 Minimum Building Structural Requirements

1) Sentence 1.6(1) lists four requirements that every school building must comply with to ensure acceptable life safety standards.

Requirement (a) in Sentence 1.6(1) is addressed at length in Sections 2-8 of these guidelines. These sections provide LDRS strength-related guidelines.

Requirement (b) in Sentence 1.6(1) is addressed in detail in Sections 10-11 of these guidelines. Diaphragms and all building connections must have adequate stiffness and strength for the effective inter-connection of LDRS and non-LDRS building elements. Diaphragms and connections have a crucial role to play in both life safety and cost-effective retrofit solutions.

Requirement (c) in Sentence 1.6(1) has already been addressed in A1.1(8). Section A5.4 provides specific requirements for checking the drift compatibility of concrete columns.

Requirement (d) in Sentence 1.6(1) has already been addressed in A1.1(7) and A1.1(9). Section 9 provides detailed guidelines for heavy partition walls.

It is vital that engineering evaluations of seismically deficient buildings focus not only on Requirement (a) but also Requirements (b), (c) and (d). All four requirements are equally important in meeting the overall life safety performance objective.

A1.7 Load Path

1) This section is a reiteration of Clause 1.6(1)(b). This reiteration is intended to emphasise the importance of load path verification and upgrading, where required.

A1.8 Assessment and Retrofit Design

1) Section A2.3 provides a more detailed explanation of the risk assessment procedure for any building.

2) Retrofit design uses the same general procedure as risk assessment as detailed in Section A2.3. Only the minimum required factored resistance values differ (retrofit values 25% higher than assessment values). The higher requirement for retrofit design is based on the rationale that, if a building is to be retrofitted, retrofit construction is usually only marginally more expensive when upgrading to a higher life safety standard.

3) Equation (1-1) permits the minimum required factored lateral resistance to be determined for any storey above the first storey. This equation uses the product of the weight at a given floor level and the square of the height of the floor above the top of the foundations to determine the dynamic distribution of building weight with building height.

4) It is crucial to verify the adequacy of the load path. Load path deficiencies can usually be upgraded in a cost-effective manner.

A1.9 Calculation of Lateral Resistance

1) The strength requirements in each of the material sections (Sections 3-8 and Section 10) apply to both existing buildings and new (retrofit) construction.

2) An LDRS is not fully effective if it commences to rock before generating its maximum resistance. Section A8 provides details on how to assess the influence of rocking on the performance of LDRSs.

3) The strength provisions of Sections 3-8 and Section 10 assume that an existing building does not have any significant strength deterioration in the principal structural materials. This assumption needs to be verified and the strength values amended accordingly if significant strength deterioration has occurred.

A1.10 Vertical Force Distribution

1) Refer to A1.8(3).

A1.11 Base Moment

1) All assessment and retrofit design resistance values are lateral resistance values. The moment at any level in a building is calculated based on the assumption that all LDRSs above that level are reaching their peak resistance simultaneously.

   The assumption of simultaneous peak resistance in all storeys is conservative for many LDRSs and is corrected through the introduction of the coefficient K_{bm}.

2) Values of K_{bm} are given in Sections 3-6. Where no values of K_{bm} are given, assume a K_{bm} value of unity.

A1.12 Drift Compatibility

1) This section is a reiteration of Clause 1.6(1)(c). This reiteration is intended to emphasise the importance of verifying drift compatibility of both LDRS and non-LDRS elements.

A1.13 Adjacency

1) Heavy localised pounding of adjacent buildings is to be avoided. The estimation of the minimum separation required to avoid pounding has been approximated by the formula given in Sentence 1.10(1).

A1.14 Irregularity

1) The building code permits structural irregularity in low-rise buildings as stated in Clause 4.1.8.7(1)(c) of the building code. The only exception in the building code is torsional sensitivity for rigid diaphragms.

   Engineering judgement needs to be exercised in assessing the seismic significance of building irregularities in low-rise school buildings. Soft storey is the most common vertical irregularity encountered in low-rise buildings. Soft storey is less common in school buildings. The Bridging Guidelines methodology permits the engineer to readily assess soft storey given that the Bridging Guidelines analysis assumes a uniform strength profile (first storey has lowest capacity/demand ratio). The Bridging Guidelines also accommodate horizontal irregularity through the maximum eccentricity provisions.

   In conclusion, buildings assessed and retrofitted using these guidelines will tolerate a reasonable degree of structural irregularity without prejudicing the life safety performance objective.

A1.15 Site Response Analysis

1) UBC research is on-going in examining the influence of soil type on earthquake damage. In the absence of specific guidance on this issue, the second edition of the Bridging Guidelines recommends that all buildings founded on Site Class E soils be subject to a site response analysis.

   See Part B of the Commentary for Tentative Guidelines for conducting a Site Response Analysis.

A1.16 Liquefaction

1) All school buildings founded on liquefiable soils (Site Class F) require a detailed geotechnical evaluation as specified in Sentence 4.1.8.4(5) of the building code.

   Liquefiable soils do not necessarily pose an unacceptable life safety risk to the building occupants. The assessment of life safety risk should be a collaborative decision of the geotechnical engineer and the structural engineer. Mitigation of building damage is outside the scope of the Ministry of Education's seismic mitigation program. Life safety is the sole objective unless agreed upon otherwise with the school district on a building by building basis.

   A future edition of the Retrofit Guidelines will provide some quantitative guidance on assessing the structural risk arising from liquefaction.

A2.0 TOOLBOX METHOD

A2.1 Introduction

1) The Toolbox method permits the engineer to utilize the contributions of all existing lateral systems in a drift compatible manner.

2) The LDRS material sections of Section 3 to Section 8 list minimum required factored resistance values for a range of drift values. Therefore, minimum required factored resistance values for a range of LDRSs can be readily chosen for a given drift limit. The Toolbox method uses this resistance/drift approach to provide a procedure that combines the strength contributions for an array of different LDRSs such that the maximum building deformation does not exceed a pre-determined drift limit.

3) The two fundamental steps in the seismic upgrade design process are risk assessment and retrofit design (if required). The Toolbox method is used in both risk assessment and retrofit design to maximize the contributions of all existing lateral systems and thereby minimize or eliminate the need for retrofit upgrading.

A2.2 Diaphragm Redistribution of Inertia Mass

1) The toolbox method can be used for (a) concrete diaphragm that have acceptable plan eccentricities (b) flexible diaphragms where the LRDSs are within 5 metres of each other or (c) LDRSs interconnected with drag struts in a line parallel to the direction of shaking.

A2.3 Detailed Toolbox Procedure

For applications that meet the requirements of Section 2.2, the procedure for combining the strength contributions from all LDRSs is as follows:

(a) First, calculate the total building weight above the mid-height of the first storey (including 25% of the roof snow load).

(b) Next, determine the maximum permissible drift for all LDRSs. The maximum permissible drift is determined by the lowest ISDL in the group of LDRSs.

If the lowest ISDL is too low for the effective contribution of many LDRSs, one option is to demolish the LDRS with the low ISDL or upgrade the LDRS to increase its ISDL. Note that vertical load-bearing elements that are not part of an LDRS must also be included in the determination of the maximum permissible drift (refer to Sentence A5.4(1)).

The final selection of the maximum permissible drift is referred to as the Governing Drift Limit (GDL).

(c) Calculate the factored resistance R_e of each LDRS based on the requirements of Sections 3.3, 4.3, 5.3, 6.3, 7.3 and 8.3. Note that, for each LDRS, R_e is expressed as a percentage of the total weight of the building calculated in step (a).

(d) For the GDL determined in step (b), determine the minimum required factored resistance R_m for each LDRS based on the requirements of Sections 3.2, 4.2, 5.2, 6.2, 7.2 and 8.2.

(e) For each LDRS, calculate the factored resistance ratio R_r (%) using equation (1-1).

The factored resistance ratio R_r is a direct measure of the percentage of the total weight of the building that the particular LDRS can support without exceeding the maximum permissible drift (GDL).

(f) Based on the results of Step (e), the total percentage of the building weight R_{rt} that can be supported by all LDRSs within the GDL is readily determined by summing all R_r values.

(g) For either risk assessment or retrofit design, the value of R_{rt} must equal or exceed 100%.

(h) If R_{rt} is less than 100% in the risk assessment phase, the building will require upgrading by the addition of new LDRSs or the upgrading of the existing LDRSs.

If R_{rt} is less than 100% in the retrofit design phase, the most cost-effective solution is to increase the lateral resistance of the new LDRSs that have been added to the building.

A3.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR LOW-RISE WOOD FRAME SCHOOL BUILDINGS

A3.1 Prototypes

1) In the second edition, wood frame prototypes are restricted to common forms of shearwall construction, both blocked and unblocked. Sheathing material is in the form of sheets or boards. Note that no distinction is made between plywood and oriented strand board (OSB) in the resistance tables given in Section 3.

   Modelling details for each of the wood frame prototypes are given in Section C5.2. Diagonal board shearwalls are assumed to have the same shape of backbone curve and the same hysteretic curve as blocked OSB/plywood shearwalls. Similarly, horizontal board sheathing is assumed to have similar backbone and hysteretic curves to those for gypsum wallboard.

   Although unblocked OSB/plywood and unblocked gypsum wallboard has some differences in their hysteretic and backbone curves, the minimum required factored resistance (R_m) results are very similar. Therefore, the R_m results for unblocked OSB/plywood have been assigned to all four forms of unblocked construction listed for prototype W-2.

2) Before proceeding with calculating the wood frame lateral resistance in accordance with Section 3, it is necessary to check that the shearwall does not rock before it develops its maximum resistance with no uplift. Figure A.8-3 in Section A8 illustrates a wood frame shearwall that is governed by uplift/rocking.

   Proceed to Section 8 for details on how to check if rocking governs. For wood frame shearwalls, use rocking prototype #R-2 for cantilevers with a maximum aspect ratio of 1.0. Adopt prototype R-3 for cantilevers with a maximum aspect ratio of 2.5. This basis for selecting wood frame prototypes reflects the greater flexibility of wood frame shearwalls relative to that for concrete, masonry or braced steel LDRSs.

A3.2 Minimum Required Lateral Factored Resistance R_m

1) For all materials, the minimum required factored resistance values for risk assessment are set at 80% of the corresponding retrofit values as detailed in Section C2.3.

   For the given seismic zone and soil type, the minimum required factored resistance for a retrofit design is read from the tables immediately below Figures 3-1 to Figure 3-3 for the selected value of the Governing Drift Limit. The corresponding assessment value is 80% of this retrofit value.

3) Minimum required lateral factored resistance reduces linearly with increasing clear storey height. Equation (3-1) is restricted to a range in clear storey height of 3-4 metres. The maximum reduction in resistance is 25% for a 4 metre clear storey height.

A3.3 Calculation of Lateral Resistance

2) The limitation on the factored resistance ratio R_r for unblocked gypsum wallboard has been introduced to ensure that a majority of the wood frame lateral resistance is generated by wood sheathing (OSB/plywood or diagonal boards).

3) Horizontal boards exhibits poor ductility characteristics and should not be the dominant LDRS in the assessment of a building.

A3.4 Wood Frame Base Moments

1) For 2-3 storey buildings, the maximum base moment is 20% - 30% less than the moment corresponding to the peak resistance being generated simultaneously in all storeys. This significant reduction in base moment primarily arises from the degradation in first storey strength at relatively high maximum permissible drifts (ISDL = 4%).

   The above reduction in base moment does not apply to one storey buildings and buildings located in Seismic Zone 2.

A3.5 Strength of Existing Materials

1) The most efficient seismic retrofit solutions incorporate the damage mitigation contributions from existing materials to complement the contributions of new materials that may be added for improved performance.

Table 3.1 gives the lateral factored strengths of three types of existing sheathing. Note that horizontal boards are not permitted in new construction because of its poor ductility characteristics.

To use the resistance values given in Table 3.1, the minimum nailing requirements of Table A3.1 must be satisfied. If nailing is less than the minimum requirements given in Table A3.1, the resistance should be prorated accordingly by the inverse ratio of nail spacing and by the ratio of individual nail resistance.

Table A.3-1 Minimum Construction Requirements for Lateral Factored Resistance of Selected Existing Materials

No.	**Sheathing or Finish**	**Minimum Requirements**	
		Fastener	*Spacing*
1	Unblocked OSB (11 mm)	51 mm nails	152 mm (edges)(1)
2	Unblocked plywood (9.5 mm)	51 mm nails	152 mm (edges)(1)
3	Gypsum wallboard (12.7 mm)	32 mm ring nails	203 mm (2)
4	Horizontal boards (19x184)	64 mm nails	2 nails/board

Notes: (1) Unblocked OSB/plywood has 51 mm nails at 305 mm spacing along intermediate supports
(2) Gypsum wallboard has 35 mm screws at 406 mm spacing along intermediate supports

A4.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR LOW-RISE STEEL SCHOOL BUILDINGS

A4.1 Prototypes

1) A steel LDRS is not a common form of construction in British Columbia schools. The small number of school buildings with steel LDRSs typically utilize concentrically braced frames. The list of steel prototypes in the second edition has been expanded to include contemporary retrofit solutions such as eccentrically braced frames. Moderately ductile steel moment-resisting frames have also been included to provide engineers with a larger choice of retrofit options.

   Modelling details for each of the steel prototypes are given in Section C5.3.

   Concentrically braced steel frames with tension/compression braces (Prototype S-2) have a range of drift limits as given in Table 1.1. The assigning of the appropriate drift limit for this steel prototype is made on the basis of the type of brace section and its d/t ratio as detailed in Table 4.1.

   Additionally the compression members of a tension/compression brace (Prototype S-2) must have a compression strength equal to 30% or more of the tension strength of the weaker brace. If the compression strength is less than 30%, use the tension only braced frame prototype (S-1).

2) Tension/compression braces exhibit good resistance in tension but a relatively low post-buckling residual strength (assumed to be 20% of its tensile strength). Refer to Section C5.3 for the shape of the hysteretic curve for this prototype.

   A single tension/compression brace is not permitted in a single bay or in a pair of bays not more than 5 metres apart as illustrated in Figure A.4-1. For effective braced frame resistance, tension/compression braces need to be provided in matching pairs as illustrated in Figure A.4-2. A matched pair has one brace in tension when its matching brace is in compression. In this configuration, the resistance of the braced frame maintains a substantial lateral resistance as opposed to the major fluctuations of a single brace that has high tensile resistance and low post-buckling compressive resistance.

   An existing steel frame with a single brace can be assessed using 80% of the resistance values given in Table A.4-1. Utilizing single brace frames as the primary LDRSs is not recommended in assessing risk. The contribution of a single brace frame can be included as that for a secondary LDRS that complements a primary LDRS of different construction.

For the reasons given above, steel frames with one braced bay with a single brace are not permitted as a method of retrofit.

3) A typical chevron braced frame is illustrated in Figure A.4-3. The major deficiency in a typical chevron frame is the tendency of the horizontal brace to fail in bending or lateral torsional buckling when the compression brace buckles (large net vertical force component acting on the beam). For this reason, chevron braces are not to be included in the contributing LDRSs in risk assessment unless a detailed analysis indicates otherwise. Existing chevron braces may be retrofitted to be effective LDRSs by upgrading the beam element to withstand the vertical brace force imbalance generated by brace buckling.

4) Before proceeding with calculating the steel lateral resistance in accordance with Section 4.3, it is necessary to check that the steel LDRS does not rock before it develops its maximum resistance with no uplift. Figure A.8-4 in Section A8 illustrates a steel braced frame that is governed by uplift/rocking.

   Proceed to Section 8 for details on how to check if rocking governs. For rocking braced frame LDRSs, use rocking prototype #R-2. For rocking steel moment frames, use rocking prototype R-3. These selections of rocking prototypes are based on yield drifts comparable with those of the steel LDRSs.

5) Table 4.1 provides guidance on the appropriate choice of the ISDL value for a concentrically braced steel frame with hollow section braces. Large diameter, thin-walled hollow section braces do not perform as well as smaller diameter, thicker-walled hollow section braces.

6) Detailing of the load path is especially important in steel LDRSs. The guidelines in this steel building section are based on connections, drag struts and similar load path detailing all being compliant with the requirements of this second edition.

A4.2 Minimum Required Lateral Factored Resistance R_m

1) For all materials, the minimum required factored resistance values for risk assessment are set at 80% of the corresponding retrofit values as detailed in Section C2.3.

2) For the given seismic zone and soil type, the minimum required factored resistance for a retrofit design is read from the tables immediately below Figures 4-1 to Figure 4-4for the selected value of the Governing Drift Limit. The corresponding assessment value is 80% of this retrofit value.

3) For steel braced frames with tension/compression braces, the minimum required lateral factored resistance values given in Figure 4-2 are based on the horizontal component of the tensile strength of the weaker tension brace in the brace pair.

Defining the resistance of a brace in terms of its tensile strength has been adopted because the residual compressive strength of a buckled brace is relatively low.

4) R_m, the minimum required factored resistance, reduces with increasing clear storey height. Equation (4-1) yields a reduction of 15% in the value of R_m for a clear storey height of 4 metres.

A4.3 Calculation of Lateral Resistance

2) Columns in steel braced frames and in steel moment-resisting frames must not be the governing weak element in the lateral system. The columns of a steel frame must have a strength in excess of the maximum column forces generated by frame action and gravity loads.

A4.4 Steel Base Moments

1) The overturning moments in steel buildings, calculated in accordance with Sentence 1.11(1), do not benefit from the reductions typical of wood frame, concrete or masonry buildings. The steel prototype backbone curves (excluding buckling portion of tension/compression brace) do not exhibit any strength degradation at higher levels of drift.

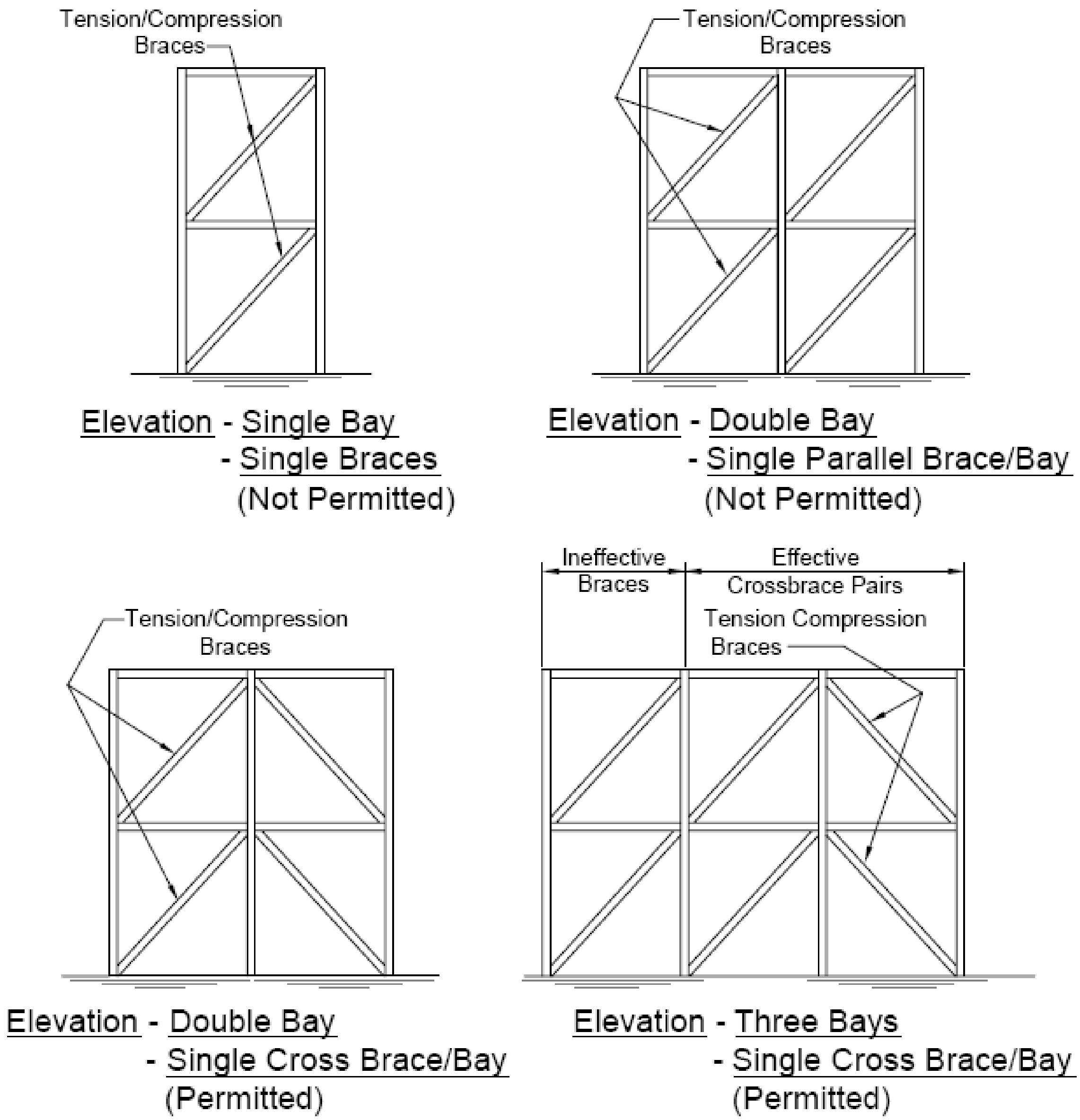


Figure A.4-1 Tension/Compression Braces Layouts

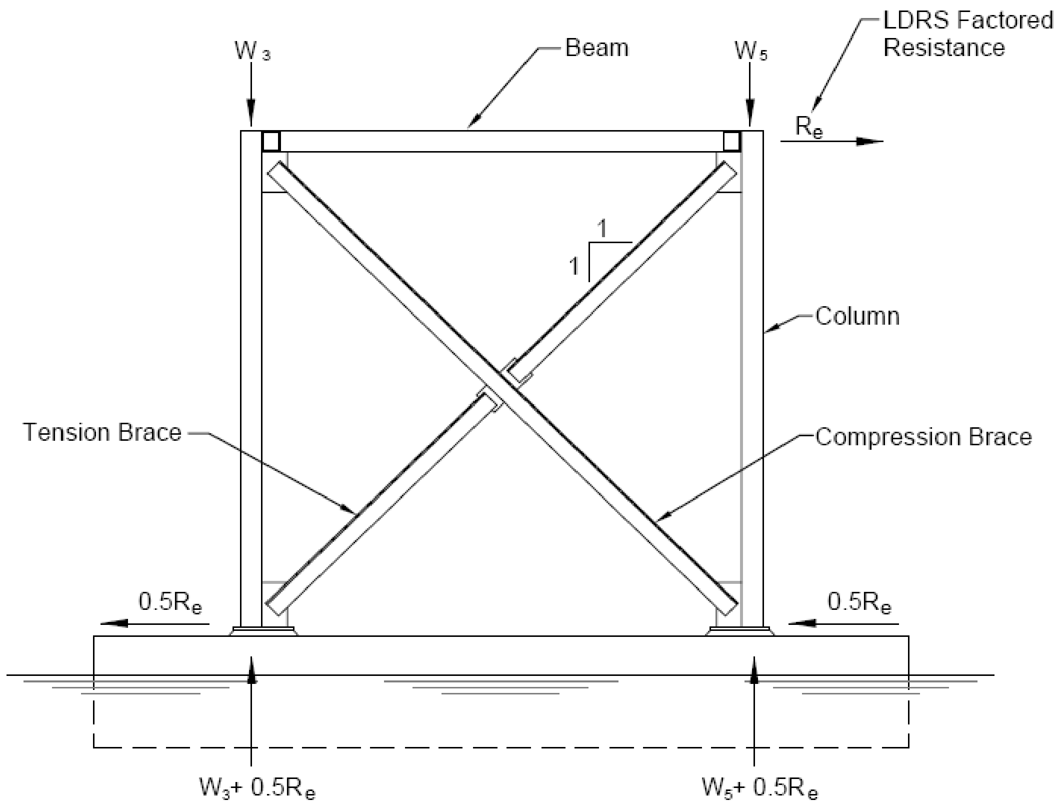


Figure A.4-2 Concentrically Braced Steel Frame with Tension/Compression Braces

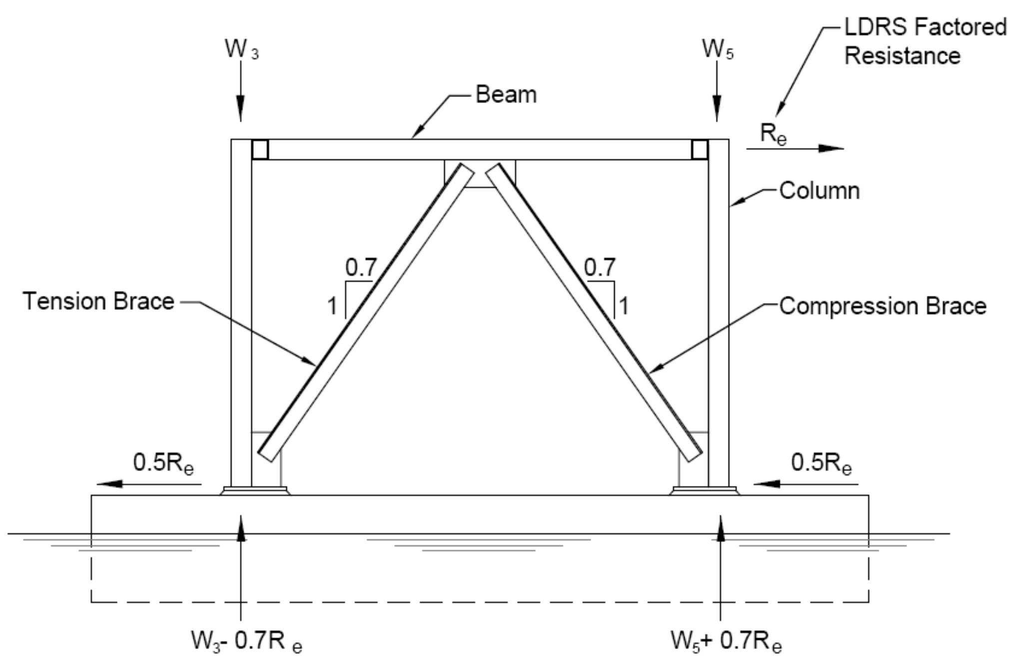


Figure A.4-3 Typical Steel Chevron Braced Frame

A5.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR LOW-RISE CONCRETE SCHOOL BUILDINGS

A5.1 Prototypes

1) The two basic types of concrete LDRSs in the second edition are shearwalls and moment frames.

   For the two shearwall prototypes, no distinction is made between shear and flexure because both prototypes have similar resistance analytical results. Both prototypes have the same backbone curve and hysteretic curves (refer to Section C5.4).

2) The distinction between a shearwall and a column is made on the basis of the element's aspect ratio. Elements with an aspect ratio in excess of 4.0 shall be treated as columns. Elements with an aspect ratio not exceeding 4.0 shall be treated as shearwalls.

3) The distinction between the three types of concrete moment frames is made on the following basis:

   (a) Ductile concrete moment resisting frames (Prototype C-3) conform to Section 21.3 or Section 21.4 of CSA-A23.3-04
   (b) Moderately ductile concrete moment resisting frames (Prototype C-4) conform to Section 21.7 of CSA-A23.3-04
   (c) Conventional construction concrete moment resisting frames (Prototype C-5) conform to Section 21.8 of CSA-A23.3-04

   Unless it can be verified otherwise, all existing concrete moment frames shall be classified as conventional construction.

4) Before proceeding with calculating the concrete lateral resistance in accordance with Section 5.3, it is necessary to examine the influence of rocking on the lateral resistance of the concrete LDRS. Figure A.8-4 and Figure A.8-5 illustrate two scenarios of concrete LDRSs exhibiting different rocking characteristics.

   Proceed to Section 8 for details on how to evaluate the influence of rocking on the lateral resistance of concrete LDRSs. For rocking concrete shearwall LDRSs, choose the appropriate rocking prototype based on the shearwall aspect ratio. For rocking concrete moment frames, use rocking prototype R-3. Prototype #R-3 has a yield drift comparable to that for a concrete moment frame.

5) Short columns have the potential for brittle shear failure for modest inter-storey drift when the clear storey height of the columns is substantially less than the inter-storey height. For this reason, short columns that have a maximum permissible drift value less than 1%, calculated using Equation (A.5-1) and prorated by h_s/h_{sc}, are to be excluded from contributing to the resistance checks in assessment and retrofit design.

A5.2 Minimum Required Lateral Factored Resistance R_m

1) For all materials, the minimum required factored resistance values for risk assessment are set at 80% of the corresponding retrofit values as detailed in Section C2.3.

For the given seismic zone and soil type, the minimum required factored resistance for a retrofit design is read from the tables immediately below Figures 5-1 to Figure 5-5for the selected value of the Governing Drift Limit. The corresponding assessment value is 80% of this retrofit value.

3) R_m, the minimum required lateral factored resistance, decreases with increasing clear storey height. The reduction in R_m is a maximum of 15% at a clear storey height of 4 metres.

A5.3 Calculation of Lateral Resistance

2) Lap splices require close scrutiny in determining the lateral resistance of existing buildings. Inadequate lap splices affect both strength and the maximum drift limit. The tensile strength of all reinforcing bars that do not meet the lap splice requirements of CSA-A23.3-04 shall be set to zero. This will result in shearwall rocking. Retrofitting of the inadequate lap splices is one option for improving performance.

A5.4 Concrete Building Base Moments

1) The base moment for a concrete LDRS is 10% less than that calculated using Sentence 1.11(1) if a two storey concrete building has a light roof or if the concrete building is three storeys in height regardless of the type of roof.

A5.5 Non-LDRS Drift Compatibility

1) Sentence 5.5(1) requires that load-bearing non-LDRS framing elements to be capable of maintaining their support of vertical load for inelastic building deformations up to the Governing Drift Limit. This issue is especially important for non-ductile concrete columns. Equation (A.5-1) given below provides an estimate of the maximum clear height drift that a potentially non-ductile concrete column can accommodate with a low probability of axial load failure.

$$ISDL(\%) = 4 \cdot \frac{1 + \tan^2(65°)}{\tan(65°) + P\left(\dfrac{s}{A_{st} f_{yt} d_c \tan(65°)}\right)} - 1 \qquad \text{(A.5-1)}$$

but need not be taken as less than 1%,

where:

- $ISDL$ = maximum permissible drift limit
- P = axial load
- s = spacing of transverse reinforcement
- A_{st} = area of transverse reinforcement
- f_{yt} = yield strength of transverse reinforcement
- d_c = depth of core (centerline to centerline of ties)

Equation (A.5-1) was developed based on a limited data set of nominally fixed-fixed columns subjected to repeated cycles until axial failure was initiated. Further details on the development and applicability of Equation (A.5-1) can be found in Elwood and Moelhle (2005).

A6.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR LOW-RISE CONCRETE MASONRY SCHOOL BUILDINGS

A6.1 Prototypes

1) The two concrete masonry prototypes are unreinforced concrete masonry that deforms by in-plane sliding and reinforced masonry shearwalls subject to flexure or shear.

   Both prototypes are classified as conventional construction shearwalls (nominal ductility). Unreinforced concrete masonry is considered to exhibit a nominal degree of ductility when it deforms by in-plane sliding within predetermined inelastic drift limits.

   For the reinforced masonry shearwall prototype, no distinction is made between shear and flexure because both prototypes have similar resistance values. The two prototypes have the same backbone but dissimilar hysteretic curves (refer to Section C5.5).

2) Before proceeding with calculating the concrete masonry lateral resistance in accordance with Section 6.3, it is necessary to examine the influence of rocking on the lateral resistance of the concrete masonry LDRS. Figure A.8-4 and Figure A.8-5 illustrate two scenarios of concrete masonry LDRSs exhibiting different rocking characteristics.

   Proceed to Section 8 for details on how to evaluate the influence of rocking on the lateral resistance of concrete masonry LDRSs.

A6.2 Minimum Required Lateral Factored Resistance R_m

1) For all materials, the minimum required factored resistance values for risk assessment are set at 80% of the corresponding retrofit values as detailed in Section C2.3.

   For the given seismic zone and soil type, the minimum required factored resistance for a retrofit design is read from the tables immediately below Figures 6-1 or Figure 6-2 for the selected value of the Governing Drift Limit. As noted above, the assessment value is 80% of the corresponding retrofit value.

3) The value of R_m, the minimum required lateral factored resistance, reduces with increasing clear storey height. The maximum reduction in R_m is 15% for a 4 metre clear storey height.

A6.3 Calculation of Lateral Resistance

2) Equation (6-2) in Sentence 6.3(2) assumes a coefficient of friction of 1.0 at the sliding interface. The vertical compressive load across the sliding surface is the weight of the masonry above the sliding surface.

A6.4 Masonry Building Base Moments

1) The overturning moment at the base of a concrete masonry LDRS is reduced by 10% for a two storey masonry building with a light roof and for a three storey masonry building, irrespective of the type of roof construction.

A6.5 Out-of-Plane Requirements

1) The out-of-plane requirements of Section 6.5 apply to all concrete masonry walls, both load-bearing and non load-bearing walls (partition and in-fill walls).

2) With one exemption, all concrete masonry walls are to be reinforced for out-of-plane behaviour. Concrete masonry walls are exempt from the reinforcement requirements if all of the requirements of Sentence 6.5(3) are met.

3) The first of two scenarios for unreinforced concrete masonry walls having acceptable out-of-plane behaviour is the combination of the building being founded on Site Class C/D soils and the unreinforced walls being fully confined top and bottom (refer to Figure A.6-1). The second scenario is a building underlain by the same type of soils (Site Class C/D) and lateral braces providing out-of-plane support for the unreinforced walls (refer to Figure A.6-2).

4) As illustrated in Figure A.6-1, confinement of unreinforced masonry walls for out-of-plane behaviour is provided by one of two methods.

   The first confinement method is provide a stiff concrete member at the top and the base of the wall such that the unreinforced wall cannot fail by out-of-plane rocking as illustrated in Figure A.6-3. For example, the top edge of a 200 mm thick unreinforced concrete masonry wall 3500 mm high will need to rise 23 mm vertically to permit total out-of-plane rocking failure (centre of wall must move 200 mm out-of-plane for total failure). If the wall was confined by a stiff concrete frame top and bottom, the top edge of the wall would not be able to rise 23 mm vertically and the wall would therefore maintain out-of-plane stability.

   The second confinement method is illustrated in Figure A.6-4. Instead of providing confinement, a vertical downward force equal to 50% of the weight of the wall is sufficient to restrict the maximum out-of-plane movement of the wall to less than the wall thickness required for total failure. As defined in Section 1.3, surcharge is the weight of the building bearing on the top of the wall. This surcharge would be fully mobilised by an unreinforced masonry wall rocking out-of-plane.

Both methods of confinement are restricted to walls at least 140 mm in thickness and no more than 3.5m in height. This combination of maximum wall height and minimum wall thickness ensures a reasonable margin of safety for both confinement and stability generated by surcharge loading.

5) Lateral bracing of unreinforced concrete masonry walls to ensure out-of-plane stability is illustrated in Figure A.6-2.

   The braces are to be spaced no more than 50% of the wall height (maximum of 2.4 metres). Each brace must have a factored moment resistance no less than R_{mb} defined in Equation (6-2). Each connection between the vertical bracing element and the brace-supported unreinforced masonry wall must have a factored resistance no less than R_{mc} defined in Equation (6-3).

6) Concrete masonry walls are not subject to peak in-plane and out-of-plane shaking simultaneously. Therefore, wall reinforcement is set equal to the greater of the in-plane or out-of-plane requirements, not the combined requirements.

A6.6 In-fill Walls

2) In-fill walls are capable of generating large in-plane resistance through the formation of a compression strut from corner to corner of the in-fill wall. The high compressive stresses in these corners can result in failure of the corner blocks. The failure of the top corner blocks is a potential life safety hazard. Hazard abatement measures need to be instituted to mitigate this hazard.

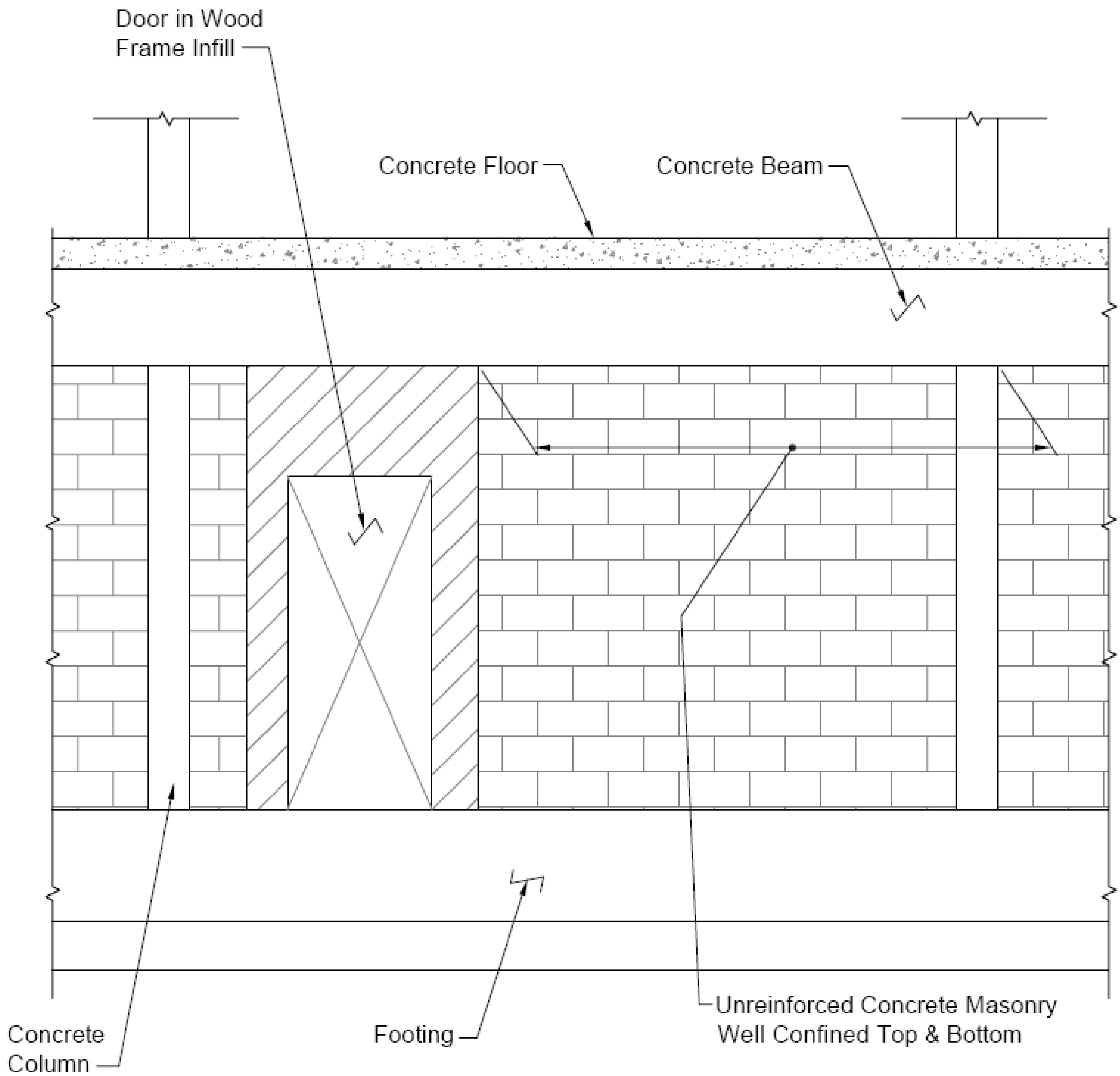


Figure A.6-1 Confined Unreinforced Concrete Masonry Wall

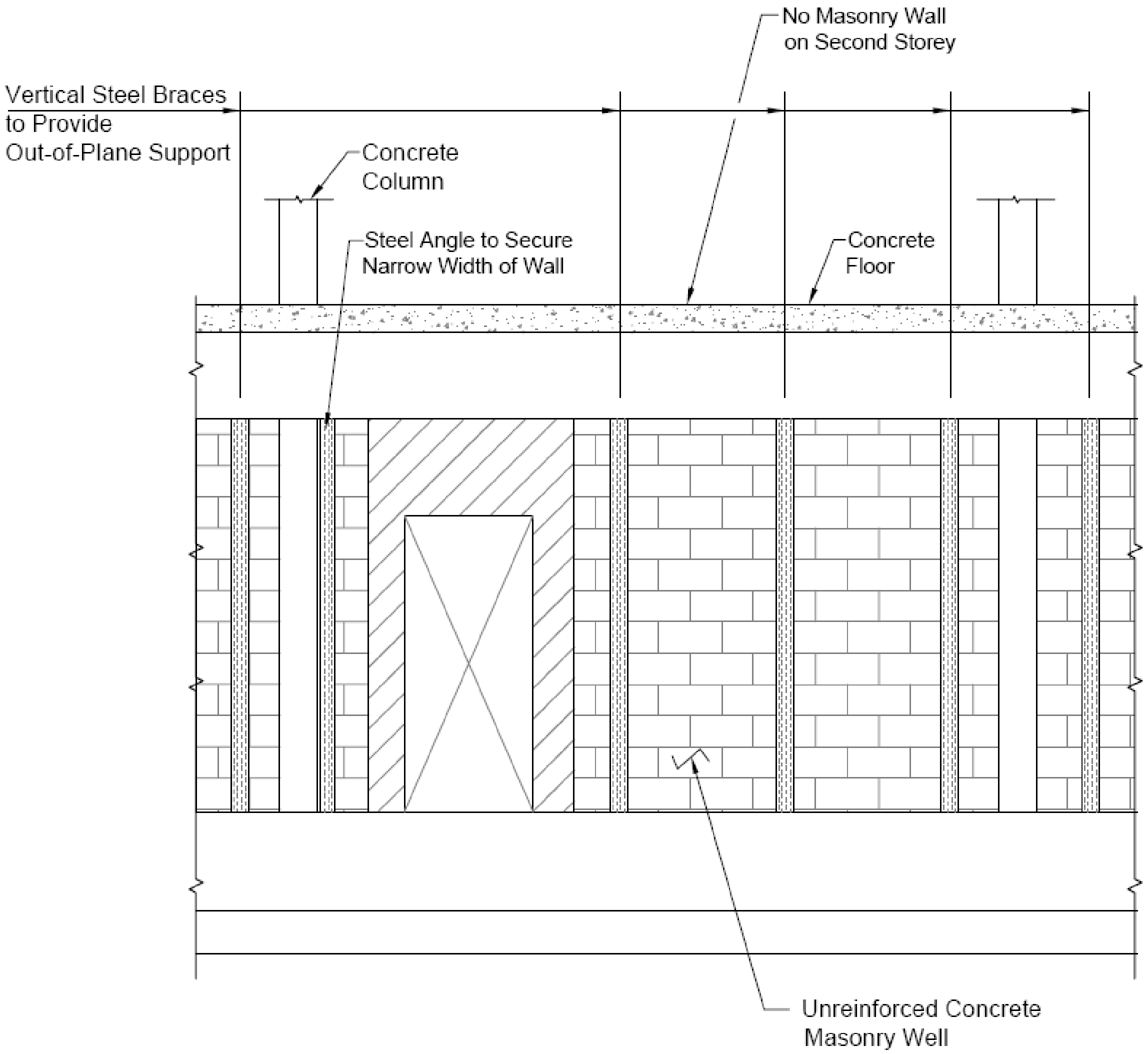


Figure A.6-2 Unreinforced Concrete Masonry Vertical Steel Bracing

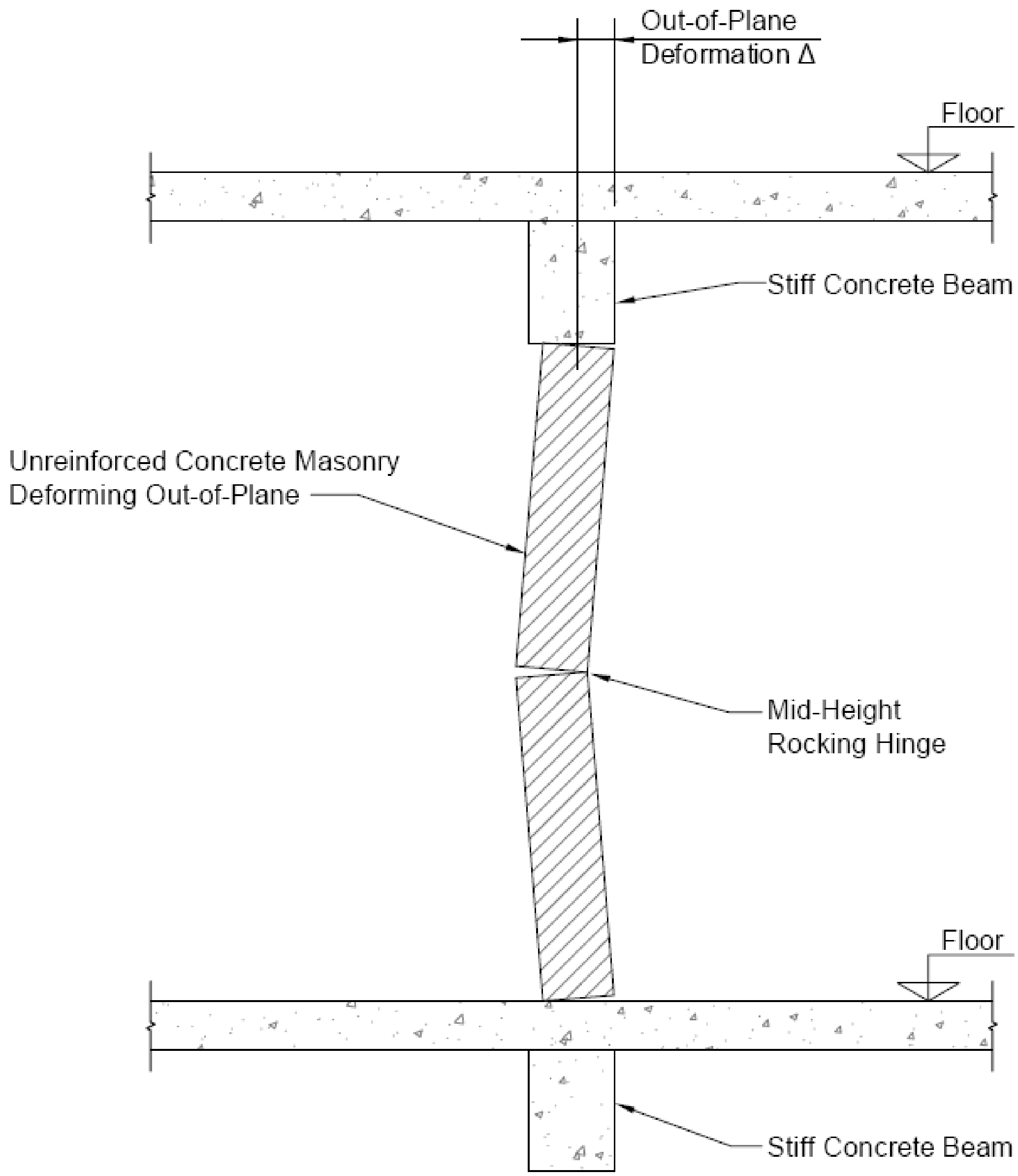


Figure A.6-3 Stiff Confinement of Unreinforced Masonry Wall

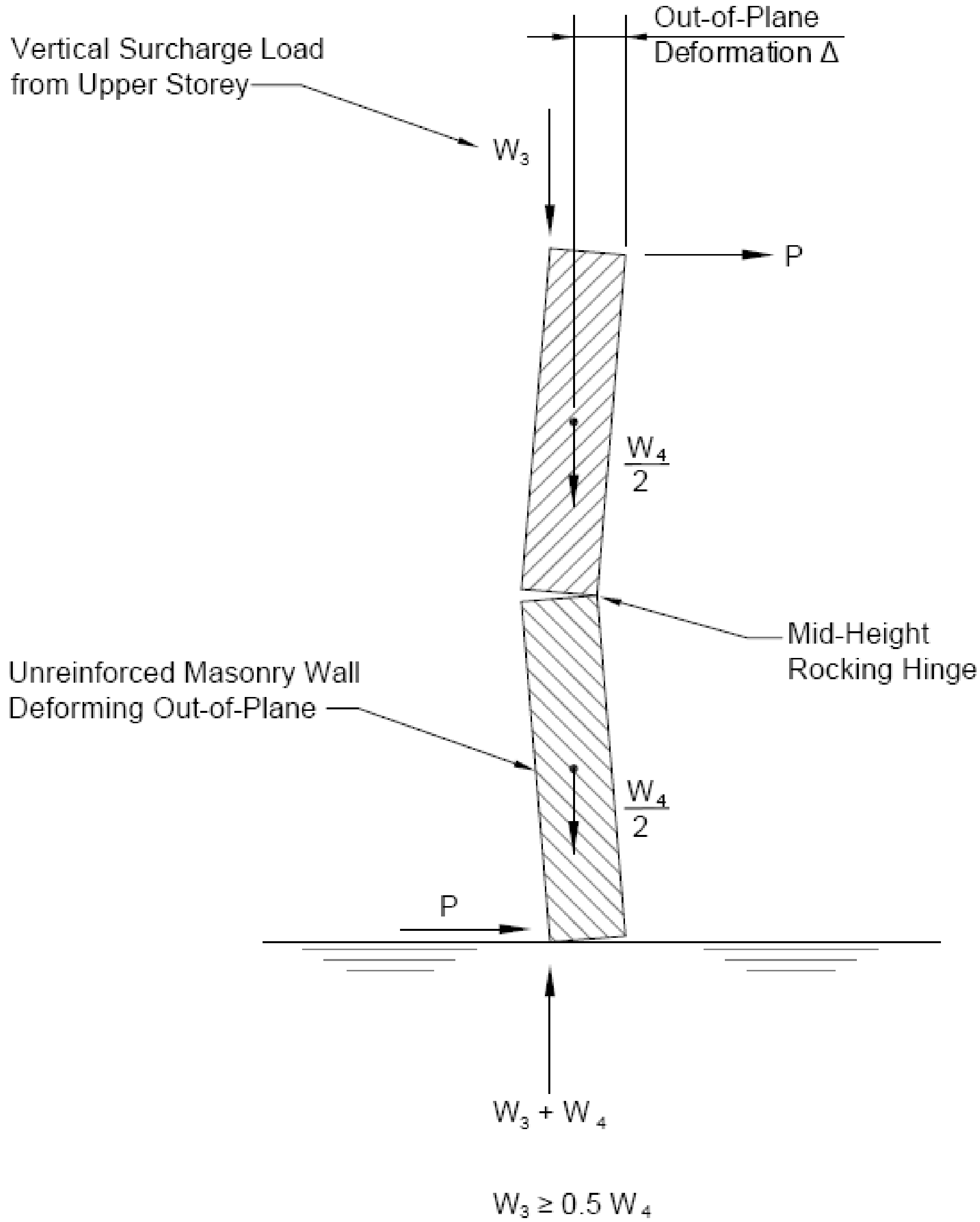


Figure A.6-4 Surcharge Stabilized Unreinforced Masonry Wall

A7.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR LOW-RISE CLAY BRICK MASONRY SCHOOL BUILDINGS

A7.1 Prototypes

1) Only one prototype is provided for clay brick masonry construction. All clay brick masonry construction is assumed to be unreinforced.

   The clay brick masonry prototype is similar to its unreinforced concrete masonry counterpart, Prototype M-1. Both prototypes deform by in-plane sliding. Unreinforced clay brick masonry is considered to exhibit a nominal degree of ductility when it deforms by in-plane sliding within predetermined inelastic drift limits (Table 1.1).

2) A second common prototype for unreinforced clay brick masonry construction is the rocking element prototype as detailed in Section 8. The geometry of clay brick masonry piers between windows is such that these piers will rock rather than slide. Refer to Section 8 for details.

A7.2 Minimum Required Lateral Factored Resistance R_m

1) For all materials, the minimum required factored resistance values for risk assessment are set at 80% of the corresponding retrofit values as detailed in Section C2.3.

   For the given seismic zone and soil type, the minimum required factored resistance for a retrofit design is read from the tables immediately below Figure 7-1 for the selected value of the Governing Drift Limit. As noted above, the assessment value is 80% of the corresponding retrofit value.

3) The value of R_m, the minimum required lateral factored resistance, reduces with increasing clear storey height. The maximum reduction in R_m is 15% for a 4 metre clear storey height.

A7.3 Calculation of Lateral Resistance

1) The lateral resistance of clay brick masonry LDRSs deforming by sliding is calculated using Equation (7-2). This equation is equivalent to Equation (6-2) for sliding unreinforced concrete masonry.

   Equation (7-2) assumes a coefficient of friction of 1.0 at the sliding interface.

A7.4 Out-of-Plane Requirements

1) The out-of-plane requirements of Section 7.4 apply to all clay brick masonry walls, both load-bearing and non load-bearing (partition) walls.

   Clay brick masonry walls have acceptable out-of-plane behaviour if they have a minimum wall thickness (three wythes), bonding courses are present at least every six courses to provide an adequate degree of composite wall action, the wall height to thickness ratio does not exceed the values given in Table 7.1 and a diaphragm provides effective out-of-plane restraint at the top of the wall.

2) All clay brick masonry walls that do not meet the requirements of Sentence (1) are to be supported out-of-plane by vertical bracing elements that conform to the strength, connection and spacing requirements of Sentence 6.4(5).

3) Clay brick masonry walls in buildings founded on Site Class E soils are to be braced as detailed in (2) or are to be subject to a custom evaluation by UBC to determine the appropriate design criteria based on the results of a site response analysis.

A8.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR ROCKING ELEMENTS IN LOW-RISE SCHOOL BUILDINGS

A8.1 Prototypes

1) Three prototypes are provided in Section 8 to model the behaviour of building elements that generate lateral resistance by rocking as opposed to flexure, shear or sliding. These three rocking prototypes are for all four major construction materials (wood, steel, concrete and masonry).

   The distinguishing feature for the three prototypes is the aspect ratio of the rocking element as detailed in Sentence (2).

2) Lower aspect ratio rocking elements are typically modelled by Prototype R-1. Sentence 8.1(2) provides the maximum aspect ratio limits for each prototype. A pier is a rocking element that has substantial confinement at both the base and the top of the pier. A cantilever has nominal confinement at the top of the cantilever. Refer to Figure A.8-1 for an illustration of these two types of rocking elements. Refer to Section 1.3 for the definitions of piers and cantilevers.

A8.2 Minimum Required Lateral Factored Resistance R_m

1) For all materials, the minimum required factored resistance values for risk assessment are set at 80% of the corresponding retrofit values as detailed in Section C2.3.

   For the given seismic zone and soil type, the minimum required factored resistance for a retrofit design is read from the tables immediately below Figure 8-1 to Figure 8-3 for the selected value of the Governing Drift Limit. As noted above, the assessment value is 80% of the corresponding retrofit value.

3) The value of R_m, the minimum required lateral factored resistance, decreases with increasing values of H, the height of the building centre of mass above the underside of the rocking footing. The maximum reduction is 33% for a value of H = 6.0 metres. Reduced values of R_m are typical of rocking cantilevers (full height concrete shearwall).

4) The value of R_m, the minimum required lateral factored resistance, increases with decreasing values of H, the height of the building centre of mass above the underside of the rocking footing. The maximum increase is 40% for a value of H = 1.5 metres. Increased values of R_m are typical of rocking piers (unreinforced masonry piers between windows).

A8.3 Calculation of Lateral Resistance

1) The lateral resistance of a building element rocking above its foundation is calculated using Equation (8-3). The generation of this lateral resistance is illustrated in Figure A.8-2.

2) If the foundation rocks, the lateral resistance is calculated using Equation (8-4). Refer to Figure A.8-3 for an illustration of this type of lateral resistance ($L=a$ and $W_3=0$).

A8.4 Rocking Check

1) The influence of rocking on lateral resistance is prototype dependent. Specific rocking characteristics for each prototype are as follows:

(a) Wood Frame Prototypes W-1, W-2 and W-3
Refer to Figure A.8-4. These wood frame prototypes will rock when the uplift restrain is not sufficient to generate the maximum lateral shear strength of the wood frame panel. The appropriate prototype is R-3 (similar yield drift to that for wood frame prototypes).

(b) Steel Prototypes S-1, S-2, S-3 and S-4
A steel prototype will rock when the uplift restrain is not sufficient to generate the maximum lateral shear strength of the steel frame. Refer to Figure A.8-5 for an illustration of a rocking steel frame. The appropriate prototype is R-1 for braced steel frames and R-3 for steel moment frames (similar yield drifts to those for steel prototypes).

(c) Concrete Prototypes C-1, C-2 and Concrete Masonry Prototype M-2
These three prototypes behave differently to the other prototypes in their rocking characteristics. These three prototypes with the potential for rocking are reinforced shearwalls that deform inelastically in flexure.

If any of these three prototypes are governed by flexure, the lateral resistance is provided by a combination of both flexure and rocking as illustrated in Figure A.8-2. As the tension steel yields and the top edge of the shearwall lifts, the shearwall will generate a vertical restraint force in addition to its self weight. The total factored resistance ratio R_{rt} for this shearwall is the sum of (i) the factored resistance ratio for flexure excluding vertical loads and (ii) the factored resistance ratio of the rocking shearwall based on the shearwall self weight and the vertical restraint force at its top edge.

Rocking is not considered significant if these three prototypes are governed by shear.

(d) Concrete Prototypes C-3, C-4 and C-5

In an unusual situation where a concrete moment frame rocks, the rocking characteristics are similar to those for the rocking steel moment frame prototype.

(e) Concrete Masonry Prototype M-1 and Clay Brick Masonry Prototype B-1

These two prototypes will rock if their overturning moment is greater than their restoring moment as illustrated in the generic Figure A.8-6.

(f) Rocking Footing

The footing of any of the above LDRSs will rock if the overturning moment exceeds the restoring moment as illustrated in Figure A.8-3.

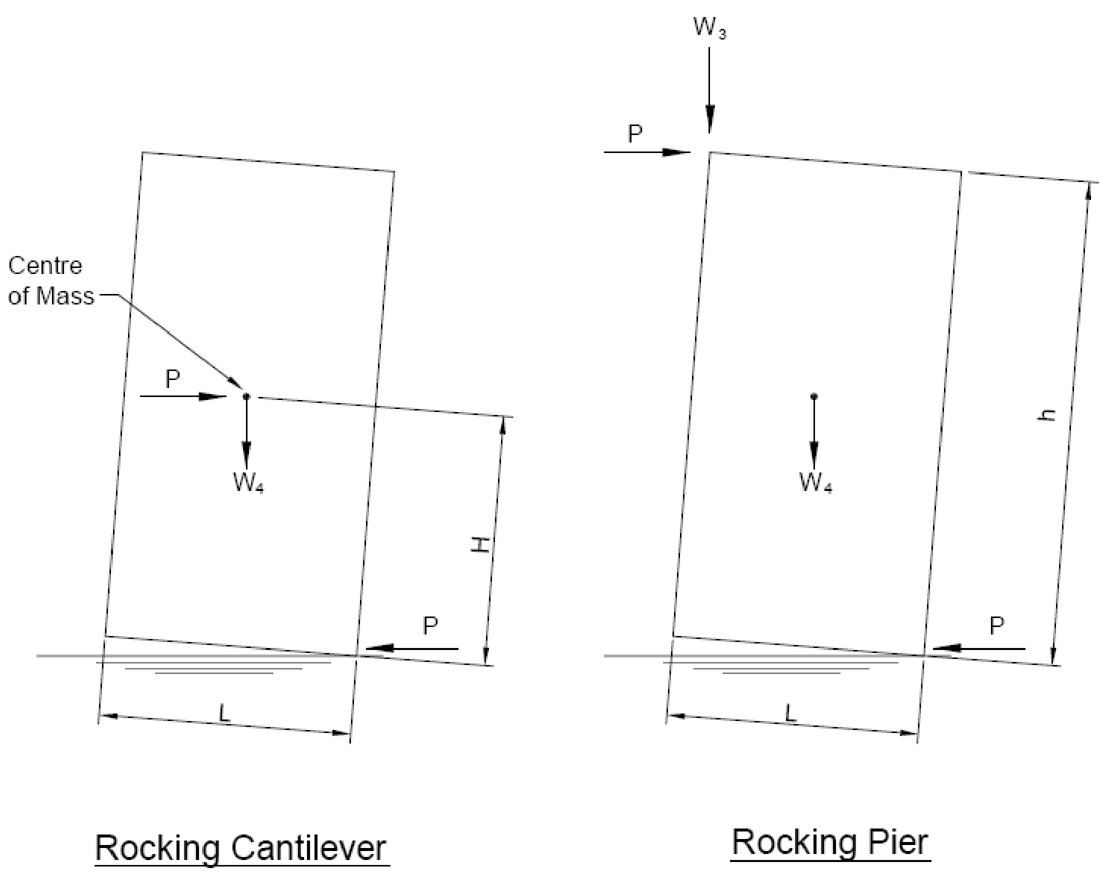


Figure A.8-1 Rocking Cantilever and Pier

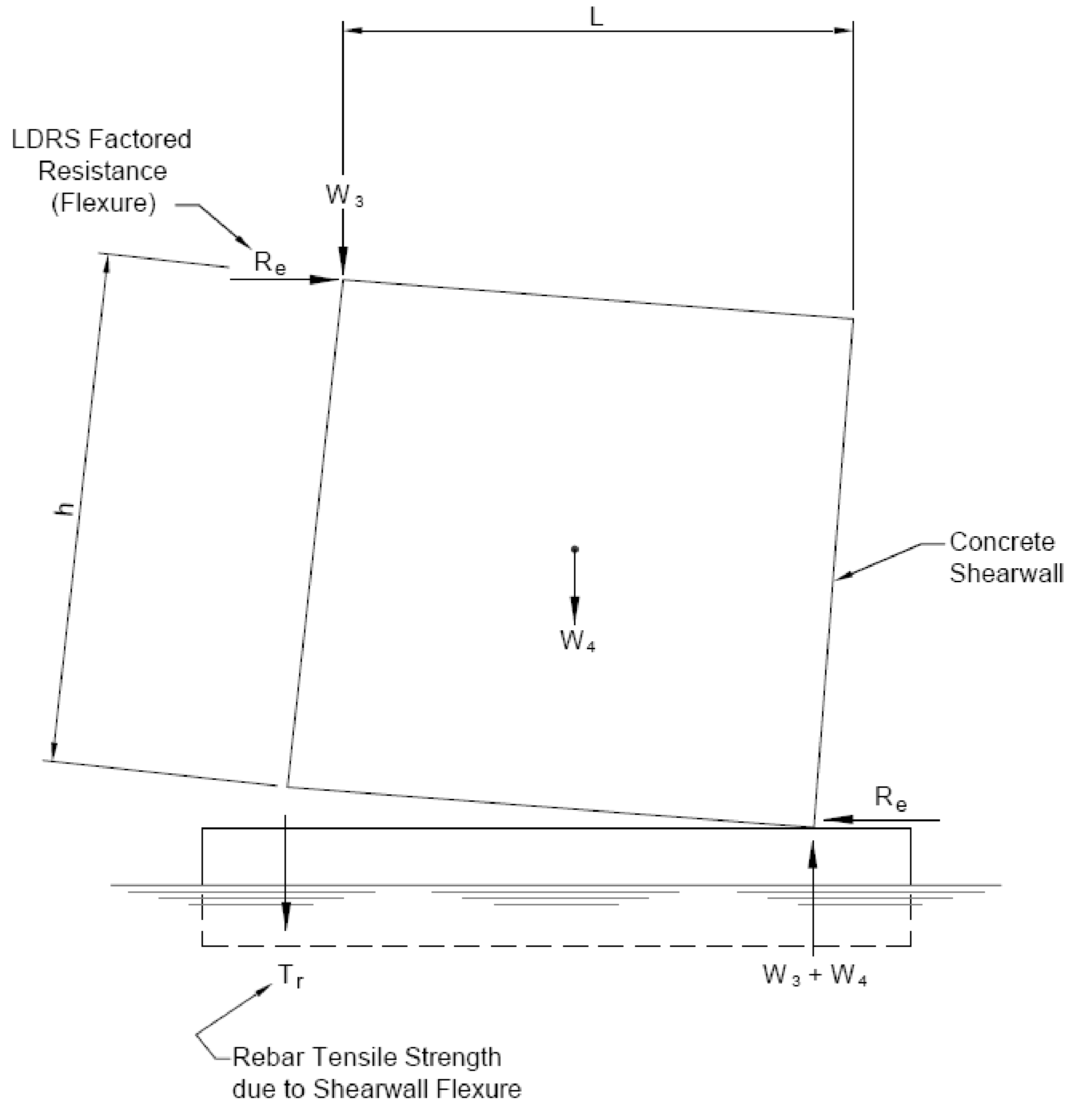
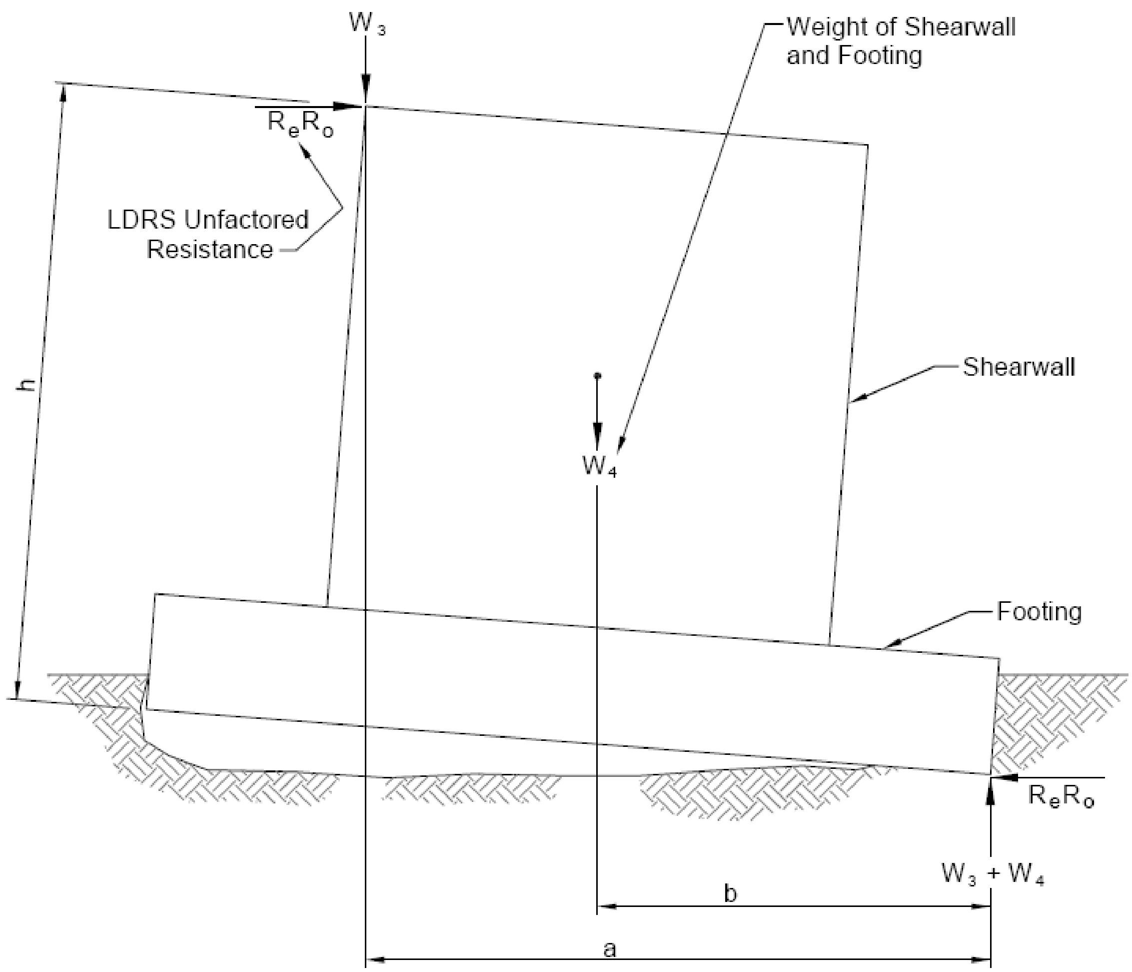
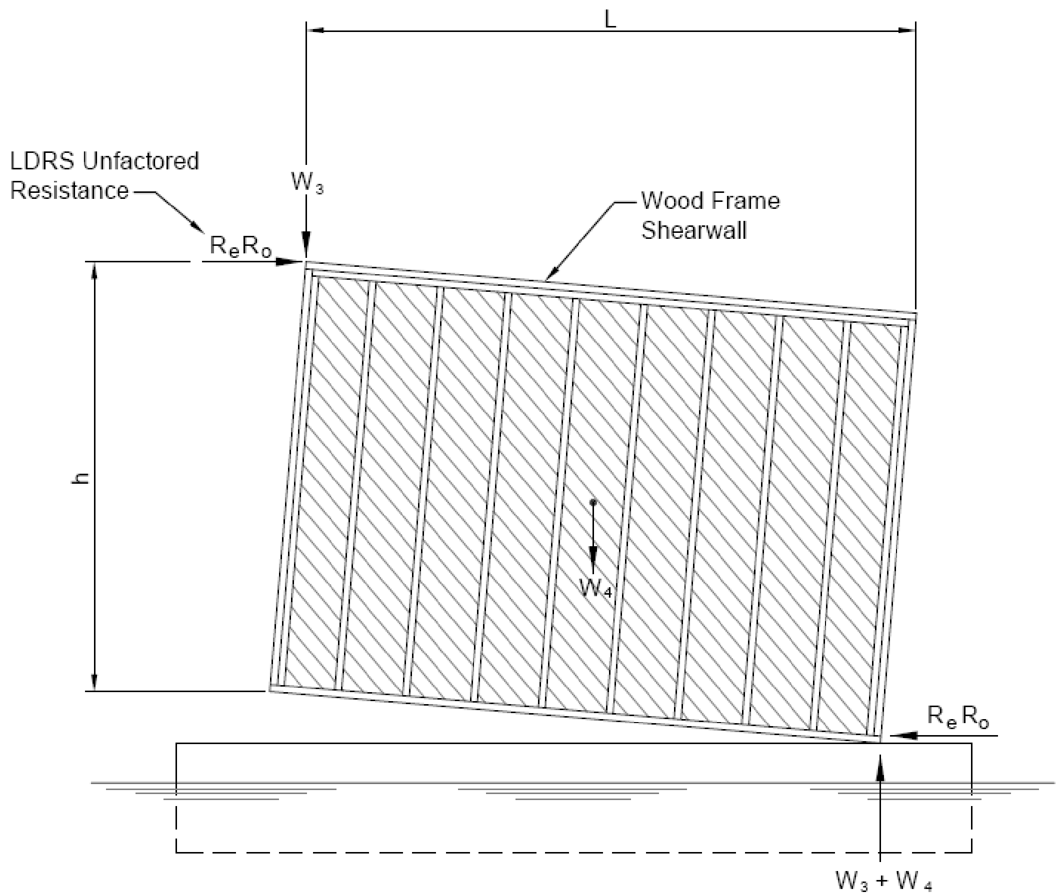


Figure A.8-2 Combined Flexure and Rocking of Concrete Shearwalls



$$R_e R_o\, h > W_3 a + W_4 b$$

Figure A.8-3 Rocking Footing



$$R_e R_o h > L(W_3 + 0.5W_4)$$

Figure A.8-4 Rocking Wood Frame Shearwall with No Holdowns

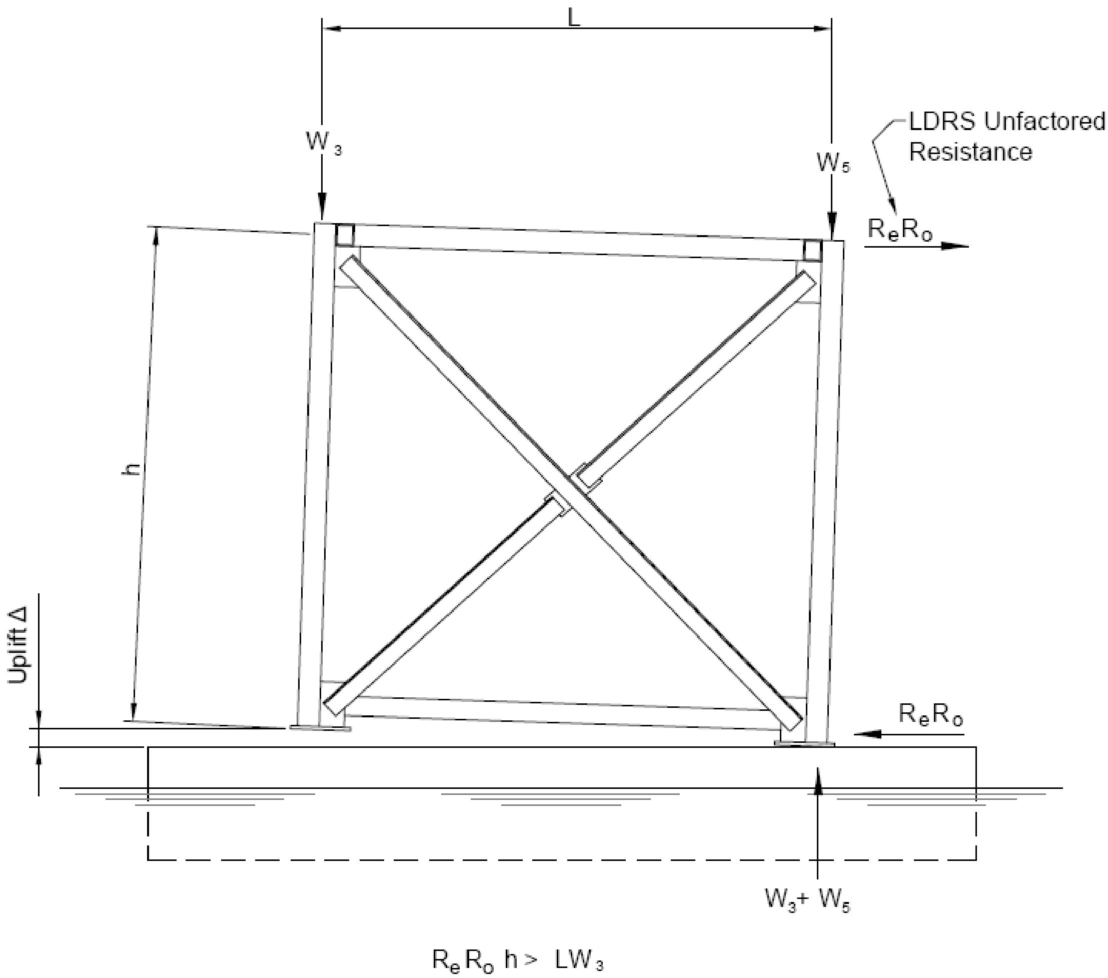


Figure A.8-5 Rocking Steel Frame

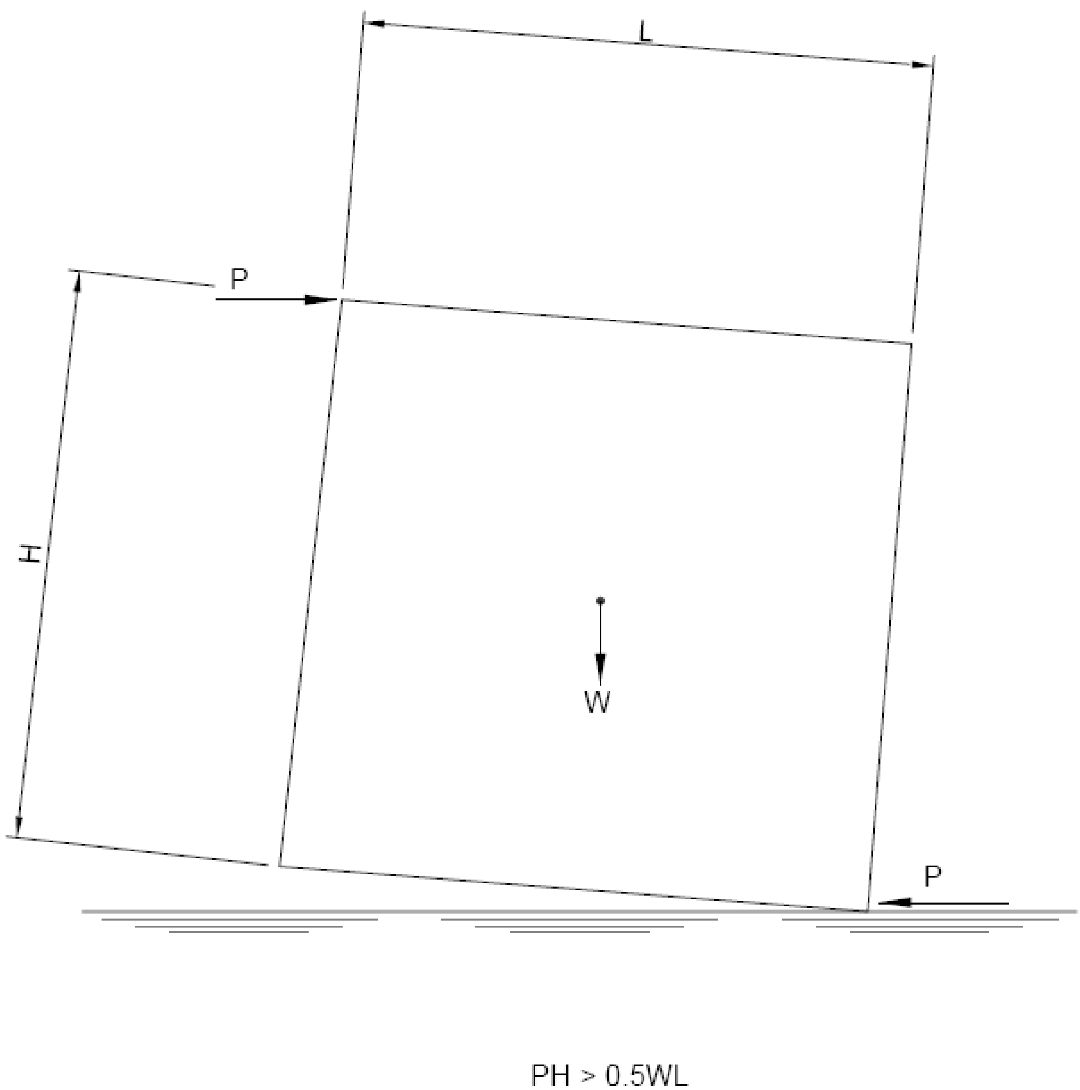


Figure A.8-6 Generic Rocking

A9.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR HEAVY PARTITION WALLS IN LOW-RISE SCHOOL BUILDINGS

A9.1 Scope of Heavy Partition Walls Guidelines

1) As stated in Sentence 1.1(7) and Sentence 1.1(9), the objective of the Bridging Guidelines is the protection of life safety through the reduction in the probability of structural collapse. Heavy partition walls that are not part of the structural system also pose a potentially high risk to the life safety of the building occupants. For this reason, the non-structural hazard abatement of heavy partition walls is included in the scope of the Bridging Guidelines.

   The common forms of heavy partition walls are listed in Sentence 9.1(1). This list does not necessarily include all possible forms of heavy partition wall construction. The guidelines provided in this section should be used with discretion in application to other types of heavy partition walls that specifically listed in this section.

2) All heavy partition walls must be removed or retrofitted to reduce the life safety risk to an acceptable level.

A9.2 In-plane Requirements

1) Non load-bearing heavy partition walls can only mobilize their limited self weight to generate in-plane lateral resistance. Therefore, in-plane lateral resistance of unreinforced heavy partition walls constructed of clay tile, pumice block and glass block is to be excluded in both the assessment and retrofit phases of a seismic upgrade.

A9.3 Out-of-plane Requirements

1) Clay tile, pumice block and glass rock are often expensive to retrofit. In such cases, it is preferable to remove and replace these types of heavy partition walls.

2) Clay brick heavy partition walls are to conform to the out-of-plane requirements of Section 7.4.

3) Unreinfored concrete masonry partition walls are to conform to the out-of-plane requirements of Section 6.5. Unreinforced concrete masonry in-fill walls are to conform to the requirements of Section 6.6.

A10.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR DIAPHRAGMS IN LOW-RISE SCHOOL BUILDINGS

A10.1 Prototypes

1) There are a total of six different diaphragm prototypes that are primarily distinguished by the type of material that generates the diaphragm's lateral resistance (e.g. blocked plywood sheathing).

   A range of wall construction types was considered in determining the governing diaphragm requirements for each diaphragm prototype. Reinforced concrete masonry determined the minimum required factored resistance for most diaphragm prototypes.

   The wood diaphragm prototypes include both blocked and unblocked construction. For all wood diaphragms, no distinction is made between plywood and oriented strandboard (OSB) in the material strength calculations.

   A typical steel roof deck diaphragm is illustrated in Figure A.10-1. References for research on steel deck diaphragms are given in Part D of the commentary. The papers by Tremblay et al. provide the latest research on the ductility of the different forms of steel deck construction. The Bridging Guidelines include two common types of steel roof deck diaphragms.

   Type A steel deck diaphragms have the best ductility and are the recommended diaphragm type for retrofit construction. The screwed side laps and the nailed deck fastening to the frame supports (joists) exhibit good ductility relative to other types of steel deck diaphragms. Type B steel deck diaphragms are the typical form of construction for the majority of existing buildings. The button punch side laps and the welded diaphragm/frame connections typical of Type B steel decks exhibit poor ductility relative to the Type A steel deck.

   Prototype D-5 comprises a horizontal concentrically braced steel frame with tension bracing only (horizontal version of steel Prototype S-1). Prototype D-6 is similar to Prototype D-5 but has tension/compression bracing (horizontal version of steel Prototype S-2).

   In the analysis of the six diaphragm types, the maximum diaphragm inelastic strain limit for each diaphragm type was assumed as given in Table A10.1.

3) The second edition does not provide any detailed guidelines for the assessment and retrofit design of concrete diaphragms. The assessment and retrofit design of concrete diaphragms are to be performed in accordance with the building code and best current practice.

A10.2 Notations

For clarification, the definition of W_d is illustrated in Figure A.10-2.

A10.3 Minimum Required Lateral Factored Resistance R_m

1) For all materials, the minimum required factored resistance values for risk assessment are set at 80% of the corresponding retrofit values as detailed in Section C2.3.

   For the given seismic zone and soil type, the minimum required factored resistance for a retrofit design is read from the tables immediately below Figures 10-1 to Figure 10-6 for the selected value of the Governing Drift Limit. As noted above, the assessment value is 80% of the corresponding retrofit value.

   Refer to Section C5.7 for detailed technical background information on the analysis of diaphragm behaviour.

A10.4 Calculation of Lateral Resistance

1) The lateral resistances of the six types of diaphragms are to be calculated in accordance with the material standards given in Section 10.4.

A10.5 Chord Force and Strength

1) The chord force for a wood or horizontal braced steel frame diaphragm is calculated on the assumption that inelastic strains generate the maximum lateral shear resistance in the diaphragm for the majority of its length. This assumption is validated by analysis results. A short mid-span portion of wood and braced steel diaphragms is usually subject to elastic strains. Refer to Figure A.10-3 for an illustration of this chord force calculation.

2) The chord force calculation for steel deck diaphragms assumes a linear variation in lateral shear ranging from the maximum diaphragm strength close to the end wall supports to zero lateral shear at mid-span (refer to Figure A.10-3).

A10.6 Diaphragm Distribution of Inertia Mass

1) Typical redistribution of inertia mass for a concrete diaphragm is illustrated in Figure A.10-4. For a rigid diaphragm, a plane of LDRSs with low lateral resistance is compensated for by the resulting torque effectively redistributing the inertia mass in the immediate vicinity of the low lateral resistance LDRSs.

   Concrete diaphragms are classified as rigid diaphragms. Wood, horizontal braced steel frame and steel deck diaphragms are classified in the non-rigid category.

2) There are limits to the ability of rigid diaphragms to redistribute inertia mass. The limits are expressed in terms of the maximum plan eccentricities as illustrated in Figure A.10-5. These eccentricity limits are LDRS dependent. Stiff LDRSs such as concrete and reinforced masonry shearwalls provide a better inertia mass redistribution than less stiff LDRSs such as wood frame or tension-only braced steel frames.

3) If a building with concrete diaphragms exceeds the eccentricity limits given in Sentence 10.6(2), the building LDRSs need to be upgraded to reduce the plan eccentricities to comply with the maximum limits.

4) Inertia mass is distributed by tributary area for non-rigid diaphragms. The one exception is the clustering of LDRSs within 5m of each other. For such a LDRS cluster, the inertia mass can be effectively redistributed between LDRSs in the cluster. Refer to Figure A.10-6 for clarification.

A10.7 Steel Deck Diaphragms

1) Type B steel deck is not recommended as new material in a Seismic Zone 3-6 seismic upgrade because of its relatively poor ductility (poor ductility relative to ductility of Type A steel deck). However, existing Type B steel deck can be retained, in whole or in part, as part of the upgraded building if it meets the minimum strength requirements.

A10.8 Wood Diaphragms in Wood Frame Buildings

1) With rare exceptions, wood diaphragms do not initiate collapse of wood frame buildings. Therefore, the Bridging Guidelines specify prescriptive requirements that permit the engineer to quickly assess the adequacy of wood diaphragms.

2) The exception to the life safety statement in Section A10.8(1) is the common use of tongue-and-groove (T&G) decking in existing diaphragms. Thick, well-spiked T&G decks are acceptable. Other T&G decks need to be upgraded. The one accommodation for such decks is the timing of the T&G deck replacement which can be delayed to coordinate with the replacement of the roof membrane in the maintenance cycle. Shiplap perpendicular to its supporting joists also needs to be upgraded.

A10.9 Inelastic Strain Limitation

1) Dissimilar LDRSs can generate potentially large inelastic strains in flexible diaphragms. Therefore, the maximum differential drift between LDRSs relative to their spacing needs to be restricted to ensure the overall performance of the diaphragm is not compromised. In checking this differential drift, each LDRS should be assumed to be deformed to its maximum permissible drift limit (ISDL). This check needs to be performed to wood, steel deck and horizontal steel frame diaphragms (flexible diaphragms).

Table A.10-1 Maximum Diaphragm Inelastic Strain Limits

Prototype	Inelastic Strain Limit
D-1, D-2	2.5%
D-3	1.0%
D-4	0.5%
D-5	3.0%
D-6	2.0%

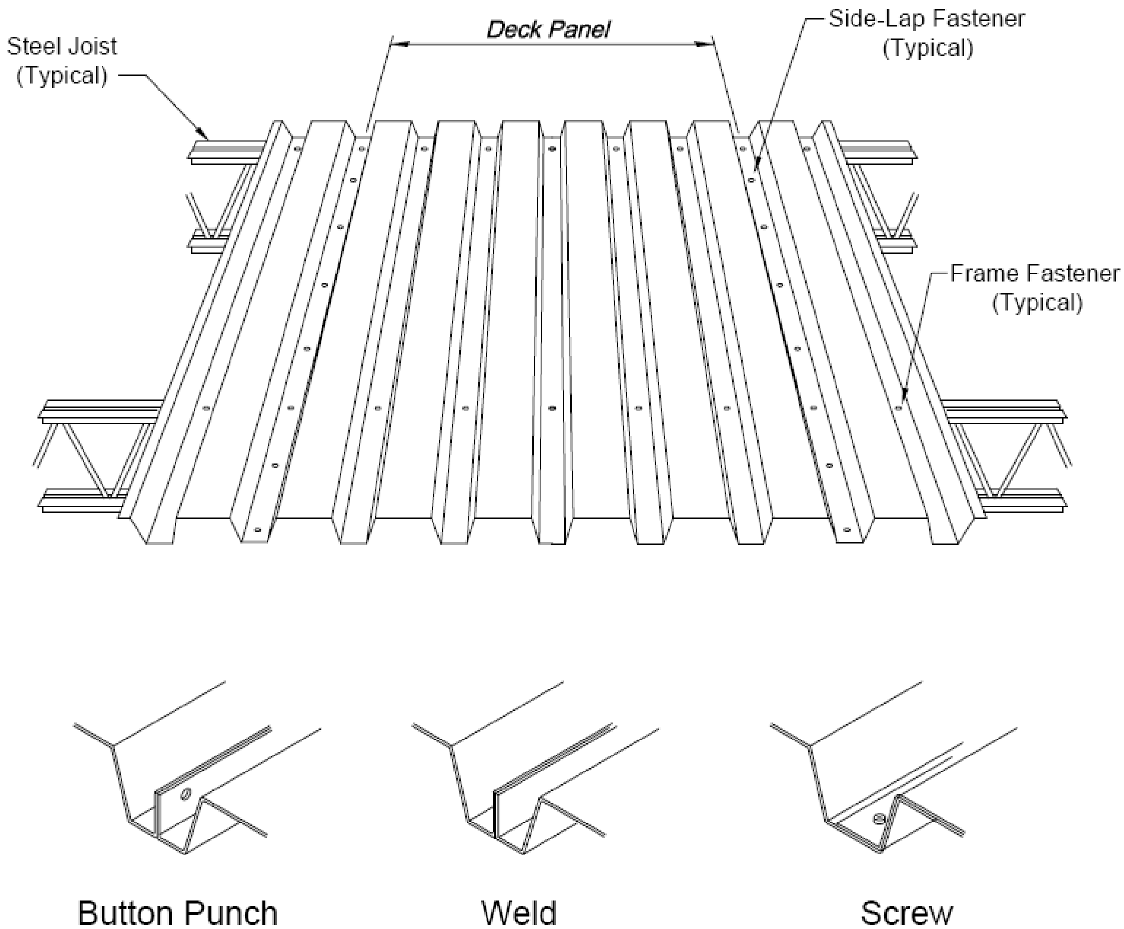


Figure A.10-1 Typical Steel Deck Diaphragm

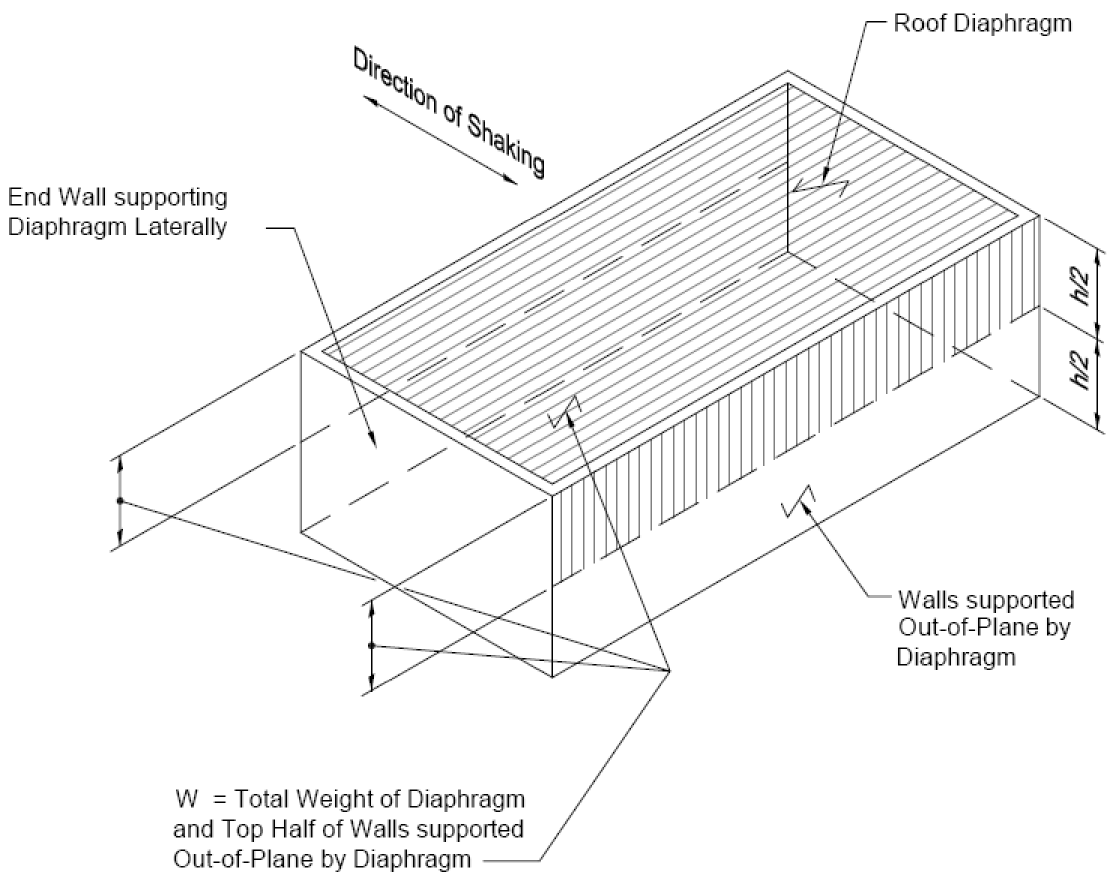


Figure A.10-2 Definition of W_d

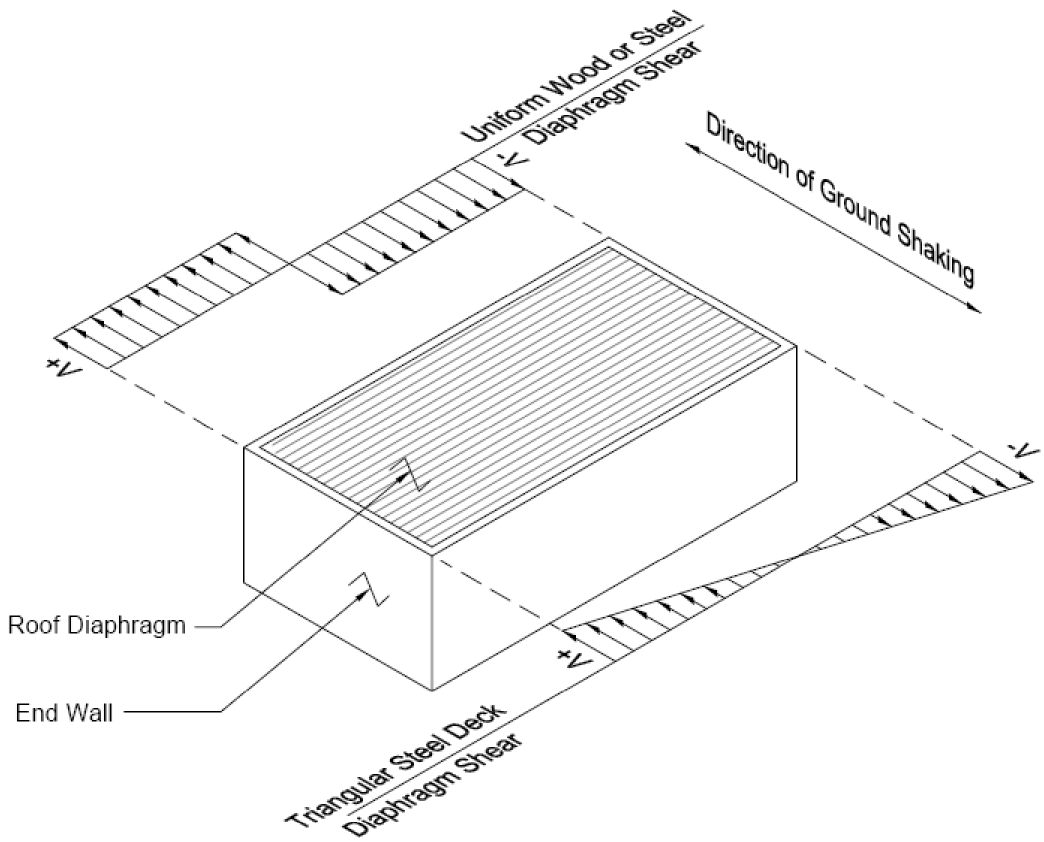


Figure A.10-3 Diaphragm Shear Distribution Assumed for Calculation of Chord Axial Forces

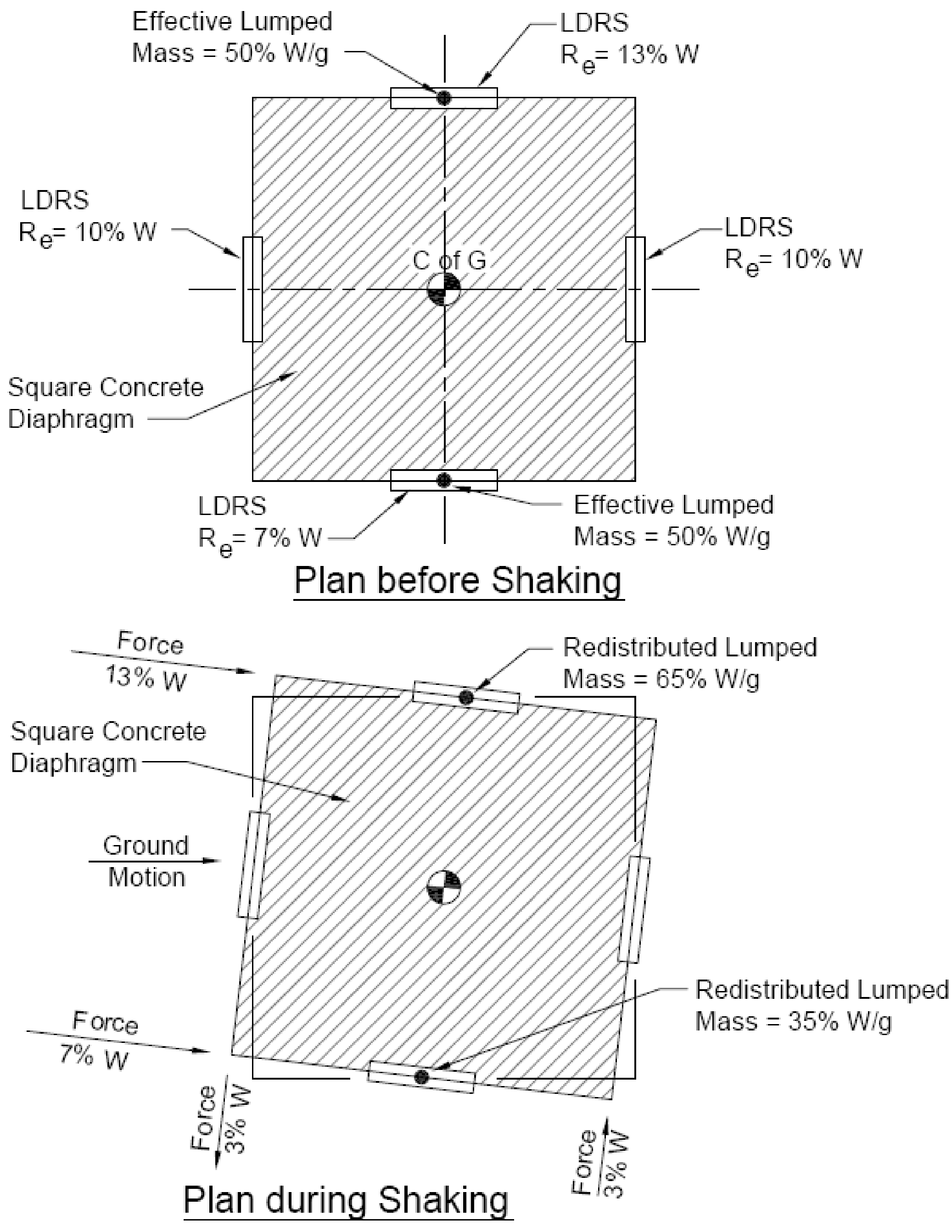


Figure A.10-4 Concrete Diaphragm Inertia Mass Redistribution

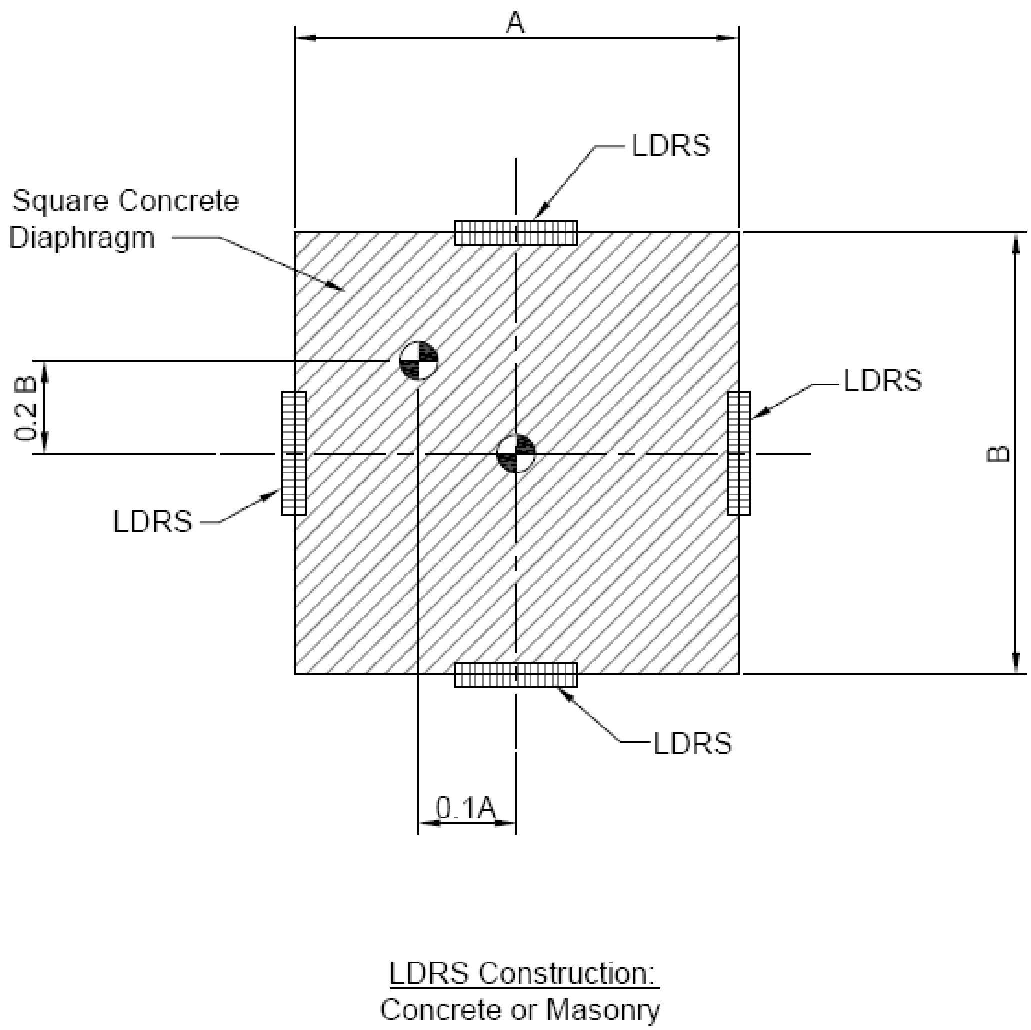


Figure A.10-5 Definition of Maximum Plan Eccentricities

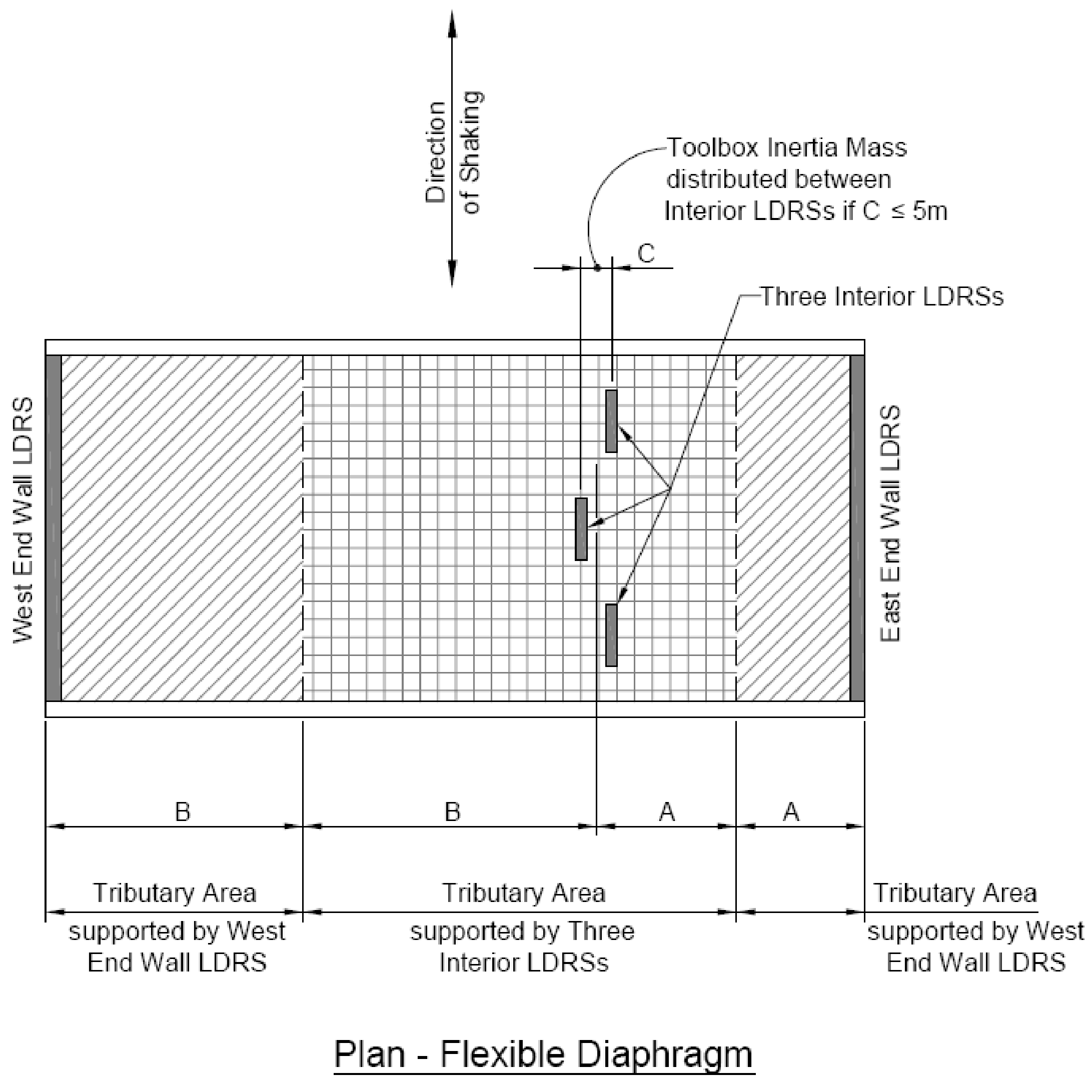


Figure A.10-6 Flexible Diaphragm Inertial Mass Distribution between LDRSs

A11.0 PERFORMANCE-BASED EARTHQUAKE RETROFIT GUIDELINES FOR CONNECTIONS IN LOW-RISE SCHOOL BUILDINGS

A11.1 Introduction

1) All connections have been classified as belonging to one of three categories that are distinctive on the basis of the type of connected building element.

   Connections play a crucial role in the continuity of the load path as mandated in Sentence 1.6(1)(b).

   In the Bridging Guidelines, the fundamental premise in the design of connections is that building collapse must be initiated by inelastic deformation and failure of the structural elements (and heavy partition walls), not by failure of connections between structural elements.

2) The descriptions of the three categories of connections are self explanatory.

A11.2 Level of Risk

1) For connections, no distinction is made in the minimum level of performance requirement for risk assessment and retrofit design. For both risk assessment and retrofit design, the factored resistance of the connection must equal or exceed the minimum required factored resistance.

A11.3 Minimum Factored Resistance - Diaphragm Connections

1) Refer to Figure A.10-6 for an illustration of the application of Equation (11-1).

   The "1.5" multiplier is to account for the values of R_o given in Table 1.1 being lower bound values. The structural element will undoubtedly be stronger than its factored resistance times R_o. The "1.5" multiplier provides some added assurance that the connection is stronger than its adjoining structural members.

   R_{ed} is the factored strength of the diaphragm along the east or west end wall parallel to the direction of shaking.

2) With reference to Figure A.10-6, n_c is the number of connections along the north or south edge of the diaphragm where the walls are supported out-of-plane by the diaphragm.

For the shear connection of the diaphragm to its end walls, n_c is the number of connectors along the east or west edge of the diaphragm.

3) The second edition does not provide any detailed guidelines for the assessment or retrofit design of connections in concrete diaphragms.

A11.4 Minimum Factored Resistance - Member Connections

1) Refer to the corresponding CSA standard.

A11.5 Minimum Factored Resistance - Attachment Connections

1) Attachment connections are to be designed using the provisions given in the building code.

PART B BACKGROUND INFORMATION

Table of Contents

B1.0 INTRODUCTION

Part B of the commentary provides additional technical background not specifically included in Part A "Explanatory Notes" or Part C "Technical Development of Resistance Tables". It also includes information on material from Part A that was not essential for proper use of the guidelines.

B1.1 Intent of Guidelines

The 2nd Edition Bridging Guidelines are intended to be used in the assessment and retrofit of existing low-rise British Columbia school buildings as part of a voluntary seismic mitigation program. These guidelines are intended to supplement the experience of a qualified professional engineer by providing a new set of seismic demands, different from those provided by the 2005 NBCC. The guidelines do not replace seismic retrofit techniques commonly used in current practice, but merely provide a new set of "earthquake force" levels that meet the seismic performance level specified by the owner, the British Columbia Ministry of Education (MoE).

B1.2 Necessary Qualifications

Engineers using these guidelines for the assessment or retrofit of a BC School building must be Professional Engineers with at least 6 years of practical experience, and must have familiarized themselves with these guidelines. Attending one of the Association of Professional Engineers and Geoscientists of British Columbia's (APEGBC) workshops or office visits qualifies for familiarization.

B1.3 Performance-based Earthquake Engineering

"*Performance-based earthquake engineering (PBEE) implies design, evaluation, and construction of engineered facilities whose performance under common and extreme loads responds to the diverse needs and objectives of owners-users and society*." (Krawinkler, 1999).

The 2nd Edition Bridging Guidelines are founded on a PBEE methodology. In this case the client/owner (MoE) has set the performance objective (see Section B1.4). These guideline have used advanced analysis techniques (see Section C3.0) to analyze the school buildings thereby determining the engineering parameters required to meet the objectives.

The innovation of PBEE, over a prescriptive-code based method, is that it allows structures to be designed to the desired performance levels, which are readily related to damage states. Cost analyses on different damage states allow the owners to make an informed decision on the performance level best suited for their buildings. Using PBEE it is possible to retrofit buildings to provide life-safety at a significantly reduced cost compared to using a full code-based approach.

B1.4 Performance Objectives

PBEE can be used to design or retrofit structures to a wide range of performance objectives, quite often in excess of code requirements. Sensitive structures such as hydro damns or nuclear power plants are designed using PBEE to ensure that they remain operational and pose no threat to the public even in severe earthquakes.

The 2005 National Building Code of Canada has a performance objective of collapse prevention for 2% in 50 year ground motions. This is achieved through a combination of an acceleration design spectra, from which minimum “earthquake forces” are derived, and by limits on maximum interstorey drift, equal to 2.5% for most buildings. Additionally, some forms of construction (such as unreinforced masonry or gypsum wallboard) are not permitted by code and thus do not contribute to the resistance of the structure using code-based calculations. The 2005 NBCC is an excellent document for the design of new buildings. It provides reliably and consistent designs for protecting the public and infrastructure. However, enforcing these requirements on existing buildings is onerous and expensive.

The 2nd Edition Bridging Guidelines have a performance objective of life-safety through enhanced collapse prevention, for 2% in 50-year ground motions. This objective is achieved by setting a drift limit (i.e. estimate of collapse) appropriate for the structural system (see Section B3.1), but using a higher demand from the ground motions (see Section C2.1). The higher demands result in an extra level of safety making the performance objective enhanced collapse prevention, instead of merely collapse prevention. Life safety is achieved by preventing the collapse or partial collapse of the structure or heavy building components, which is accomplished with enhanced collapse prevention.

The 2nd Edition Bridging Guidelines provide a means for PBEE to be applied to wide range of structures. By using less conservative drift limits and making use of almost all existing materials, the 2nd Edition Bridging Guidelines are able to achieve a life-safety performance objective, but significantly reduce construction costs compared to using the 2005 NBCC for performing seismic retrofits on school buildings.

B2.0 SEISMICITY

The 2nd Edition Bridging Guidelines use the 2005 NBCC specified seismic hazard data to set the level for the ground motion inputs into the analysis methods used in the generation of the resistance tables. Regional seismicity data was taken from Appendix C Table C2 of the 2005 NBCC. This data was used in conjunction with Sentence 4.1.8.4.(4) of the 2005 NBCC to develop site class specific design spectra for four different seismic zones (see Section B2.1). These spectra are shown in Section C4.

B2.1 Seismic Zones

Section A1.2 describes the procedure for determining which seismic zone a site is located in. A seismic hazard map is included on the 2nd Edition Bridging Guidelines CD, which shows the division between the seismic zones in BC. Figure B.2-1 shows the South-Western British Columbia, which is a small portion of the map.

Seismic Zones 2 through 5 are covered by the 2nd Edition Bridging Guidelines. The ground motion suite (see Section C2) used to simulate the seismic hazard in these areas are all crustal type earthquakes.

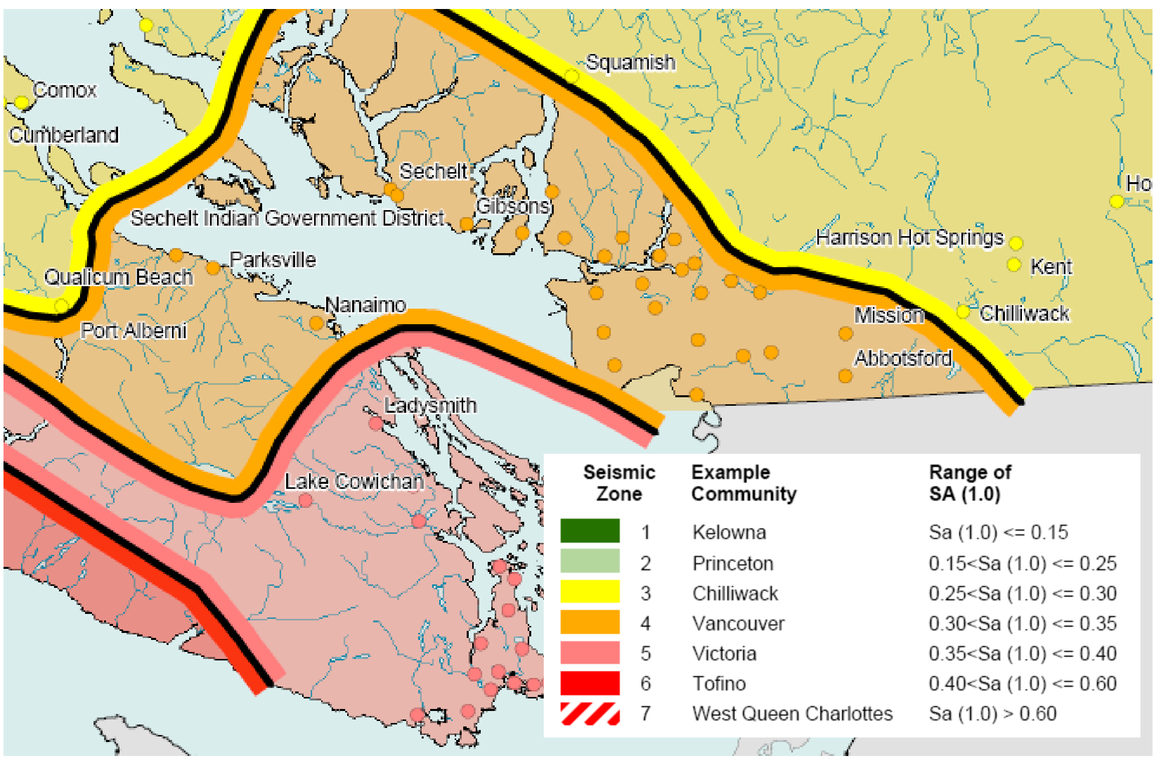


Figure B.2-1 Seismic Hazard Map for South-western British Columbia

Seismic Zone 1 is considered a region of low seismicity, and all schools in these regions are considered to have a low seismic risk.

Seismic Zones 6 and 7 are regions of high seismicity, and can be subjected to subduction type ground motions. The ground motions used in the 2nd Edition Bridging Guidelines do not account for these types of ground motions and therefore these Zones 6 and 7 are not covered. Future editions of these guidelines will address this important issue.

The seismic zone of some municipalities may be difficult to determine from the Seismic Hazard Map. Table B.2-1 lists all the seismic zones for all municipalities in British Columbia.

B2.2 Site Class

The soil type (site class) that the schools are located on are needed to determine the minimum required factored strength from the resistance tables in Sections 3 through 8. The 2nd Edition Bridging Guidelines uses the same site classifications as the 2005 NBCC, found in Section 4.1.8.4.

The appropriate site properties must be determined for a detailed assessment or retrofit. For preliminary or concept analysis a pair of Soil Hazard Maps are available for use on the 2nd Edition Bridging Guidelines CD. The maps include site class information on the Lower Mainland of British Columbia and Victoria. Figure B.2-2 shows a small sample of these maps.

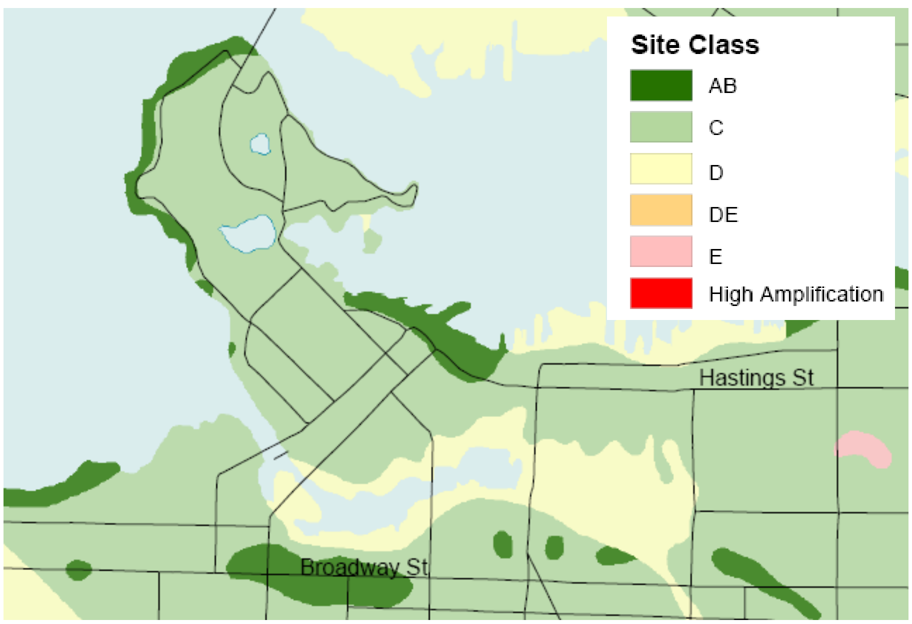


Figure B.2-2 Soil Hazard Map for Downtown Vancouver

Table B.2-1 Seismic Zones by Municipality for British Columbia

Municipality	Seismic Zone
100 Mile House	1
Abbotsford	4
Alert Bay	2
Anmore	4
Armstrong	1
Ashcroft	2
Belcarra	4
Bowen Island	4
Burnaby	4
Burns Lake	1
Cache Creek	2
Campbell River	3
Castlegar	1
Central Saanich	5
Chase	1
Chetwyn	1
Chilliwack	3
Clinton	2
Coldstream	1
Colwood	5
Comox	3
Coquitlam	4
Courtenay	3
Cranbrook	1
Creston	1
Cumberland	3
Dawson Creek	1
Delta	4
Duncan	5
Elkford	1
Enderby	1
Esquimalt	5
Fernie	1
Fort Nelson	1
Fort St, James	1
Fort St. John	1
Fraser Lake	1
Fruitvale	1
Gibsons	4
Gold River	4
Golden	1
Grand Forks	1
Granisle	1
Greenwood	1
Harrison Hot Springs	3

Table B.2-1 (continued) Seismic Zones by Municipality for British Columbia

Municipality	Seismic Zone
Hazelton	1
Highlands	5
Hope	3
Houston	1
Hudson's Hope	1
Invermere	1
Kamloops	1
Kaslo	1
Kelowna	1
Kent	3
Keremeos	1
Kimberley	1
Kitimat	1
Ladysmith	5
Lake Country	1
Lake Cowichan	5
Langford	5
Langley City	4
Langley District	4
Lillooet	3
Lions Bay	4
Logan Lake	2
Lumby	1
Lytton	3
Mackenzie	1
Maple Ridge	4
Masset	4
McBride	1
Merritt	2
Metchosin	5
Midway	1
Mission	4
Montrose	1
Nakusp	1
Nanaimo	4
Nelson	1
New Denver	1
New Hazelton	1
New Westminster	4
North Cowichan	5
North Saanich	5
North Vancouver City	4
North Vancouver District	4
Oak Bay	5
Oliver	1

Table B.2-1 (continued) Seismic Zones by Municipality for British Columbia

Municipality	Seismic Zone
Osoyoos	1
Parksville	4
Peachland	1
Pemberton	3
Penticton	1
Pitt Meadows	4
Port Alberni	3
Port Alice	2
Port Clements	5
Port Coquitlam	4
Port Edward	2
Port Hardy	2
Port McNeill	2
Port Moody	4
Pouce Coupe	1
Powell River	3
Prince George	1
Prince Rupert	2
Princeton	2
Qualicum Beach	4
Quesnel	1
Radium Hot Springs	1
Revelstoke	1
Richmond	4
Rossland	1
Saanich	5
Salmo	1
Salmon Arm	1
Sayward	2
Sechalt Ind Gov District	4
Sechelt	4
Sicamous	1
Sidney	5
Silverton	1
Slocan	1
Smithers	1
Sooke	5
Spallumcheen	1
Sparwood	1
Squmish	3
Stewart	1
Summerland	1
Surrey	4
Taylor	1
Telkwa	1

Table B.2-1 (continued) Seismic Zones by Municipality for British Columbia

Municipality	Seismic Zone
Terrace	1
Thasis	5
Tofino	6
Trail	1
Tumbler Ridge	1
Ucluelet	6
Valemount	1
Vancouver	4
Vanderhoof	1
Vernon	1
Victoria	5
View Royal	5
Warfield	1
Wells	1
West Vancouver	4
Whistler	3
White Rock	4
Williams Lake	1
Zeballos	5

B3.0 LATERAL DEFORMATION RESISTING SYSTEMS

Lateral Deformation Resisting Systems (LDRS) are the structural components that resist earthquake motion. The 2nd Edition Bridging Guidelines covers 17 different structural systems (LDRSs) found commonly in schools as either existing systems or systems that are used for retrofits. These seismic demands on these systems have been determined by performing non-linear dynamic analysis (NLDA) on representative prototype models (see Commentary Part C).

These prototype models are idealized and may not apply to all existing or new systems. Sections 3 through 8, and A3 through A8, list criteria which must be met for the real systems to be represented by the LDRS. If the real system cannot be match to and LDRS the options are to either remove the system, or to use the 2005 NBCC (see Sentence 1.1(6)). However if the 2005 NBCC is used, it must be used for the entire building, which may result in a more costly retrofit than using the 2nd Edition Bridging Guidelines.

B3.1 Instability Drift Limits

Commentary C examines the details of the analysis methods used to generate the resistance tables found in Sections 3 through 8. A detailed description of the models is given including a discussion into some of the properties.

One of the most important LDRS parameters, the Instability Drift Limit (ISDL) (see Section 1.3 for a definition) was not incorporated into the prototype models. The point of failure was not incorporated into the numerical models because:

- The experimental tests from which the models were developed either were terminated before collapse or indicate collapse at extremely high drifts
- Capturing the instant of collapse during a NLDA is onerous and inconsistent depending on the numerical stability of the model

In general NLDA methods work very well up to the point of collapse, but do not give particularly meaningful results at the point of collapse. When and where collapse occurs is better left to engineering judgment. As such, the models were analyzed for a range of drift levels, and only the drift levels equal to or below the ISDL were presented in the resistance tables.

The ISDLs were based on non-linear acceptance criteria presented in FEMA 356 (ASCE, 2000). Specifically the Tables 5-6, 5-7, 6-8, 6-18, 6-19, 7-4, 7-7, 8-4, were used for the steel, concrete, masonry and wood-frame prototypes. The secondary collapse prevention (CP) values were considered the closest match to the definition of the ISDL. While FEMA-356 suggests that LDRS should be designed with lower limits (i.e. the primary collapse prevention), it was felt that the added factor of safety on the ground motions (see Section C4) allowed for the more realistic estimate of the actual collapse limit.

The ISDL for the concrete frame prototypes (C-3, C-4 and C-5) are larger than those listed on table 6-8 of FEMA-356. However, the 2nd Edition Bridging Guidelines restricts the drift for concrete columns based on Equation (A.5-1), which often reduces the ISDL to levels lower than found in FEMA-356.

Rocking prototypes have a large ISDL of 4%. However, rocking is seldom the only LDRS in a system, and as such the GDL will be limited by the ISDLs of the other prototypes.

The drift limits for eccentric braced frames (EBF) (Prototype S-3) and steel moment frames (prototype S-4) are only 4%, which is considerably lower than what can be found in FEMA-356. These have been limited to 4% for this edition of the guidelines because it was felt that drift in excess of 4% could result in a life-safety issue from plate glass windows that were not properly protected with a seismic safety film.

B3.2 Toolbox Method

One of the most significant innovations of the Bridging Guidelines is the "Toolbox Method", which is a rational method by which the contribution of different structural systems can be combined. Section 2 describes the procedure to use the Toolbox method and Section A2 offers additional guidance.

Existing school buildings often have two or more structural systems resisting lateral motion. Unlike newer buildings which are designed with compatible systems, these older structural systems may have very different properties; it can be difficult to properly assess the combined behaviour of incompatible systems using code-based methods.

For LDRSs to be combined using the toolbox method, they must be assessed/retrofitted to a common maximum drift limit, called the Governing Drift Limit (GDL). The GDL is usually equal to the lowest Instability Drift Limit (ISDL) of the LDRS being combined. The contribution of each LDRS in the toolbox method is based on their individual response (i.e. if the LDRS was the only system in a building) at the GDL. If the sum of the contributions (capacity/demand) is equal to or greater than 1, then the drift of the structure will be limited to the GDL.

The major assumption in the Toolbox Method is that the contribution of the systems designed to the same drift can be simply summed together. The problem with this assumption is that while the systems individually have their drift limited to the GDL, they each had a unique time-history response to the ground motion suite. Therefore the instant at which one LDRS reached the GDL is not the same instant in which another did.

To verify the validity of the Toolbox assumption, two independent investigations were undertaken by observing the response of multiple LDRSs analyzed together and compared to the demands using the Toolbox Method. Section C6.3 lists the results of the

UBC investigation. The External Peer Review Technical report gives the findings of the independent analysis of the Toolbox Method.

Both investigations concluded that the Toolbox Method is in fact conservative. The combination of the different systems limits drift more than the sum of the individual systems' contributions. One explanation for this conservatism is that different systems may compliment the weaknesses of the others.

B3.3 Strength Distribution

The prototypes for the 2nd Edition Bridging Guidelines were modeled as two storey structures with equal strength on both floors (see Section C5 for more details on the models). The mass was lumped at the top of the first floor and on the roof. The mass of the roof was 80% of that for the second floor. This strength and mass distribution resulted in the vast majority of the inelastic deformations occurring in the first floor.

This particular configuration results in the greatest drift demands for a given strength of an LDRS. Independent checks by the EPR (see EPR report) and UBC show that both a different number of stories (1 or 3) and different strength distributions results in lower drift demands. This why it was possible to use a single two-storey model to represent all low-rise structures.

Different strength distributions (i.e. case where the upper stories have less capacity than the first storey) result in a better distribution of inelastic drift between the stories, there by reducing the maximum inelastic drift in the building. The strength limits for stories above the first, given in Sentences 1.8(3) and 1.8(4), are there to ensure that the inelastic deformation in the upper stories do not govern.

B3.4 Vertical Force Distribution

The assumed vertical force distribution is defined by Equation (1-1) under Section 1.10. The primary purpose of this equation is for the calculation of the overturning moment. This equation is based on the square of the height, and conservatively results in high forces at the roof level.

B3.5 Storey Height

The storey height of the building has a significant impact on its drift demand. A taller storey height indicates more flexibility and less drift for an absolute displacement compared to a shorter storey height. UBC analysis results indicated that increasing the storey height from 3 to 4 metres resulted in reductions strength demand (for a given drift level) of up to 25% for wood-frame prototypes and 15% for other prototypes.

Equations 3-1, 4-1, 5-1, 6-1, and 7-1 can be used to reduce the minimum required lateral factored resistance, obtained from the appropriate tables, for LDRSs with storey heights greater than 3 metres but no more than 4 metres. These equations were introduced instead of supplying an additional set of tables for 4 metre heights. The demand reduction is limited to 4 metre heights because that was what was analyzed and also because typical school (non-gymnasium) buildings do not have taller storey heights. The equations give a linear reduction in strength demand inversely proportional to the height of the storey. There is no reduction at 3 metres and a 25% to 15% reduction at 4 metres depending on the prototype.

Section B10.2 discusses the influence of storey height, and thus centre of gravity for rocking elements.

B4.0 GUIDELINES FOR SITE RESPONSE ANALYSIS

Sentence 1.15(2) recommends a site response analysis for building locations underlain by all Site Class E and F soils. This section provides the guidelines for the above site response analysis by qualified geotechnical engineers.

B4.1 Confirmation of Site Class

A geotechnical engineer or professional geologist member of the school seismic retrofit project team confirms that the building is founded on Site Class E or F soils as defined in the Table 4.1.8.4A of the 2005 NBCC.

B4.2 Design Ground Motions

UBC will provide the geotechnical engineer with a suite of time histories, and scaling factors, to be used as outcrop motions for the site response analysis. This suite of ground motions are the same used in developing the resistance tables (see Section C4) and are recordings of crustal type ground motions on Site Class C soil. There are 10 records in the suite.

B4.3 Geotechnical Analysis

The site response analysis shall be conducted using methods of analysis that have appeared in refereed publications and have a track record of use in local geotechnical practice. Equivalent linear analysis is considered to have reduced reliability for ground motions in excess of 0.4g in softer soils or shear strain amplitudes exceeding 1%-2%. In such cases true nonlinear analyses are preferred.

Examples of two suitable programs that are commonly used in BC practice are the equivalent linear program SHAKE (Idriss and Sun, 1995) and the nonlinear program DESRA (Lee and Finn, 1978).

The depth at which the outcrop motion is input into the analysis depends on the stratigraphy of the site. The motion should be input where the site properties match the conditions of Site Class C. The outcrop motion should be allowed to propagate both upwards and downwards.

The analysis for each ground motion is to be performed for a reasonably probable upper and lower bound of soil properties based on the potential for ground motion amplification.

B4.4 Presentation of Site Response Analysis Results

The geotechnical engineers send the results of the site response analysis to UBC. The results are to be presented in the form of acceleration time histories for surface lateral shaking for the upper and lower bound soil properties and for the suite of ground motions.

B4.5 Building Data

The structural engineer delivers a summary of the relevant building data to UBC to assist UBC in its analysis of the building for the surface ground motions generated from the site response analysis. The building data to be delivered to UBC includes the following:

(1) Seismic Zone.
(2) List of existing and new LDRSs prototypes.
(3) List of existing and new diaphragm prototypes and their span lengths.

B4.6 UBC Analysis

Using the two new suites of ground motions from site response analysis results, UBC generates new tables of minimum required factored resistances for the specified LDRSs and diaphragms. These tables are forwarded to the structural engineer to use in the assessment or retrofit of the school building located on the site. These tables are used in lieu of the resistance tables found in Sections 3-8 for the site under investigation only. The tables are based on the highest demand from the upper and lower bound ground motion suites.

B4.7 Liquefaction

Only the influence of lateral ground shaking is accounted for in the generation of the resistance tables. Liquefaction is not addressed by the 2nd Edition Bridging Guidelines (see Section 1.16).

B4.8 Database of Site Response Analyses

UBC reserves the right to use the findings of the site response analyses for future research.

B5.0 WOOD FRAME BUILDINGS

The provisions for wood frame buildings are covered in Sections 3 and A3. This section discusses the reasons behind some of the provisions.

B5.1 Limits on Gypsum Wallboard and Shiplap

Sentences 3.3(2) and 3.3(3) limit the contribution of GWB and horizontal boards to 50% of the total strength of the school building, in a given direction. Section A3.3 offers additional guidance. Additional reasons for these limitations are:

- The experimental tests from which the prototype models (Section C5.3) were developed did not examine the response of shearwalls constructed solely of GWB or horizontal boards, but with GWB and horizontal boards in combination with other systems.
- The strengths of GWB and horizontal shiplap listed in Table 3.1 are based on well constructed laboratory specimens. Existing GWB and horizontal boards may have significantly different capacities. There is more uncertainty in the strengths of GWB and horizontal boards than the other wood frame materials.
- In general, GWB and horizontal boards are less suitable for seismic retrofits than OSB/plywood shearwalls because of their lesser ductility.

B5.2 Rocking Prototypes

Wood frame shearwalls governed by rocking use prototypes R-2 and R-3 (for aspect ratios (H/L) of 2.5 or more. These prototypes (R-2 and R-3) are used because the yield drift of these prototypes matches more closely to the yield drift of the wood frame prototypes (W-1 and W-2). The R-1 rocking prototype typically used for low aspect ratio walls is too stiff (i.e. low yield drift) to be used with wood frame shearwalls.

B6.0 STEEL BUILDINGS

The provisions for steel buildings are covered in Sections 4 and A4. This section discusses the reasons behind some of the provisions.

B6.1 Connections for Steel Braced Frames

The steel braced frame prototypes (S-1 and S-2) are representative of systems that follow capacity design principles, which is to say that the brace elements are the inelastic fuse in the system. The connections are assumed to be significantly stronger than the elements, as should be any real LDRS that uses the strength demands of these prototypes.

Some existing steel braced frame systems may not be able to develop the full yielded, and strain hardened, due to premature failure of the connections. These systems should not be used for assessment or retrofit using the prototypes in the 2nd Edition Bridging Guidelines. Such systems may have significant capacity, but the S-1 and S-2 prototypes represent systems that under go significant inelastic deformations. In LDRSs where the brace members remain elastic, the drifts must remain below the yield drift, which is approximately 0.3%. The 2nd Edition Bridging Guidelines only provide strength demands for systems with maximum drifts between 1% and 4%.

It should be noted that maintaining very small maximum drifts (0.5% or less) is a very onerous requirement for 2% in 50-year design ground motion.

B6.2 Single Tension/Compression Braces

Prototype S-2 assumes that there is equal capacity in both directions. In cases where there are more braces in one direction than the other, the lesser tension capacity is used. In cases where there is no tension capacity in one direction, the prototypes may not be used.

The problem with braces in one direction (or single braces) is that they are not symmetrical. They have resistance in the tension direction, but almost no strength in the compression direction, because of the reduction in strength after the compression brace buckles. This leads to very large drifts in one direction.

These types of braces may be acceptable for wind loading, but are not acceptable for seismic design or retrofit.

B6.3 Drift Limits for Hollow Sections in Braced Frames

Table 4.1 lists drift limits for hollow sections used as compression members in prototype S-2. The limits are in the form of d/t ratios (diameter and thickness), which were based on inelastic compression deformation criteria from Table 5-7 of FEMA-356 (ASCE, 2000). The original values were presented in imperial units and have been converted to metric in these guidelines.

These limits are intended to prevent significant local damage in the compression direction from compromising the strength of the brace in tension.

B6.4 Rocking Prototypes

Steel braced frames (including EBFs) governed by rocking use prototypes R-2 and R-3 (for aspect ratios (H/L) of 2.5 or more. These prototypes (R-2 and R-3) are used because response of these prototypes matches more closely to the response of the steel frame prototypes (S-1, S-2 and S-3). The R-1 rocking prototype typically used for low aspect ratio walls is too stiff to be used to represent steel braced frame prototypes.

Rocking moment frames (S-4) of any aspect ratio should use prototype R-3 for rocking.

B7.0 CONCRETE BUILDINGS

The provisions for concrete buildings are covered in Sections 5 and A5. This section discusses the reasons behind some of the provisions.

B7.1 Drift Limits on Columns

Section A5.5 provides an equation (A.5-1) for determining the ISDL for Non-LDRS columns. This equation also applies to the concrete moment frame prototypes, C-3, C-4 and C-5. The ISDL of 4% assigned to the concrete moment frame prototypes is high and intended to mitigate damage from potential P-delta failure or the shattering of large plate glass windows. This ISDL does not protect against premature shear failure in columns, which is a major issue, especially for older concrete buildings. Consequently, all concrete columns should be checked with Equation (A.5-1).

B7.2 Short Columns

Columns with lengths significantly shorter than the storey height, must have their ISDL (calculated from Equation A.5-1) reduced by the ratio of h_s/h_{sc}. If the resulting ISDL is less than 1%, then the short columns are too stiff and brittle to work with the other LDRSs in the building, and must be ignored, removed or retrofitted. Short columns with an ISDL of 1% or more are an acceptable LDRS. Note that if the aspect ratio (H/L) of the short column is 8 or less than it should be treated as a concrete shearwall (prototypes C-1 or C-2). See Section B7.4.

B7.3 Limits on Aspect Ratio

The reinforced concrete prototype models (see Section C5.5) have assumed yield drifts. These yield drifts for shearwalls (0.25%) and moment frames (1.0%) were chosen as it was felt they best represented a “typical” system within a school building. Wall thicknesses, concrete strength and the percentage of reinforcement varies, but is often not significantly different to influence the yield drift.

One factor that does have an impact on yield drift is the aspect ratio of the walls. Figure B7.1 compares the equivalent yield drift versus aspect ratio for a reinforced concrete shear wall both fixed at the base (cantilever) and fixed both top and bottom (pier). The results indicate a yield drift of 0.25% (i.e. what is used in prototypes C-1 and C-2) occurs at aspect ratios of 5 and 10 for cantilevers and piers respectively. However since the analysis (described below) was assuming a fixed base, the aspect ratios limits used in the 2nd Edition Bridging Guidelines were reduced to 4 and 8 (cantilevers and piers) to account for some flexibility between the foundation and soil.

The analysis results presented in Figure B7.1 were calculated with Response-2000 (Bentz, 2000) using the following properties: wall length = 3m, thickness = 200mm, zone steel in each end – (4) 20M, distributed steel – (2) 10M@300mm both directions,

f_c'=30MPa, Fy=500MPa, dead load = 900 kN (5%$A_g f_c$'). The curves were generated by incrementally increasing the displacement at the top of the wall (while maintaining the boundary conditions). The different aspect ratios were achieved by changing the height of the wall.

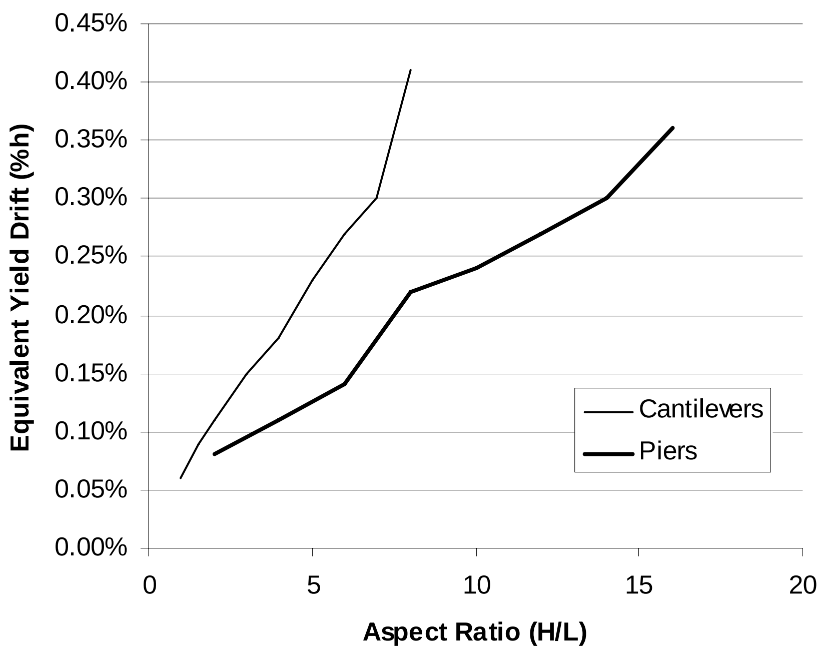


Figure B.7-1 Yield Drift vs. Aspect Ratio for Typical Concrete Shearwalls

B7.4 Rocking Prototypes

Rocking concrete moment frames (prototypes C-3, C-4 and C-5) should use rocking prototype R-3, because their yield drift is most similar to the yield drift for prototype R-3.

B8.0 CONCRETE MASONRY BUILDINGS

The provisions for concrete masonry buildings are covered in Sections 6 and A6. This section discusses the reasons behind some of the provisions.

B8.1 Out-of-plane Requirements

The out-of-plane requirements of concrete masonry walls are given in Section 6.5. Options are given in lieu of using vertical reinforcement. Sentence 6.5(4) gives requirements to achieve vertical confinement of the out-of-plane walls. Confinement is provided by either a surcharge or a stiff upper beam. Both serve to limit the maximum out-of-plane deformations in the wall due to rocking, and to ensure that the wall is properly restrained at the top, thus preventing cantilever rocking. Sentence 6.5(5) gives requirements for external bracing of the URM walls. These requirements are intended to provide the same level of protection as vertical reinforcing bars.

B8.2 Stack Bond

Concrete masonry walls configured in a stack bond formation are not formally recognized in the 2nd Edition Bridging Guidelines. If the removal of such walls is not prudent, the rocking prototypes (R-1 through R-3) could be used as an appropriate LDRS. In this case stack bond walls should be treated as a series of pier rocking elements. The appropriate rocking prototype (R-1 through R-3) should be selected based on the aspect ratio of the stacks (see Sentence 8.1(2)).

B8.3 Aspect Ratio

The aspect ratio of unreinforced concrete masonry walls is not specifically limited in these guidelines. URM walls are either governed by sliding shear failure (prototype M-1) or rocking (prototypes R-1 through R-3). Lower aspect ratio walls will tend to be governed by sliding shear failure, as their overturning resistance will be very high.

Reinforced masonry walls (prototype M-2) behave in the same way as reinforced concrete walls (see Section B7.4).

B9.0 CLAY BRICK MASONRY BUILDINGS

The provisions for clay brick masonry buildings are covered in Sections 7 and A7. This section discusses the reasons behind some of the provisions.

B9.1 Out-of-Plane Requirements

Out-of-plane requirements for unreinforced clay brick masonry walls are given in Section 7.4. Table 7.1 lists h/t ratios adapted from Table 7-5 of FEMA-356 (ASCE, 2000). These h/t ratios were confirmed for BC seismicity and construction practices by shake-table tests at UBC (Meisl et al, 2005).

The additional requirements of (b) minimum 3 wythes, (c) minimum 6 courses between bonding courses, and (d) adequate restraint at the top of the wall, are needed to use the limits listed in Table 7.1, because they were present in the test specimens at UBC.

B9.2 Aspect Ratio

The in-plane aspect ratio requirements for clay brick masonry walls are the same as for unreinforced concrete masonry walls (see Section B8.3).

B10.0 ROCKING ELEMENTS

The provisions for rocking elements are covered in Sections 8 and A8. This section discusses the reasons behind some of the provisions.

B10.1 Aspect Ratio

The stiffness of the rocking element prior to uplift affects the elastic period of the rocking element. The horizontal displacement (at the top of the rocking element) at which uplift occurs, will be referred to as the "yield drift". This does not imply that yielding is occurring, but is simply the name for the point on the backbone curve where the stiffness changes. Yield drift is a function of strength and stiffness, and is thus affected by the elastic stiffness of the rocking element. The sensitivity study in Section C6.2.3 shows that the rocking model is very sensitive to the yield drift.

In order to investigate the relationship between yield drift and aspect ratio, the yield drift must first be calculated. FEMA-356 (ASCE, 2000) lists Equation (B.10-1) for the rocking resistance of an element (i.e. overturning strength).

$$R_{e-rock} = 0.9\alpha \frac{PL}{h} \qquad \text{(B.10-1)}$$

Where, α = 0.5 for cantilever, 1.0 for piers (restrained top and bottom)

$R_{e\text{-}rock}$ is the required lateral force applied at the top of the rocking element to commence rocking action.

The elastic displacements (i.e. displacement up to the point of rocking) for concrete or masonry shearwall cantilevers or piers was taken from Kim and White (2003) listed in Equations (2.72 and 2.73). The displacement of the walls is function of the lateral load, which will be replaced by Re-rock to establish the yield drift. Equation (B.10-2) gives the yield drift for cantilever walls in rocking, and Equation (B.10-3) the yield drift for pier walls:

$$\frac{\Delta_y}{h} = R_{e-rock}\left(\frac{h^2}{3EI} + \frac{1.2}{GA}\right) \qquad \text{(B.10-2)}$$

$$\frac{\Delta_y}{h} = R_{e-rock}\left(\frac{h^2}{12EI} + \frac{1.2}{GA}\right) \qquad \text{(B.10-3)}$$

I and *A* should be based on the state of the wall at the rocking load. This may be either gross section properties or an equivalent cracked section (secant stiffness not tangent stiffness). While the axial load on the wall is not in Equations (B.10-2 and 3) it does influence the displacements by affecting *I* and *A* and through Equation (B.10-1).

Using these equations and properties of a concrete masonry wall, the relationship between yield drift and aspect ratio for cantilever and pier rocking elements was calculated. This relationship is shown in Figure B.10-1, which clearly shows that the yield drift becomes exponentially larger as the aspect ratio increases, for both cantilever and pier walls. This is because while squat walls are shear dominated, the taller walls are flexurally dominated, and height is a function of the flexural stiffness.

Figure B.10-1 indicates that the yield drifts of the rocking models R-1, R-2 and R-3, which are 0.15%, 0.6% and 1.2% respectively, are related to the aspect ratios(H/L) of 1, 2.5 and 3.5 for cantilever walls, and twice that for piers. This information is reflected in the aspect ratio limits in Sentence 8.1(2). More slender rocking elements are beyond the scope of these guidelines.

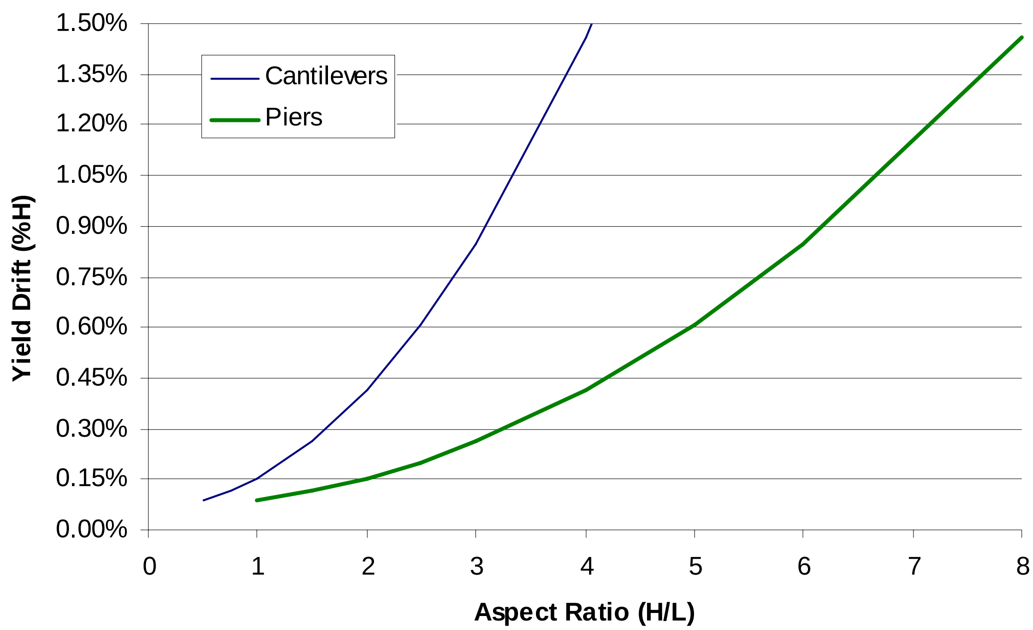


Figure B.10-1 Yield Drift vs. Aspect Ratio for Typical Rocking Elements

B10.2 Centre of Mass

The prototype models in the 2^{nd} Edition Bridging Guidelines are based on 2-storey models with the majority of the deformation occurring in the 1^{st} storey. This is a conservative assumption, and works well for buildings from 1 to 3 stories in height (see Section B3.3).

This assumption also works well with the rocking prototypes when they re used to represent pier rocking elements stacked on top of each other. However this is an overly conservative estimate for a single multi-storey cantilever wall.

Equation (8-1) was introduced to account for the use with multi-storey rocking cantilevers. It is used to modify the existing prototype resistance tables to account for a center of gravity from 3 to 6 meters. This equation is empirical and is based on analysis done in Quakesoft.

Conversely Equation (8-2) accounts for very short rocking cantilevers, which have a higher demand than the base prototypes. This equation increases the demands for centers of gravity between 1.5 and 3 meters.

These two equations replace the need for a separate storey height equation (see Section B3.5).

B11.0 DIAPHRAGMS

The modeling and analysis of the diaphragms is covered in Section C5.8. In order to maintain a relatively simple procedure for assessing or retrofitting diaphragms several assumptions and idealizations where made:

- The diaphragms were shear dominated. This works well for low aspect ratios, but does not work well when flexure dominates. See Section B11.4.
- A constant element length of 2.5 metres was used, regardless of the diaphragm span. This provided a consistent measurement of the shear strain. This distance was felt to be representative of the space between major roof supports.
- The diaphragms were analyzed on semi-rigid end walls, which did not yield. This is conservative for when the end walls are flexible and yielding, as the walls will absorb some of the energy from the ground motions. It is not conservative for masonry or concrete walls. It was felt that it would be too complex to have two sets of tables based on end wall construction, and thus the more conservative option was chosen.
- Unlike LDRSs, where the designer has options in selecting the Governing Drift Limit (GDL), the diaphragm performance criteria are set (see Section B11.1). The strengths listed in the resistance tables for diaphragms have met all the performance criteria.

B11.1 Performance Criteria

There are two performance criteria for the diaphragm prototypes to meet the performance objective (see Section B1.4) of life-safe for a 2% in 50-year probability of exceedance. These two criteria are to meet the Maximum Diaphragm Inelastic Strain Limit (DISL) as listed in Table A.10-1 (these are prototype specific), and to have a maximum displacement of 200mm. A displacement of 200mm is roughly equivalent to half the width of a typical clay brick masonry wall.

The DISL protects the diaphragm for excessive damage, such that it is still able to transfer sufficient load to the LDRSs.

The second criterion is to protect URM out-of-plane walls from a rocking failure. The out-of-plane requirements for URM walls (sections B8.1 and B9.1) assume that there is a good connection at the top of the wall, and that the displacements at the top and the bottom are similar.

B11.2 Shear Strains from Different LDRSs

Sentence 10.9(2) lists maximum shear strains due to dissimilar GDLs from LDRSs. This situation can only arise for flexible diaphragms (as rigid diaphragms must use a single GDL for all LDRSs), and is only significant when extremely different LDRSs are close together (but more than 5 metres apart (see Sentence 10.6(4)).

The procedure is to convert the GDLs into displacements at the top of the 1st storey. Take the difference of the displacements, and divide it by the distance between the LDRSs. If the resulting drift is greater than the permissible value, the GDLs must be changed.

B11.3 Eccentricity Limits

Maximum plan eccentricities for rigid diaphragms are given in Sentence 10.6(2). The calculation of eccentricity is based on strength, instead of stiffness. The strength of an LDRS is proportional to the mass of the building (or tributary area) for which it provides lateral deformation resistance. This is why Section 10.6 refers to the "redistribution of inertial mass".

Analyses in Quakesoft and CANNY were done to investigate the influence of eccentricities, and how much eccentricity still resulted in a reasonable match with the values in the resistance tables. A 3-dimensional single storey model with 4 LDRSs (each side of a square structure) was analyzed. The combined strength of both walls in an orthogonal direction was equal to the values in the resistance table, but the strength of the walls themselves varied. The eccentricity limits were based on this analysis.

While it might be expected that any eccentricity would be less conservative than none, the out-of-plane LDRSs limit the torsional rotations reasonably well.

LDRSs with a pinched or slip type hysteretic behaviour (see Table C.5-1) did not perform as well with an eccentric arrangement of strengths. One explanation for this is that these prototypes offer little initial resistance upon load reversal, which is important for the out-of-plane LDRSs resisting the torsional rotations.

B11.4 Aspect Ratio

The diaphragm models do not include the flexural response of the diaphragms. For small aspect ratios, this is a valid assumption, because the shear deformations (elastic and inelastic) are far greater than the elastic flexural deformations. For diaphragms with a high aspect ratio, 4:1 or more, these values may be inappropriate.

B11.5 Diaphragm Chord Forces

Diaphragm chord forces are given in Section 10.5. The diaphragm force distributions for which these equations are based on are shown in Figure A.10-3.

The derivation of Equations (10-1) is as follows:

> The diaphragm has a constant shear stress from the centre diaphragm to either side. This represents the response to a point load acting at the centre.

The point load is equal to (the 100 converts the R_{md} into a decimal):

$$P = \frac{2 \cdot R_{md} W_d}{100}$$

The moment in the diaphragm is:

$$M = \frac{PL}{4} = \frac{R_{md} W_d}{200}$$

Chord force is equal to the moment divided by the distance between the chords:

$$F_c = \frac{M}{s} = \frac{R_{md} W_d}{200s}$$

The derivation of Equations (10-2) is as follows:

This equation is the “typical” equation to calculate chord force. There is a uniformly distributed load on the diaphragm (the 100 converts the R_{md} into a decimal):

$$w = \frac{2R_{md} W_d}{100L}$$

The moment in the diaphragm is:

$$M = \frac{wL^2}{8} = \frac{R_{md} W_d}{400}$$

Chord force is equal to the moment divided by the distance between the chords

$$F_c = \frac{M}{s} = \frac{R_{md} W_d}{400s}$$

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PART C DEVELOPMENT OF RESISTANCE TABLES

Table of Contents

C1.0 INTRODUCTION

The resistance tables provided in the 2nd Edition Bridging Guidelines provide engineers the rational means to assess or retrofit structural systems to a desired level of performance (i.e. drift). These tables are the product of a very large analysis program that has been validated and thoroughly reviewed by an external peer review committee.

Non-linear dynamic analysis (NLDA) was the primary analysis method used to generate the values for the resistance tables. NLDA was chosen for this role, because it offers the most precise measurement of the maximum displacements, in the inelastic range, of the lateral deformation resisting systems (LDRS). Inelastic displacement, or drift, is a relatively simple yet effective means of gauging damage and subsequently structural performance.

The 2005 NBCC permits the use of NLDA as an analysis method for establishing the demands form earthquake loading, provided a special study is performed. The internal and external validation and review of the analyses used in the 2nd Edition Bridging Guidelines satisfy this condition.

This chapter contains the details of the development of the resistance tables found in the 2nd Edition Bridging Guidelines. Section C2 contains general information. Section C3 describes the analysis methods used to generate the tables, and Section C4 presents the ground motions used in those analyses. Section C5 provides the details of the prototype models for all of the LDRSs. Section C6 shows a comparison of the values generated from the analysis, and Section C7 compares the analysis results between UBC and the EPR.

C2.0 GENERATING RESISTANCE TABLES

Advanced analysis techniques (see Section C3.0) were used to generate the resistance values in the 2nd Edition Bridging Guidelines. Each prototype was analysed multiple times for each combination of seismic zone and site class. These multiple analyses provided data to develop a relationship between strength (resistance) and maximum interstorey drift. Many more analyses were performed than are shown on the resistance tables, for the purposes of defining the trends and validation. The resistance tables summarize the resistance required to limit the drifts to desired level. This level ranges from the ISDL down to a recommended level, below which the prototype can no longer efficiently limit the drift.

C2.1 Ground Motions for Factored Resistance Tables

The resistance tables are based on the results of a suite of ground motions (see Section C4). The values shown on the table are the retrofit numbers, and are based on the mean plus one standard deviation response of the individual ground motions in the suite. The standard deviation gives an extra measure of safety to the performance objective.

The numbers derived from the analyses are divided by the appropriate overstrength factor, R_o, for each LDRS, before being entered in the tables. Thus the values on the tables are considered to be factored, and are compatible with strength calculations from the appropriate material design codes.

The assessment values are 80% of the values shown on the tables.

C2.2 Minimum and Maximum Code-Referenced Resistance

Upper and lower bounds were imposed on the analysis results for the resistance tables, at the ISDL of each prototype. The lower bound was 60% of the base shear from the 2005 NBCC quasi-static analysis, with an assumed period of 0.2 seconds. The maximum is 100% of code. Both minimum and maximum values use an importance factor of I=1.0. The minimum value is a conservative measure. The maximum is only applied in cases where the nonlinear dynamic results are in excess of the full code level, which was typical on Site Class E (see Section C4.4). For drifts less than the ISDL, in cases where the maximum was applied at the ISDL, the demand will be greater than the code maximum because a higher performance level is implied.

Values for $S_a(0.2)$ were based on one specific municipality for each Seismic Zone (see Section B5.3) and were taken from Appendix C, Table 2C of the 2005 NBCC. F_a was taken as 1, based on values given on Table 4.1.8.4.B of the 2005 NBCC, and R_d and R_o factors for the prototypes were taken from Table 4.1.8.9.

Not all of the R_d and R_o values are the same as listed on Table 4.1.8.9. The prototypes M-1 and B-1 use R_d and R_o factors of 1.5 and 1.5, where the NBCC stipulates values of 1.0 and 1.0 respectively. Rocking prototypes used R_d of 2.0 and R_o of 1.0.

Table C.2-1 lists all of the assumed R_d and R_o values for the prototypes, as well as the minimum and maximum resistances for Seismic Zones 2 through 5 on Site Class C. Tables C.2-2 and C.2-3 list the same information for Site Classes D and E. These maximum values were calculated from Sentence 4.1.8.11.1 of the 2005 NBCC, and include the $^2/_3$ reduction for having an R_d value of 1.5 or greater.

A few particular prototypes had a lower bound limit of 50% of the code base shear, instead of 60%. These prototypes were: W-2 (Unblocked Wood-frame shearwalls), M-1 and B-1 (Unreinforced masonry in sliding shear), and R-1 (low aspect ratio rocking element). These prototypes all have relatively conservative R_d and R_o values, which results in high base shear demand. It was felt that the models used for these prototypes in Section C5 were detailed and adequately accounted for the inherent behaviour of the material, thus allowing less conservatism in rating their performance.

Table C.2-1 Minimum and Maximum Factored Resistances for Site Class C

Prototype Parameters			***Factored Resistance (%W) on Site Class C***							
LDRS	**R_d**	**R_o**	**Zone 2** Sa(0.2)=0.42		**Zone 3** Sa(0.2)=0.73		**Zone 4** Sa(0.2)=0.94		**Zone 5** Sa(0.2)=1.2	
			60%	*100%*	*60%*	*100%*	*60%*	*100%*	*60%*	*100%*
W-1	3.0	1.7	3%	5%	6%	10%	7%	12%	9%	16%
W-2*	2.0	1.7	*4%*	8%	*7%*	14%	*9%*	18%	*12%*	24%
S-1	3.0	1.3	4%	7%	7%	12%	10%	16%	12%	21%
S-2	3.0	1.3	4%	7%	7%	12%	10%	16%	12%	21%
S-3	4.0	1.5	3%	5%	5%	8%	6%	10%	8%	13%
S-4	3.5	1.5	3%	5%	6%	9%	7%	12%	9%	15%
C-1	2.0	1.4	6%	10%	10%	17%	13%	22%	17%	29%
C-2	1.5	1.3	9%	14%	15%	25%	19%	32%	25%	41%
C-3	4.0	1.7	2%	4%	4%	7%	6%	9%	7%	12%
C-4	2.5	1.4	5%	8%	8%	14%	11%	18%	14%	23%
C-5	1.5	1.3	9%	14%	15%	25%	19%	32%	25%	41%
M-1*	1.5	1.5	*6%*	12%	*11%*	22%	*14%*	28%	*18%*	36%
M-2	1.5	1.5	7%	12%	13%	22%	17%	28%	21%	36%
B-1*	1.5	1.5	*6%*	12%	*11%*	22%	*14%*	28%	*18%*	36%
R-1*	2.0	1.0	*7%*	14%	*12%*	24%	*16%*	31%	*20%*	40%
R-2	2.0	1.0	8%	14%	15%	24%	19%	31%	24%	40%
R-3	2.0	1.0	8%	14%	15%	24%	19%	31%	24%	40%

* - this prototype has a lower bound associated with 50% - see Section C2.2

Table C.2-2 Minimum and Maximum Factored Resistances for Site Class D

Prototype Parameters			*Factored Resistance (%W) on Site Class D*							
LDRS	**R_d**	**R_o**	**Zone 2** Sa(0.2)=0.42		**Zone 3** Sa(0.2)=0.73		**Zone 4** Sa(0.2)=0.94		**Zone 5** Sa(0.2)=1.2	
			60%	*100%*	*60%*	*100%*	*60%*	*100%*	*60%*	*100%*
W-1	3.0	1.7	4%	7%	6%	11%	8%	13%	10%	16%
W-2*	2.0	1.7	*5%*	10%	*8%*	16%	*10%*	20%	*12%*	24%
S-1	3.0	1.3	5%	9%	8%	14%	11%	18%	13%	21%
S-2	3.0	1.3	5%	9%	8%	14%	11%	18%	13%	21%
S-3	4.0	1.5	3%	6%	5%	9%	7%	11%	8%	14%
S-4	3.5	1.5	4%	7%	6%	10%	8%	13%	9%	15%
C-1	2.0	1.4	7%	12%	12%	19%	15%	25%	17%	29%
C-2	1.5	1.3	11%	18%	17%	28%	21%	35%	25%	42%
C-3	4.0	1.7	3%	5%	5%	8%	6%	10%	7%	12%
C-4	2.5	1.4	6%	10%	9%	15%	12%	20%	14%	23%
C-5	1.5	1.3	11%	18%	17%	28%	21%	35%	25%	42%
M-1*	1.5	1.5	*8%*	15%	*12%*	24%	*15%*	31%	*18%*	36%
M-2	1.5	1.5	9%	15%	14%	24%	18%	31%	22%	36%
B-1*	1.5	1.5	*8%*	15%	*12%*	24%	*15%*	31%	*18%*	36%
R-1*	2.0	1.0	*9%*	17%	*14%*	27%	*17%*	34%	*20%*	41%
R-2	2.0	1.0	10%	17%	16%	27%	21%	34%	24%	41%
R-3	2.0	1.0	10%	17%	16%	27%	21%	34%	24%	41%

* - this prototype has a lower bound associated with 50% - see Section C2.2

Table C.2-3 Minimum and Maximum Factored Resistances for Site Class E

Prototype Parameters			*Factored Resistance (%W) on Site Class E*							
LDRS	**R_d**	**R_o**	**Zone 2** Sa(0.2)=0.42		**Zone 3** Sa(0.2)=0.73		**Zone 4** Sa(0.2)=0.94		**Zone 5** Sa(0.2)=1.2	
			60%	*100%*	*60%*	*100%*	*60%*	*100%*	*60%*	*100%*
W-1	3.0	1.7	5%	9%	6%	11%	7%	12%	8%	14%
W-2*	2.0	1.7	*7%*	13%	*8%*	16%	*9%*	17%	*11%*	21%
S-1	3.0	1.3	7%	12%	8%	14%	9%	15%	11%	18%
S-2	3.0	1.3	7%	12%	8%	14%	9%	15%	11%	18%
S-3	4.0	1.5	5%	8%	5%	9%	6%	10%	7%	12%
S-4	3.5	1.5	5%	9%	6%	10%	7%	11%	8%	14%
C-1	2.0	1.4	10%	16%	12%	20%	13%	21%	15%	26%
C-2	1.5	1.3	14%	23%	17%	28%	18%	30%	22%	37%
C-3	4.0	1.7	4%	7%	5%	8%	5%	9%	6%	11%
C-4	2.5	1.4	8%	13%	9%	16%	10%	17%	12%	21%
C-5	1.5	1.3	14%	23%	17%	28%	18%	30%	22%	37%
M-1*	1.5	1.5	*10%*	20%	*12%*	24%	*13%*	26%	*16%*	32%
M-2	1.5	1.5	12%	20%	15%	24%	16%	26%	19%	32%
B-1*	1.5	1.5	*10%*	20%	*12%*	24%	*13%*	26%	*16%*	32%
R-1*	2.0	1.0	*11%*	23%	*14%*	27%	*15%*	30%	*18%*	36%
R-2	2.0	1.0	14%	23%	16%	27%	18%	30%	22%	36%
R-3	2.0	1.0	14%	23%	16%	27%	18%	30%	22%	36%

* - this prototype has a lower bound associated with 50% - see Section C2.2

C2.2.1 Diaphragm Resistances

Diaphragm resistance values have not been limited to any percentage of their equivalent code-based design, nor do diaphragms have to remain elastic. See Section B11.1 for information on diaphragm performance criteria.

C3.0 ANALYSIS METHODS

Two types of advanced analysis methods used in the process of generating the data for the resistance tables. These methods were non-linear dynamic analysis (NLDA) and non-linear static analysis (Push-over). Both of these methods are able to predict inelastic deformations between stories, which are required for a precise measurement of the life-safety performance of a structure.

C3.1 Non-linear Dynamic Analysis

Two computer programs were used to perform the NLDA:

Quakesoft, developed by TBG Seismic Consultants, was the package used to generate the majority of the results used in the resistance tables. Quakesoft is an implicit (directly solves equation of motion using very small time-steps [10^{-4} seconds]) non-linear dynamic analysis that uses displacement time histories as the seismic input. While Quakesoft is limited to only shear-spring elements (representing the response of a storey as a SDOF), these elements can be customized to have virtually any shape of backbone curve (including negative slope) or hysteretic behaviour, and damping can be modified for each branch of the elements response.

CANNY was the program used primarily as a validation tool, but was also used to generate some of the results in the tables. CANNY uses an explicit (uses larger timesteps [0.005 seconds] but iterates to find the correct solution) stiffness-based approach (direct stiffness method) with the seismic input based on acceleration time histories. CANNY has many different types of structural elements and hysteretic models to choose from, but unlike Quakesoft, they can not be readily customized. Further details on the CANNY program itself can be found in the CANNY Technical Manual (Kanging, 2006).

The time histories used for both programs are described in Section C4. The details of the prototype models are outlined in Section C5.

C3.2 Non-linear Static Analysis

The push-over analysis was performed using the provisions in FEMA 440 (ATC, 2005) referred to as the Coefficient Method (also known as Displacement Modification Method). FEMA-440-DM was used primarily as a validation tool.

FEMA-440-DM was used to calculate the total displacement demand, at the top of the structure, of most of the prototypes. This total displacement was converted into inter-storey drifts at both levels. The proportioning of drifts at each level, for each prototype, was based on observed inter-storey drift ratios from the CANNY NLDA.

The parameters used in all of the FEMA-440-DM models were as follows. C_0 (the conversion from single to multi-degree of freedom factor) was set to 1.2 (triangular load pattern for a two-storey building). C_1 (conversion of elastic to inelastic displacements) was used for all of the prototypes. C_2 (degrading hysteretic parameter) was used for some of the prototypes, as indicated below. C_3 (instability factor) is an old factor from FEMA-356 (ASCE, 2000) and was not used in this study. In lieu of C_3, an R_{max} cut-off was used. Any prototype combination with an R (total force reduction factor) greater than R_{max} was considered to be unstable. To calculate R_{max}, the slope of the back-bone curve after yielding is required, as is the length of the plateau of maximum strength. The influence of P-delta was included here, and modified the slope of the back-bone curve after yielding, based on the capacity of the prototype.

The FEMA-440-DM analysis uses the periods and yield drifts (and thus backbone curves) from the CANNY model. These are outlined in Section C5.

The FEMA-440-DM analysis does not require the use of ground motion records, merely an acceleration spectrum. The acceleration spectra corresponding to the seismic zone and site class from the 2005 NBCC was used (see Section C4.6).

C4.0 GROUND MOTIONS

Nonlinear dynamic analysis requires ground motions records (or time histories) to simulate the seismic demands on the modeled structures. Records are typically obtained from databases of real earthquake recordings, although sometimes simulated or artificial records are used when no suitable real records are available.

The ground motion records must be carefully selected to ensure that they adequately reflect the seismic conditions of the site or region. In addition, due to the uncertainties of seismic hazards and sensitivities of nonlinear analysis, not one, but a suite of records should be used for the analyses.

C4.1 Suite of Ground Motions

The ground motion suite for the 2nd Edition Bridging Guidelines is made up records from a number of sources: PEER Strong Motion Database, COSMOS Virtual Data Centre and the ATC-55 Project (i.e. FEMA-440). The records were selected based on the following criteria:

1) Site Class C record (firm ground)
2) Crustal Earthquake (real, not artificial or simulated)
3) Average spectral velocity within ±40% of the 2005 NBCC Vancouver Site Class C design spectrum (55.4 cm/s)

Based on the above criteria, the following records, listed on Table C.4-1, were selected.

Table C.4-1 Ground Motion Suite for Bridging Guidelines 2nd Edition

No.	Station	PGA *cm/s^2*	PGV *cm/s*	PGD *cm*	Sv* *cm/s*
SO90	Sherman Oaks - 105 deg	210	29.4	8.7	44.3
WW235	Wadsworth - 235 deg	297	32.9	9.8	48.6
WW325	Wadsworth - 325 deg	382	21.3	4.6	42.0
CC0	Canyon Country - 0 deg	389	44.1	11.2	66.7
Sara0	Saratoga - 0 deg	495	32.6	17.2	69.3
CP196	Canoga Park - 196 deg	381	59.8	12.4	78.0
CP106	Canoga Park - 106 deg	343	34.1	8.8	52.3
PK90	Pacoima Kagel - 90 deg	295	30.9	10.6	66.0
MD35	12520 Mulholland Drive - 35 deg	577	29.4	6.2	49.0
Gil67	Gilroy Gavilon College - 67 deg	349	22.8	5.7	39.6

Note: Sv* is the average spectral pseudo velocity (5% damping) taken between 0.5-1.5 sec.

Figure C.4-1 on the next page illustrates the acceleration time history for each of the ground motions. Following that, Figure C.4-2 shows the unscaled acceleration spectra of the suite, and Figure C.4-3 the velocity spectra.

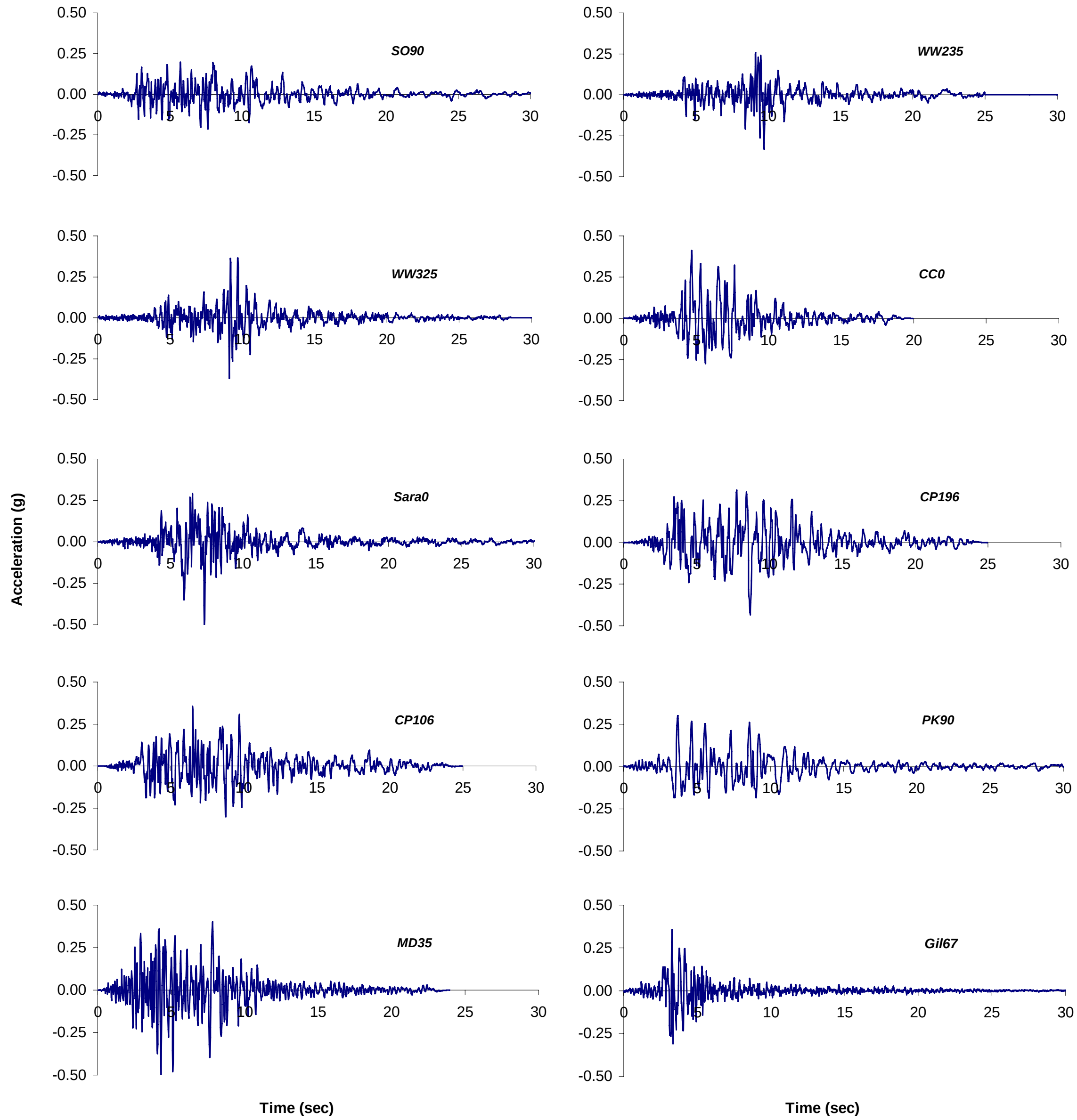


Figure C.4-1 Acceleration Time Histories of Ground Motion Suite

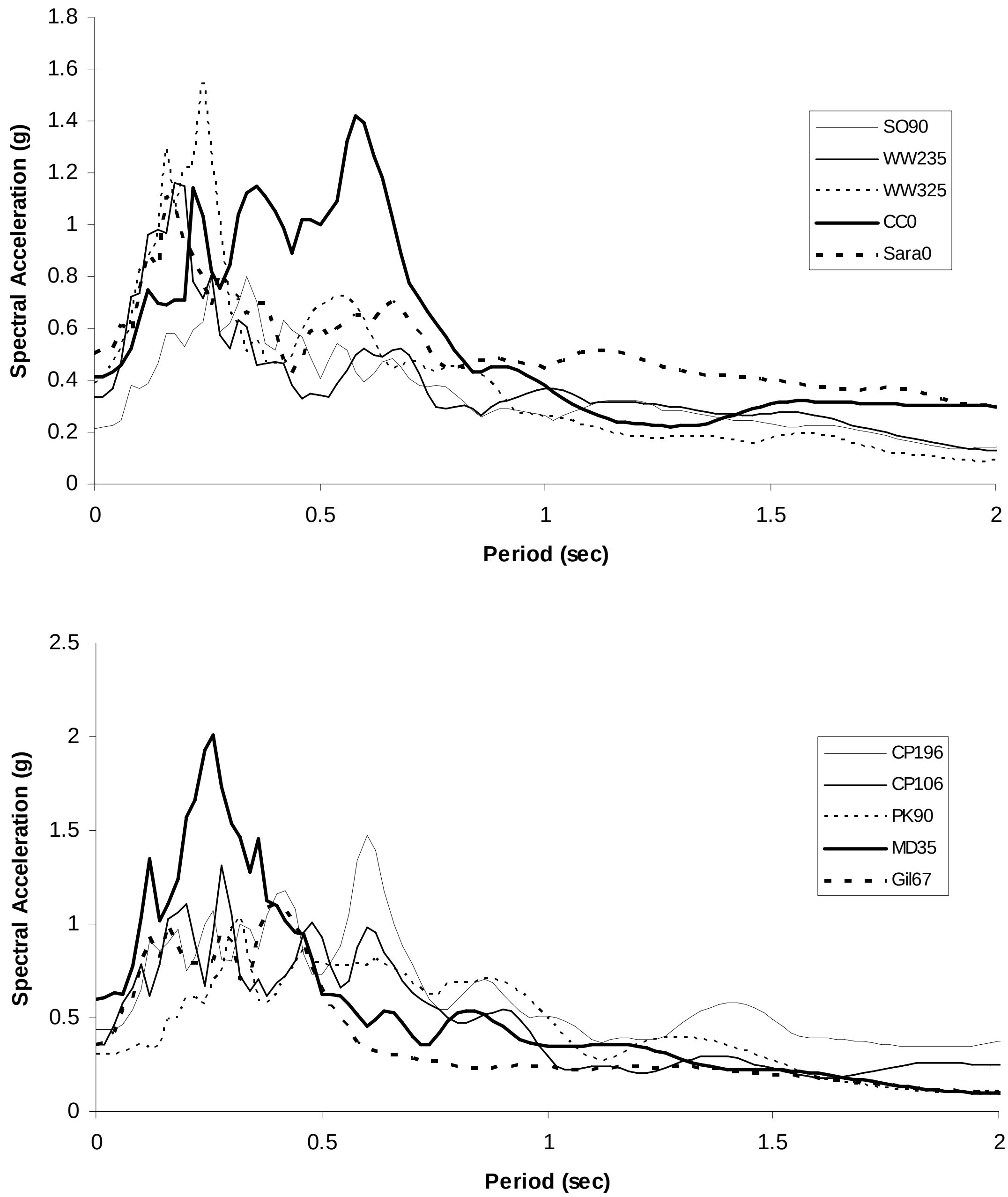
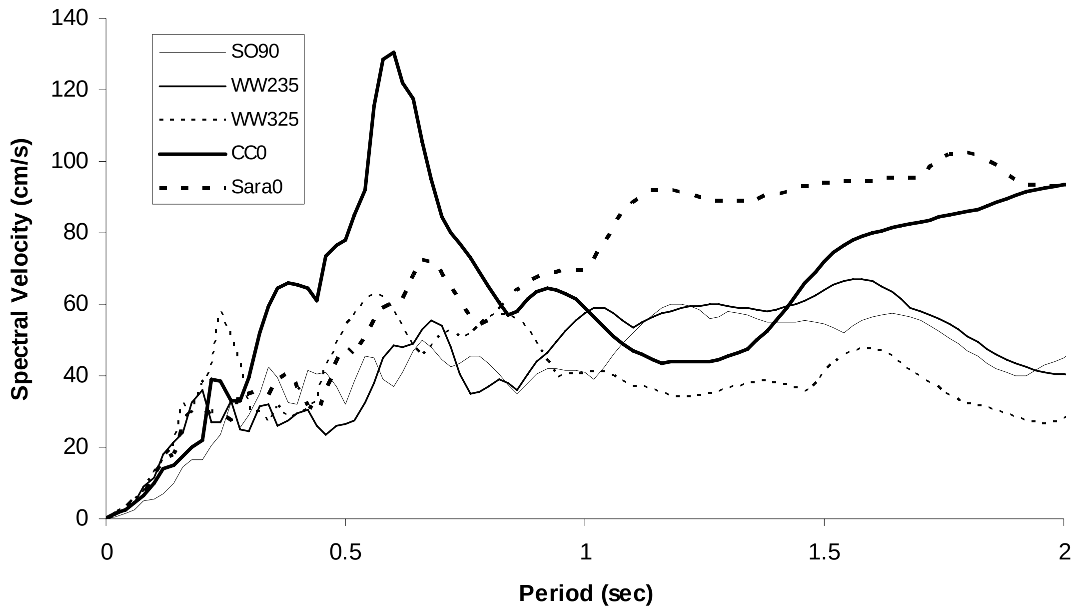


Figure C.4-2 Acceleration Spectra of Ground Motion Suite



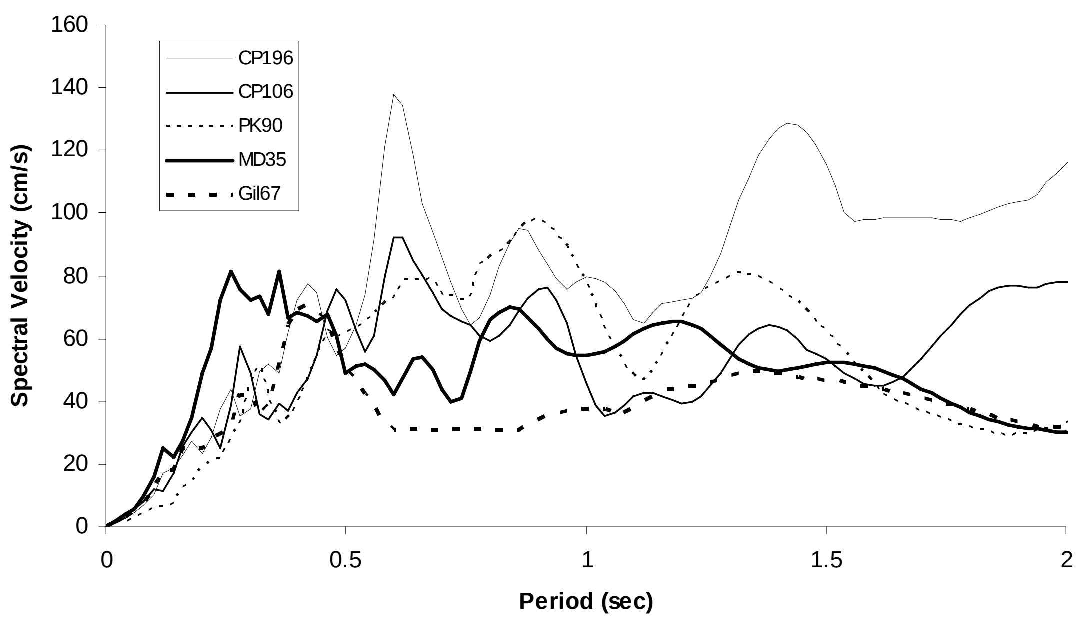


Figure C.4-3 Velocity Spectra of Ground Motion Suite

C4.2 Scaling Ground Motions to Seismic Demands on Site Class C

The scaling factor is scalar applied to the acceleration time history when the records are input into the analysis (structural and/or soil). The scaling procedure does not alter the frequency content of the original records.

The S_v*, on Table C.4-1, is calculated at each 0.01 second interval. The scaling factor for each record, on a particular seismic zone for Site Class C, is calculated by dividing the S_v* for each record by the S_v* for the appropriate NBCC design spectra.

The Acceleration spectra for Seismic Zones 2-5 are shown in Figure C.4-4. The corresponding Velocity spectra are shown in Figure C.4-5, which also include the S_v* for the spectra.

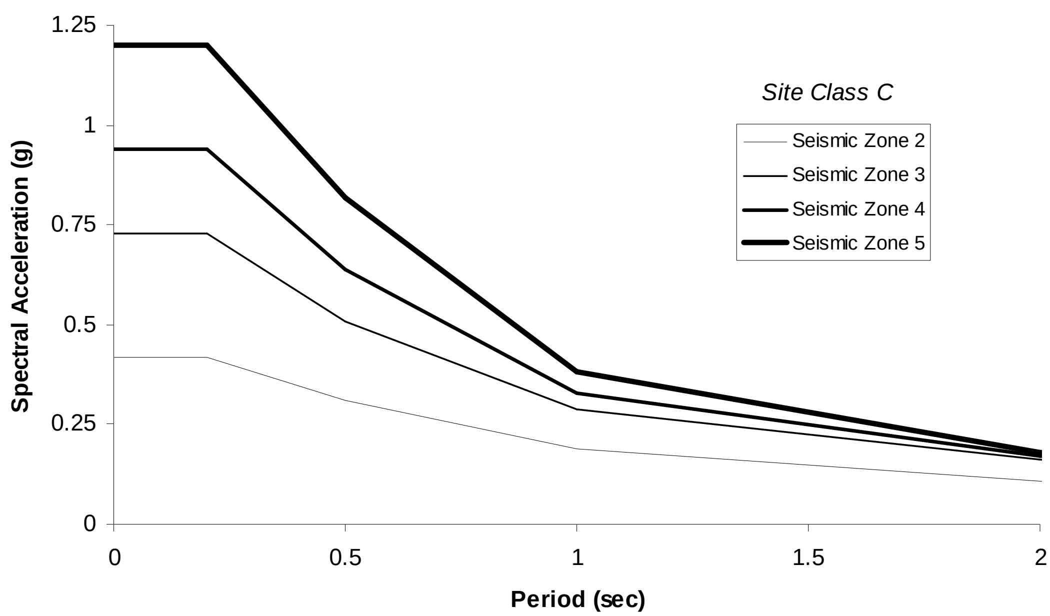


Figure C.4-4 Acceleration Spectra for all BC Seismic Zones on Site Class C.

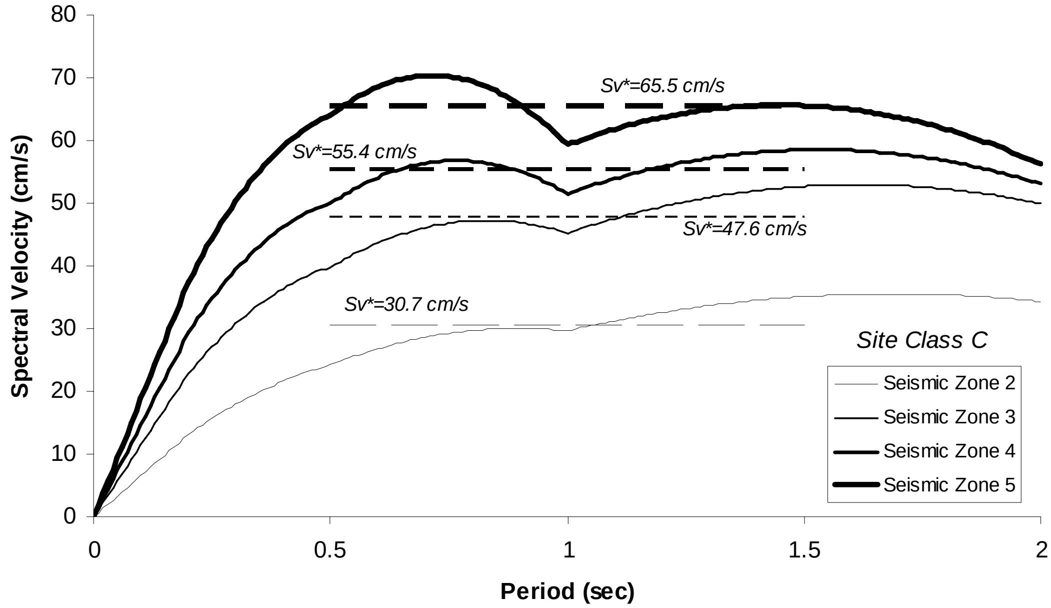


Figure C.4-5 Velocity Spectra for all BC Seismic Zones on Site Class C.

The scaling factors used for the ground motion suite for all the seismic zones in British Columbia are shown in Table C.4-2.

Table C.4-2 Ground Motion Scaling Factors on Site Class C for All Seismic Zones

Ground Motion	Zone 2 (Princeton)	Zone 3 (Chilliwack)	Zone 4 (Vancouver)	Zone 5 (Victoria)
SO90	0.69	1.07	1.25	1.48
WW235	0.63	0.98	1.14	1.35
WW325	0.73	1.13	1.32	1.56
CC0	0.46	0.71	0.83	0.98
Sara0	0.44	0.69	0.80	0.95
CP196	0.39	0.61	0.71	0.84
CP106	0.59	0.91	1.06	1.25
PK90	0.47	0.72	0.84	0.99
MD35	0.63	0.97	1.13	1.34
Gil67	0.78	1.20	1.40	1.66

C4.3 Scaling Ground Motions to Seismic Demands on Site Class D

The scaling for Site Class D was carried out in the same fashion as with Site Class C (see Section C4.2). The same suite of real ground motions was also used. Figures C.4-6 and C.4-7 show the 2005 NBCC seismic demands over all seismic zones, and Table C.4-3 gives the scaling factors applied to the ground motion suite.

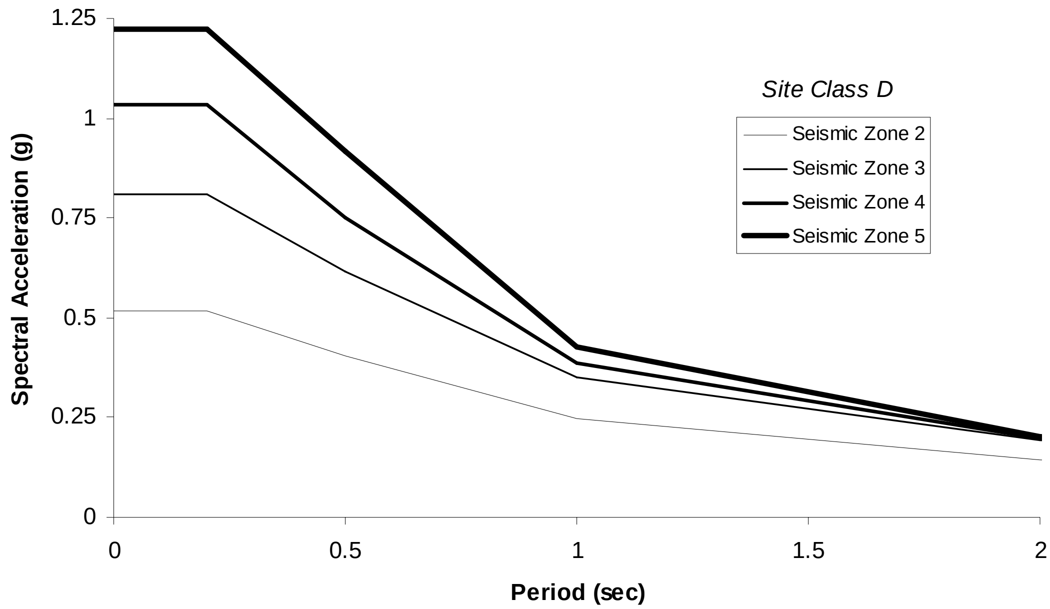


Figure C.4-6 Acceleration Spectra for all BC Seismic Zones on Site Class D.

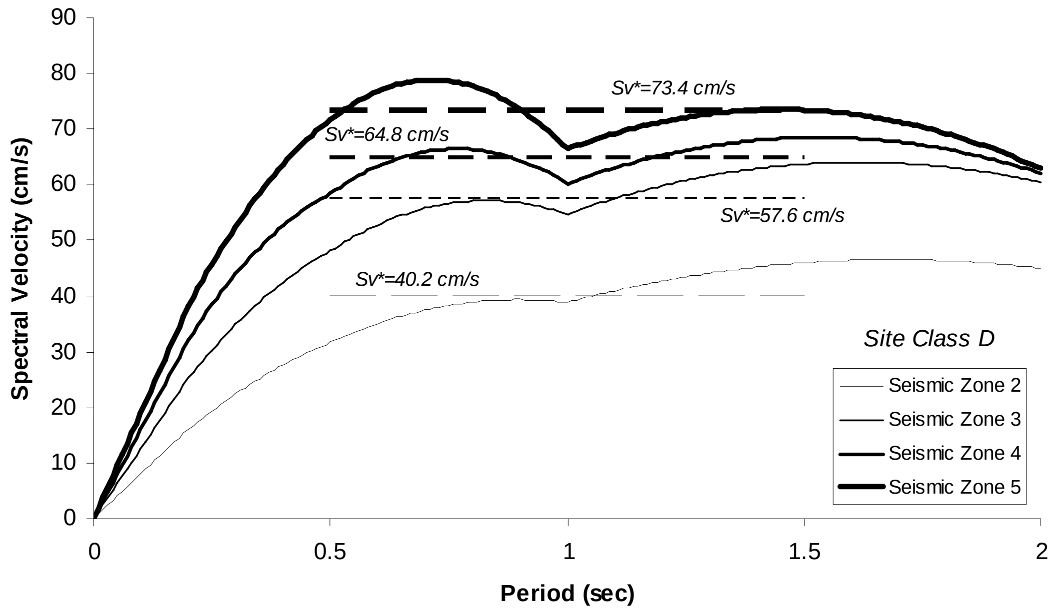


Figure C.4-7 Velocity Spectra for all BC Seismic Zones on Site Class D.

Table C.4-3 Ground Motion Scaling Factors on Site Class D for All Seismic Zones

Ground Motion	Zone 2 (Princeton)	Zone 3 (Chilliwack)	Zone 4 (Vancouver)	Zone 5 (Victoria)
SO90	0.91	1.30	1.46	1.66
WW235	0.83	1.18	1.33	1.51
WW325	0.96	1.37	1.54	1.75
CC0	0.60	0.86	0.97	1.10
Sara0	0.58	0.83	0.94	1.06
CP196	0.52	0.74	0.83	0.94
CP106	0.77	1.10	1.24	1.40
PK90	0.61	0.87	0.98	1.11
MD35	0.82	1.17	1.32	1.50
Gil67	1.02	1.46	1.64	1.86

C4.4 Scaling Ground Motions to Seismic Demands on Site Class E

The ground motion scaling for Site Class E has been done differently than on Site Class C and D. Initial analyses were conducted using the same scaling method as on Site Class C and D. However the results indicated base shear demands well above the 2005 NBCC requirements, and also above the upper bound results (see Section C2.2).

Different scaling factors were calculated for each set of material prototypes. These scaling factors were calibrated such that the prototypes have the same base shear demands, at the ISDL, as if they had been designed according to the 2005 NBCC (Section C2.2). This effort was put into the resistance tables, instead of just defaulting to the 2005 NBCC, so that assessments and retrofits could still benefit from the Toolbox method (Section 2.0) and the resistance of systems not recognized by the 2005 NBCC (e.g. URM).

It is highly recommended that each school on Site Class E undergo a Site Response Analysis (see Section B4.0). A Site Response Analysis has a significant chance of reducing the base shear demands for all prototypes.

The seismic base shear demand was calculated using the provisions in the 2005 NBCC, with an assumed period of 0.2 seconds. The Site Class E spectra, for Seismic Zones 2-5, are shown in Figure C.4-8. Using this base shear capacity, each prototype model was then taken and analysed in Quakesoft. Each ground motion was scaled so that the maximum drift from each analysis was equal to the ISDL for that prototype. The scaling factors were then averaged for the entire suite. The final scaling factors are shown below in Table C.4-4

Future editions of these guidelines will investigate the appropriate scaling factors for Site Class E. The 2005 NBCC spectra appear to be inappropriate for the combination of the methods of analyses, method of ground motion scaling, and the typical fundamental periods of the prototypes used in this study.

C4.5 Combining Values for Site Classes

For certain combinations of prototypes and seismic zones, the results for two of the site classes may have been very close, such that their lines were difficult to distinguish on the resistance tables. In these instances, the more conservative of the two sets of values were used for both site classes.

C4.6 Why Velocity is Used for Scaling

The 2nd Edition Bridging Guidelines use the velocity spectra as the basis for scaling. While accelerations are often used for scaling, the velocity in a structure is a better parameter to gauge structural damage, because it is related to kinetic energy. Accelerations are a better prediction for criteria such as non-structural damage. As the performance objective for these guidelines was life safety, which is strongly related to structural damage, spectral velocity was considered of paramount importance.

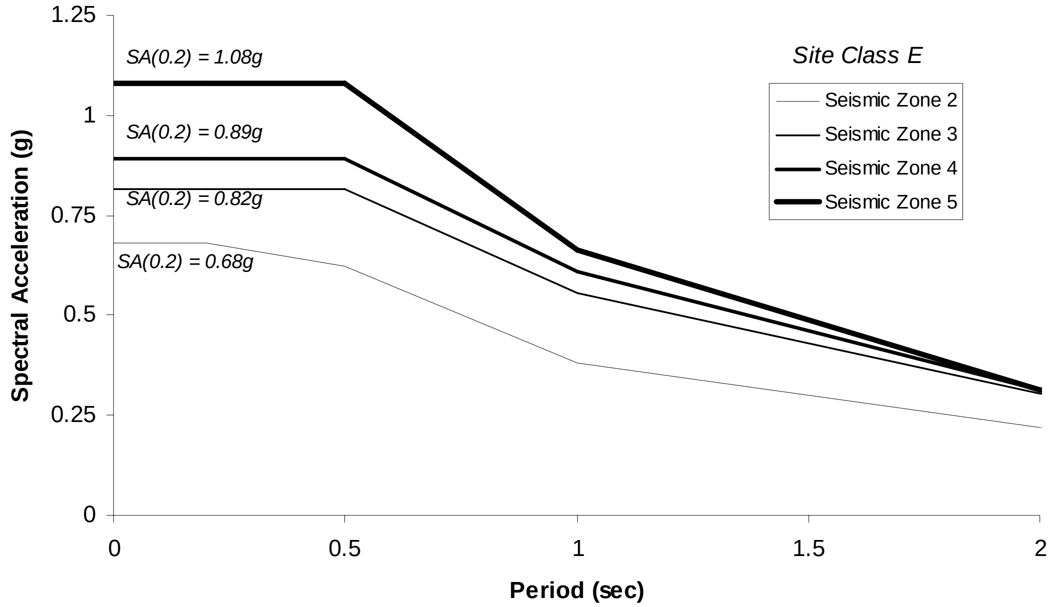


Figure C.4-8 Acceleration Spectra for all BC Seismic Zones on Site Class E.

Table C.4-4 Ground Motion Scaling Factors on Site Class E for All Seismic Zones

Prototype	Zone 2 (Princeton)	Zone 3 (Chilliwack)	Zone 4 (Vancouver)	Zone 5 (Victoria)
W-1	1.13	1.23	1.30	1.40
W-2	1.25	1.32	1.36	1.60
S-1	0.90	1.02	1.05	1.17
S-2	0.79	0.86	1.00	1.18
S-3	0.70	0.75	0.85	1.05
S-4	0.55	0.62	0.70	1.00
C-1	1.05	1.17	1.17	1.32
C-2 and M-2	0.94	1.04	1.07	1.23
C-3, M-1, B-1	0.86	1.00	1.06	1.17
C-4 and C-5	0.55	0.86	1.00	1.18
R-1 to R-3	0.73	1.04	1.17	1.32
D-1	1.13	1.23	1.30	1.40
D-2	1.25	1.32	1.36	1.60
D-3 and D-4	0.73	1.04	1.17	1.32
D-5	0.90	1.04	1.05	1.17
D-6	0.73	0.86	1.00	1.18

C4.7 Spectra for Non-linear Static Analysis

The FEMA-440-DM method uses the spectral acceleration at the effective period of the prototype. As the 2nd Edition Bridging Guidelines use the seismic hazard data from the 2005 NBCC, the FEMA-440-DM used the acceleration spectra from the code, shown in Figures C.4-4, C.4-6 and C.4-8.

C5.0 PROTOTYPE MODELS

This section presents the details of the prototypes (i.e. models) used to represent the different lateral deformation resisting systems (LDRSs) covered in the 2nd Edition Bridging Guidelines. As outlined in Section C3.0, three different analyses (Quakesoft, CANNY and FEMA-440-DM) were conducted, and the differences between the models for these methods will be noted below.

Section C5.2 covers background information on the generic prototype modeling. Sections C5.3 through C5.7 provide details of the LDRS prototype models used in the analysis. Section C5.8 covers the prototype models used to generate the diaphragm resistance tables. General information about the prototype LDRSs, such as physical description, material specifications, and minimum detailing requirements are given in Commentary A, Sections A3.0 to A7.0.

These models are independent of the assigned Instability Drift Limits (ISDL). See Section B3.1 for a discussion of the ISDLs.

C5.1 Structural Models for LDRS

All LDRS prototype models are based on 2-storey structures with equal strength on both stories. A depiction of the structural model used for the Quakesoft analysis, and the majority of the CANNY analysis, is shown in Figure C.5-1. This two-dimensional model is comprised of a single degree of freedom (horizontal deformation), with masses lumped at the 2nd floor and the roof. The total weight of the building was divided into three parts. Two parts were placed at the 2nd floor and the third at the roof. For the resistance tables shown in Sections 3 to 8 in the 2nd Edition Bridging Guidelines, each clear storey height was 3 meters. Additional analysis was conducted using storey heights to derive the empirical storey height equations 3-1, 4-1, 5-1, 6-1, and 7-1 (see Section B3.5). Analyses of the rocking prototypes were done using a variety of storey height and centre of mass combinations to derive the equations 8-1 and 8-2.

While the structural model is simple, the force-deformation behaviour of each spring is specific and complex. Table C.5-1 summarizes the backbone curve, general hysteretic behaviour and other properties of the models. Both Quakesoft and CANNY used the same characteristics, with slightly different methods of implementation.

For moment frame prototypes (S-3, S-4, C-3, C-4, and C-5) CANNY used the frame structural model shown in Figure C.5-2, such that the P-delta influence was properly modelled. In the case of prototype S-3, the beam element was the inelastic element, and in the others the column element was inelastic. Each bay was 3 meters wide.

C5.1.1 Damping

The CANNY analysis used 5% Rayleigh damping in the first two modes, except for the rocking models (R-1, R-2 and R-3), which had 3% damping. The Quakesoft analysis uses a constant viscous damping of 5%.

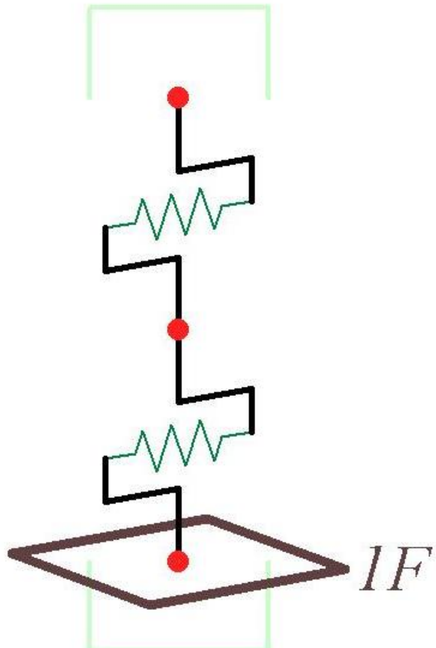


Figure C.5-1 Structural Model for Quakesoft and most CANNY Analysis

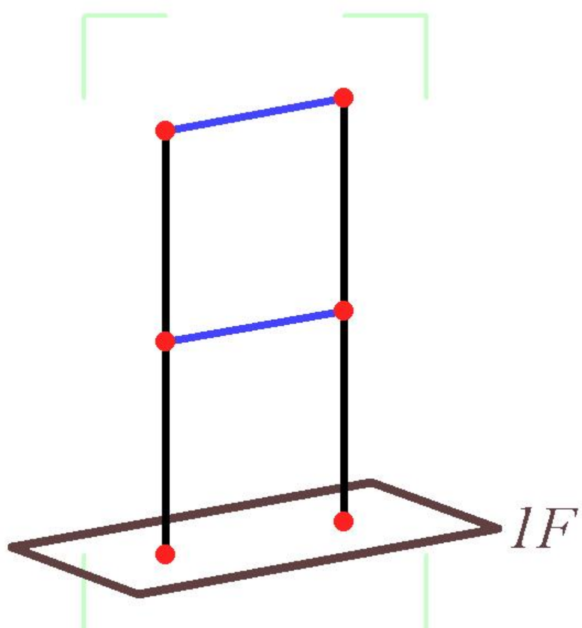


Figure C.5-2 Structural Model for CANNY Prototypes S-3, S-4, C-3, C-4, and C-5

Table C.5-1 Summary of LDRS Backbone, Hysteretic and other Properties

Material Group	Prototype No.	Prototype Description and Failure Mode	Backbone Curve Properties δ_c	F_c	δ_y	μ_Δ	δ_{zero}	ISDL	R_o	Hysteretic Properties
Wood	W-1	Blocked OSB/plywood shearwall	0.4%	0.7P	1.3%	3	10%	4.0%	1.7	Pinched/Stiff. Deg.
	W-2	Unblocked OSB/plywood shearwall	1.7%	0.9P	2.5%	-	10%	4.0%	1.7	Pinched/Stiff. Deg.
Steel	S-1	Concentric braced frame (tension only)	-	-	0.3%	-	-	4.0%	1.3	Slip
	S-2	Concentric braced frame (tension/compression)	-	-	0.3%	-	-	1-2.5%	1.3	Slip/Buckling
	S-3	Eccentric braced frame	-	-	0.5%	-	-	4.0%	1.5	Elastic-Plastic
	S-4	Moment frame (moderately ductile)	-	-	1.0%	-	-	4.0%	1.5	Elastic-Plastic
Concrete Masonry	M-1	In-plane unreinforced shearwall bed-joint sliding	-	-	0.1%	-	-	1.5%	1.5	Elastic-Plastic
	M-2	In-plane reinforced masonry	-	-	0.25%	-	-	1.5%	1.5	Stiffness Deg.
Reinforced Concrete	C-1	Shearwall (moderately ductile)	-	-	0.25%	-	-	2.0%	1.4	Stiffness Deg.
	C-2	Shearwall (conventional construction)	-	-	0.25%	-	-	1.5%	1.3	Stiffness Deg.
	C-3	Moment frame (ductile)	-	-	1.0%	-	-	4.0%	1.7	Stiffness Deg.
	C-4	Moment frame (moderately ductile)	-	-	1.0%	3	8%	4.0%	1.4	Stiffness Deg.
	C-5	Moment frame (non-ductile)	-	-	1.0%	1.5	5%	4.0%	1.3	Stiffness Deg.
Clay Brick Masonry	B-1	In-plane shearwall bed-joint sliding	-	-	0.1%	-	-	1.0%	1.5	Elastic-Plastic
Rocking	R-1	Low Aspect Ratio Rocking Element	-	-	0.15%	-	-	4.0%	1.0	Non-linear Elastic
	R-2	Medium Aspect Ratio Rocking Element	-	-	0.60%	-	-	4.0%	1.0	Non-linear Elastic
	R-3	High Aspect Ratio Rocking Element	-	-	1.20%	-	-	4.0%	1.0	Non-linear Elastic

Notes: δ_c - drift at "cracking"
δ_y - "yield" drift
δ_{zero} - drift at which strength is reduced to zero
F_c - "cracking" strength
μ_Δ - length of zero stiffness plateau relative to δ_y
P - full strength of material

C5.2 Background Information on Generic Prototype Models

This section discusses a number of miscellaneous topics associated with the non-linear analysis methods presented in this chapter.

C5.2.1 Idealization of Modeling

Non-linear analysis methods allow for an estimate of the inelastic deformations that occur in a structure to a given demand. For this estimate to have meaning, the mathematical models must adequately represent the structural systems being analyzed. However, real systems are highly complex, and do not have obvious elastic and inelastic components often assumed in engineering mechanics.

Figure C.5-3 shows a hypothetical force-deformation (or backbone) curve of a generic system loaded well beyond its maximum resistance. This figure also demonstrates how a system like this is approximated with, in this case, a bi-linear analysis model.

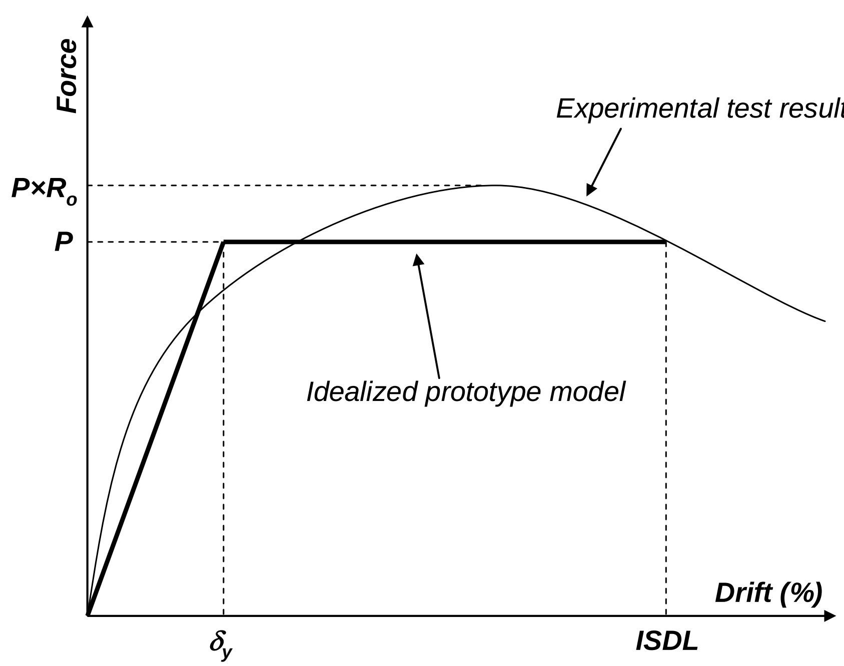


Figure C.5-3 Simplified Approach of Modeling a Non-linear System

Typically most real structures undergo a significant change in stiffness, even within their "elastic" range. This is often modeled with an equivalent, or secant, stiffness from zero deformation up to the "yield" displacement. The exact location of the "yield" point is based on a combination of the assumed strength and appropriate equivalent stiffness. It is not intended to reflect "first yield point" in shearwall systems. The ISDL is the maximum allowable drift that still meets the performance objectives. See Section B3.1 for information on ISDLs.

C5.2.2 Concept of Generic Prototypes

The prototypes in the 2nd Edition Bridging Guidelines represent the overall behaviour of generic low-rise structural systems. The prototype models the load-deformation response at each storey, which can be a combination of shear and/or flexure. Since they are low-rise systems, the shear response is a significant factor in the overall response.

The systems also assume that many of the properties for a given prototype will be the same, based on the typical designs in schools. These are: ISDL, yield drift, and to a certain extent, storey height (see Section B3.5). A constant yield drift means that the strength and stiffness are proportional. Aschheim (2002) demonstrates that the yield displacement is an excellent parameter for seismic design, and also shows that the yield displacement of many typical systems is independent of member size or the quantity of reinforcement.

While generic prototype models cover a wide range of typical systems found in schools, they may not be appropriate for all of them. See the prototype model descriptions in each section C5.3 to C5.7 for limits on the scope of their use.

C5.2.3 Period Ranges for Prototype Models

The period of the prototype models were a function of the assumed yield drift and strength, as these parameters define the associated stiffness and relative mass. Table C.5-2 lists the range in period for some of the two storey prototypes based on the range of strength values given in the resistance tables on for Seismic Zone 4. T_{short} corresponds to prototypes with the lowest drift limit, and T_{long} is for prototypes at the ISDL. As these periods are based on equivalent stiffnesses, they are longer than initial periods of undamaged structures (gross section properties) and those based on the empirical equations presented in the 2005 NBCC. These periods were calculated in CANNY, one of the non-linear dynamic analysis programs (Section C3.1) and were directly used for input into FEMA-440-DM the non-linear static analysis method (Section C3.2).

Table C.5-2 First Mode Initial Periods for Prototype Models

Prototype	T_{short} *sec*	T_{long} *sec*
W-1	0.26	0.57
W-2	0.26	0.72
S-1	0.20	0.51
S-2	0.21	0.45
S-3	0.27	0.49
S-4	0.34	0.69
M-1	0.16	0.28
M-2	0.21	0.35
C-1	0.21	0.37
C-3	0.38	1.02
C-4	0.37	0.98
C-5	0.40	0.75

C5.3 Wood Frame Prototypes

The wood-frame prototypes are based on experimental results from the EQ-99 project at UBC (EERF, 2006) and the CUREE Wood-frame project (CUREE, 2003).

Both of the wood frame prototypes are for shear walls. These prototypes represent walls designed and built to "standard" practice. See Section A3.5 for the material and connection specifications of these prototypes. Irregular designs not within the listed specifications (e.g. very small or very large nail spacing) may not qualify to use these prototypes, as it would influence factors such as the yield drift or the length of the yield plateau.

Each prototype has a different backbone curve (Figures C.5-4 to C.5-5), but share the same hysteretic rule (Figure C.5-6).

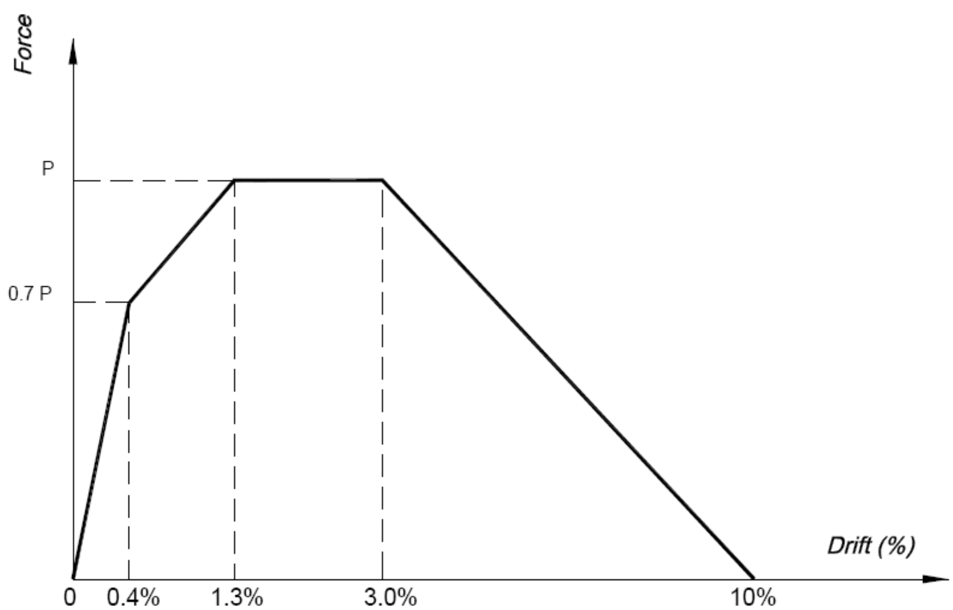


Figure C.5-4 Backbone Curve for Prototype W-1: Blocked OSB/Plywood

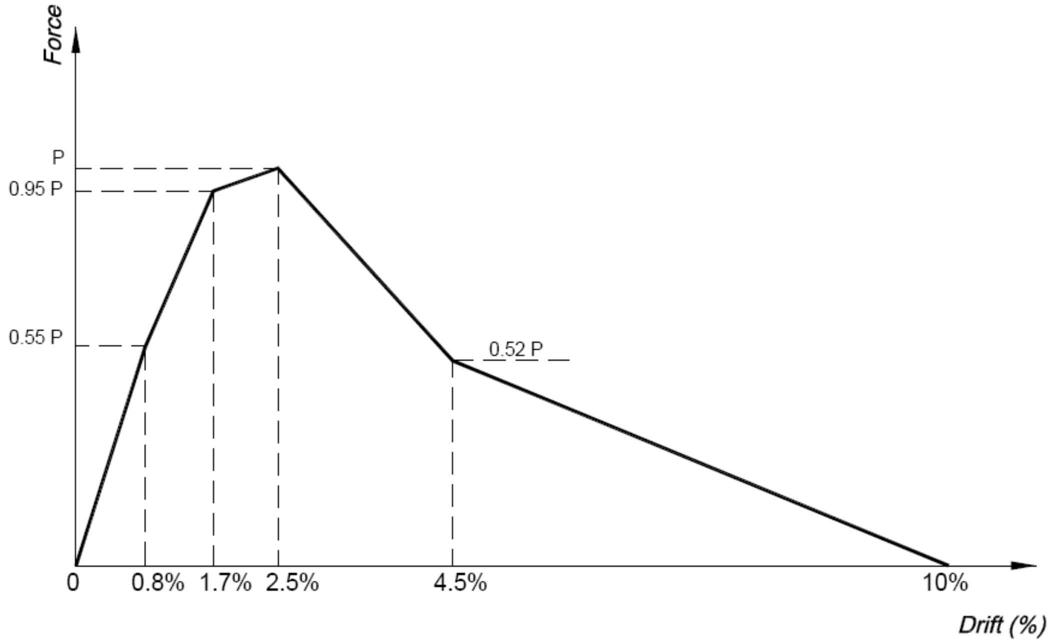


Figure C.5-5 Backbone Curve for Prototype W-2: Unblocked OSB/Plywood

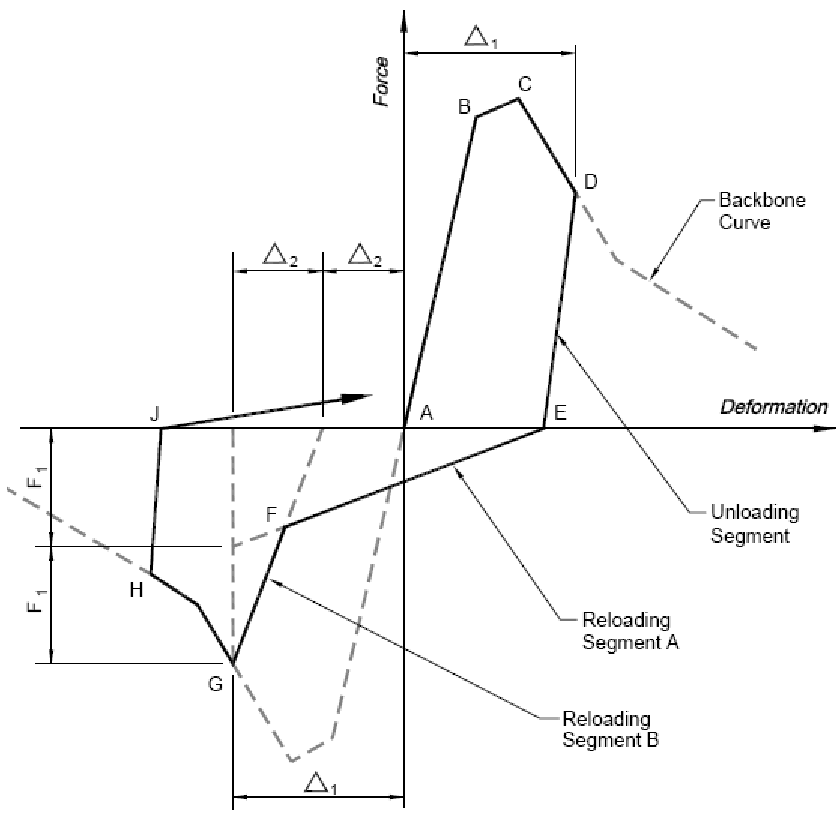


Figure C.5-6 Hysteretic Rules for Wood-frame Prototypes

C5.4 Steel Frame Prototypes

Steel prototypes (S-1, S-3, and S-4) were based on models commonly found in the literature (Saatcilglu and Humar, 2003). Model S-2 (tension/compression CBF) was based on the Jain-Goel model shown in FEMA 274 (ATC, 1997) (Figure C5-18).

Backbone curves for the steel prototypes are shown in Figures C.5-7 to Figure C.5-9. Hysteretic rules are shown in Figures C.5-10 to Figure C.5-13. Yield drifts for the frame systems (0.3%) are based on a brace angle of 45 degrees, and no significant strain-hardening is used in the models.

Prototype S-2 assumes that the compression strut has a maximum strength of 60% of the tension brace. The strength of the compression strut drops to 20% of its original capacity after buckling. In addition, the strength of prototype S-2 is based only on the strength of the tension strut.

A number of the modeling assumptions for the steel prototypes were investigated in a sensitivity study. Below is a brief summary and details are given C6.2.1.

- The response of the steel moment frame is influence by the assumed yield drift. Moment frames (Prototype S-4) with yield drifts between 0.8% and 1.2% are adequately represented by the S-4 table. Stiffer moment frames should use the tables for eccentric braced frames (S-3). More flexible moment frames are outside the scope of these guidelines.
- Prototype S-1 was not sensitive to either variations in the angle of the brace or to the level of strain hardening. These results should also be valid for prototype S-2.
- The sensitivity study demonstrated that the assumed strength of the compression strut (60% of the tension strut), in prototype S-2, gave reasonable results for compression strengths ranging from 30 to 90% of the tension strut.

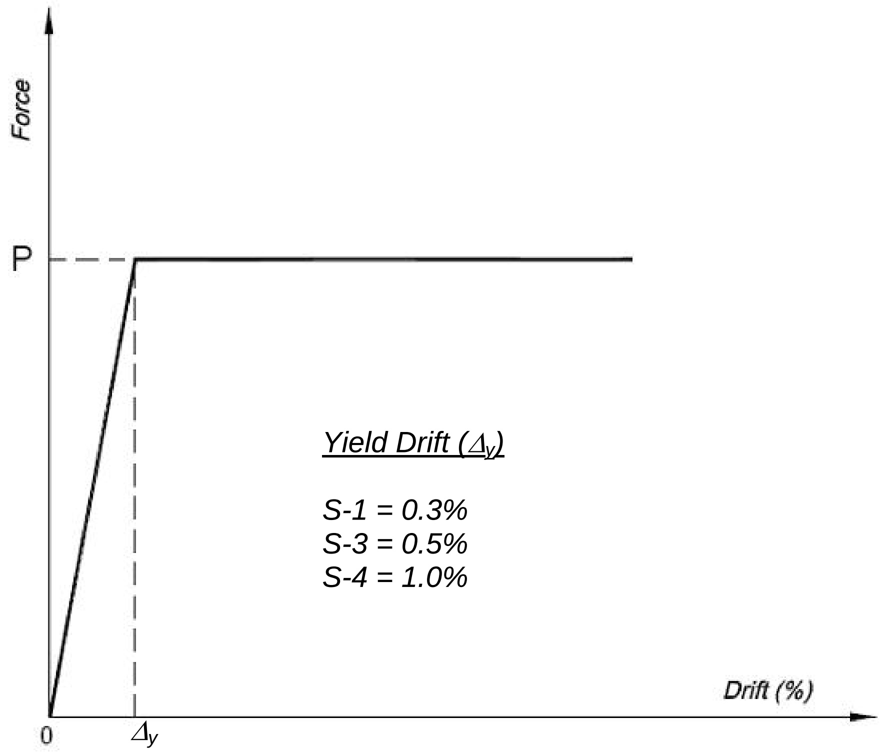


Figure C.5-7 Backbone Curve for Steel Prototypes S-1, S-3, and S-4

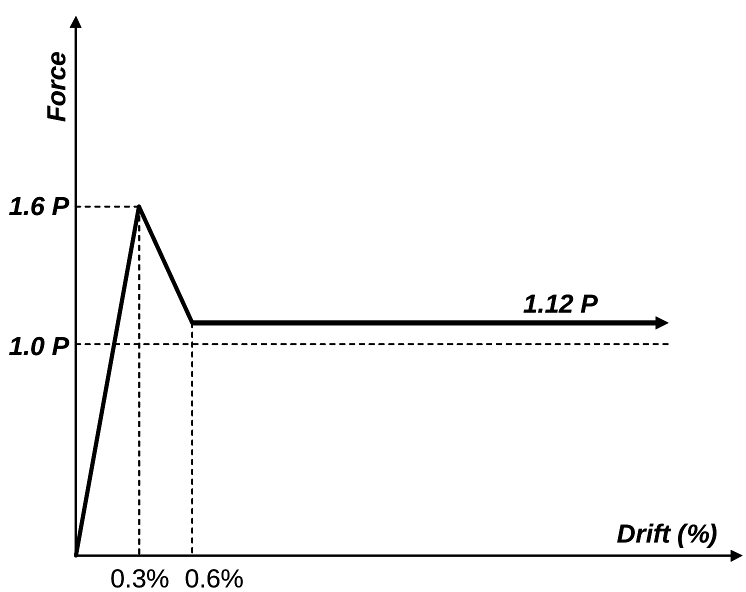


Figure C.5-8 Backbone Curve for Steel Prototype S-2

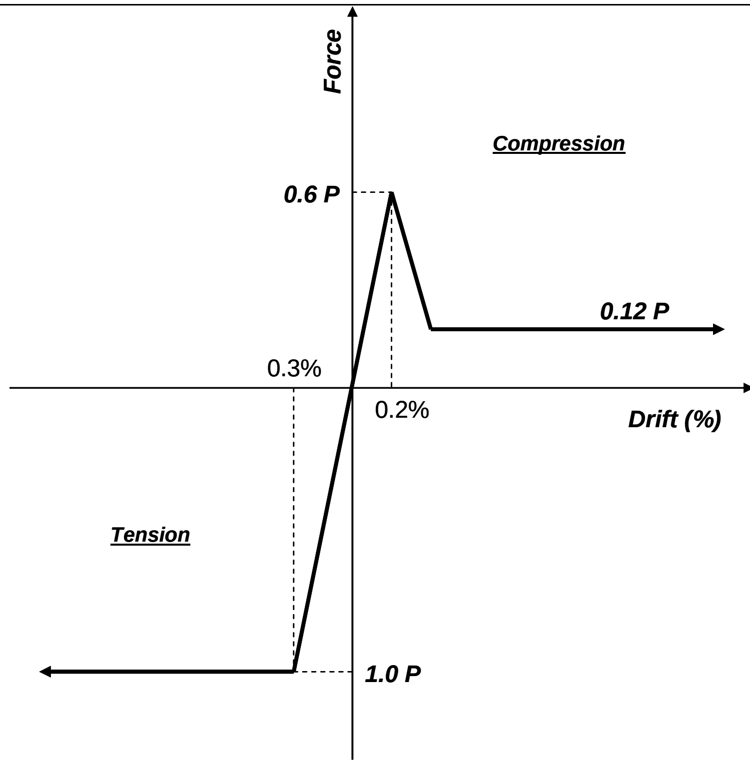


Figure C.5-9 Backbone Curve for Steel Prototype S-5

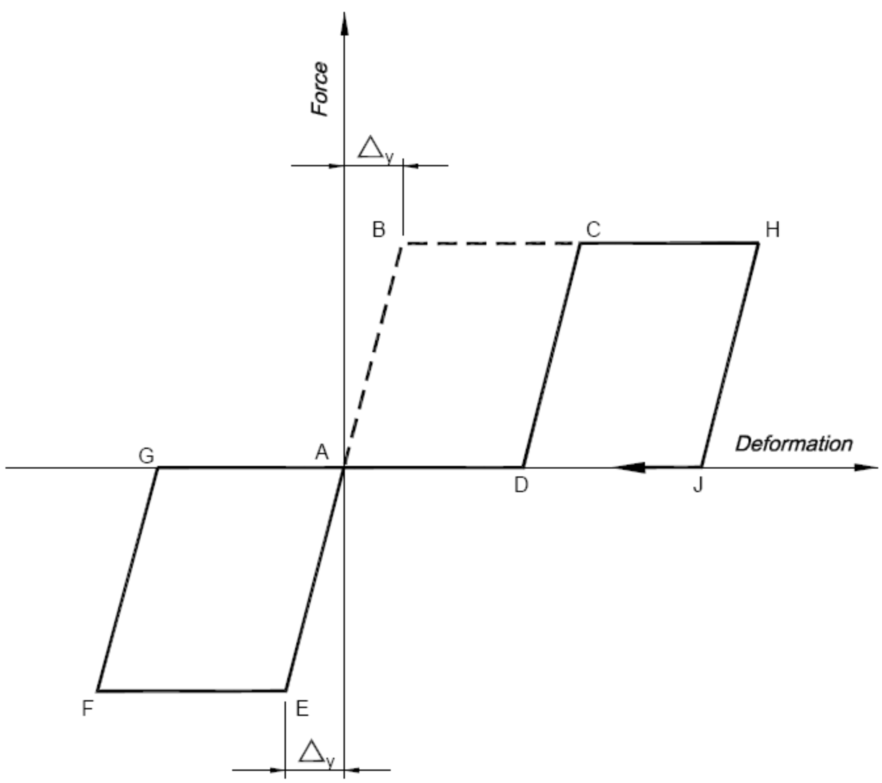


Figure C.5-10 Hysteretic Rules for Steel Prototype S-1

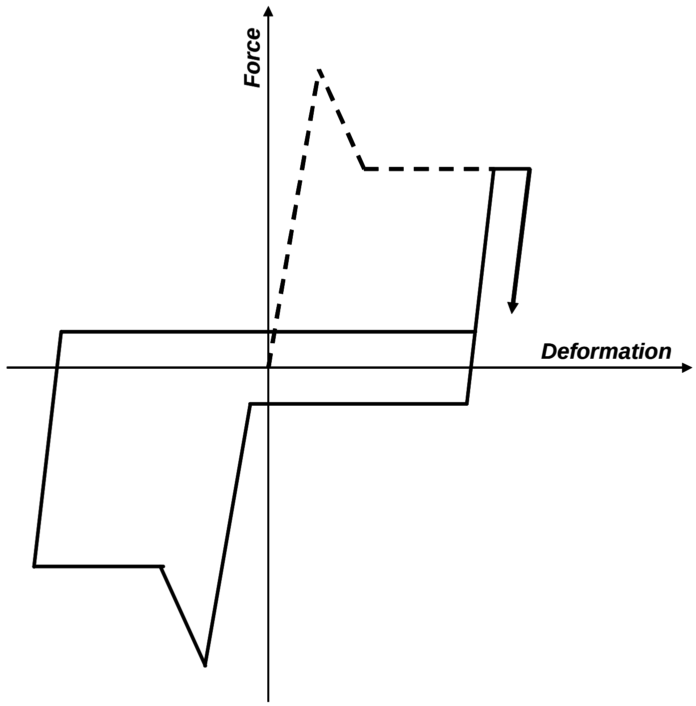


Figure C.5-11 Hysteretic Rules for Steel Prototype S-2

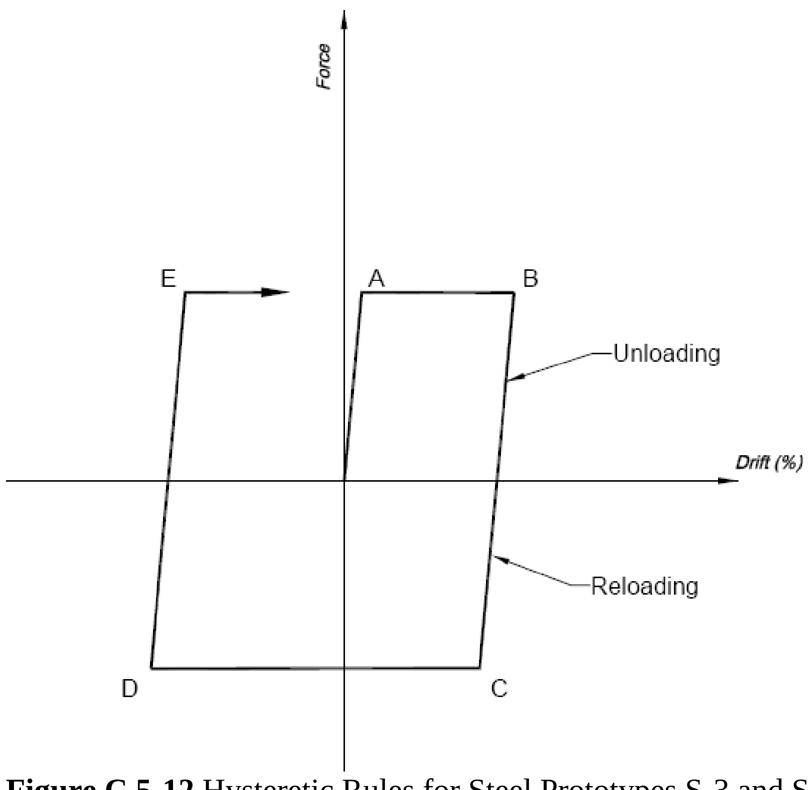


Figure C.5-12 Hysteretic Rules for Steel Prototypes S-3 and S-4

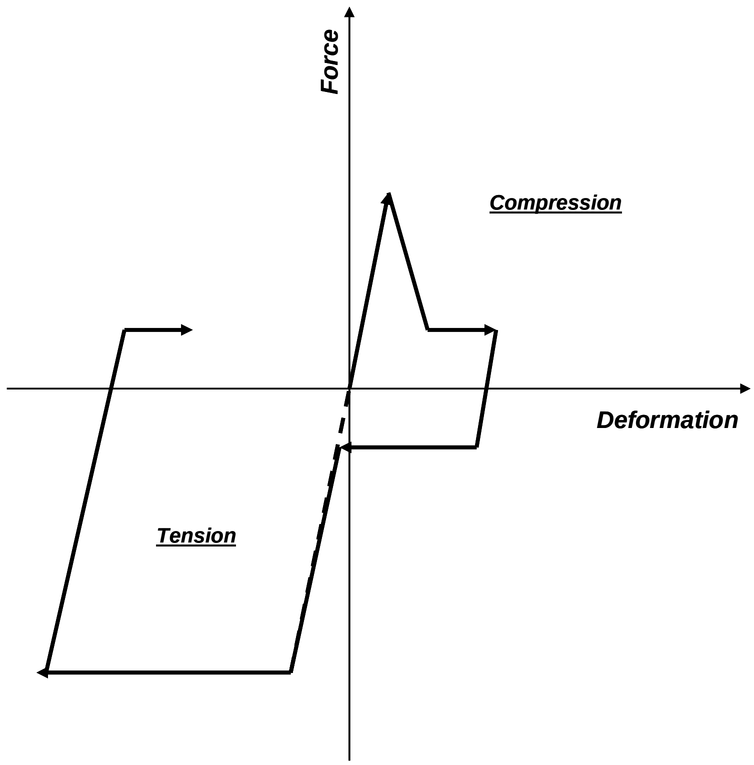


Figure C.5-13 Hysteretic Rules for Steel Prototype S-5

C5.5 Concrete Prototypes

The modified Clough model (Figure C.5-14) was used for the hysteretic properties of all of the reinforced concrete prototypes. This is a common model used in the literature to simulate the cyclic behaviour of reinforced concrete, as it incorporates significant stiffness degradation for reloading (Saatcilglu and Humar, 2003). The backbone curves (Figures C.5-15 through C.5-18) of the individual prototypes further distinguish the different types of reinforced concrete.

Prototypes C-1 and C-2 are for reinforced concrete shearwalls. The individual prototypes represent differences in reinforcement detailing, equivalent to the "moderately ductile" and a "conventional construction" as per the 2005 NBCC. See CSA-A23.3-04 for the detailing specifications. The models for the two prototypes are the same, but the resistance tables are different based on different ISDLs and values of R_o. As these prototypes represent low-rise squat shearwalls, the models account for a combination of inelastic and shear deformations. Regardless of the final mode of failure, flexure or shear, the same prototype model is used.

Prototypes C-3 through C-5 are for concrete moment frames with three different levels of detailing, see Section A5.1(3). There are significant differences in the backbone curves for the different concrete moment frames.

For all of the reinforced concrete models, the resistance must be calculated using a code-based approach. However, the prototype models presented here do not account for deficiencies that may be present in some existing LDRSs (e.g. insufficient development length). Theses must be accounted for in the calculation of the resistance of the LDRS. See Section B7.3 for guidance.

Many reinforced concrete shearwalls may rock on their foundations. Instead of implementing multiple prototypes to account for different levels of rocking, the shearwall prototypes can be combined using the Toolbox Method (see Section B3.2). The rocking model is discussed in Section C5.8.

The concrete prototype sensitivity study in Section C6.2.2 investigates the influence of the assumed yield drift for both the concrete wall (C-1 and C-2) models, as well as the yield drift for the non-ductile concrete frame model (C-5).

- The study shows that the models (C-1 and C-2) used here are conservative for walls with yield drifts less than 0.25%, but unconservative for more flexible walls. This limits the aspect ratio of the walls to 4 for cantilever and 8 for piers, because of the relationship between yield drift and aspect ratio (see Section B7.4).
- The influence on yield drift for concrete moment frames with yield drifts was also investigated. The results indicate that the prototype C-5 adequately models frames with a yield drift of 1% or less, but is unconservative for more flexible frame systems.

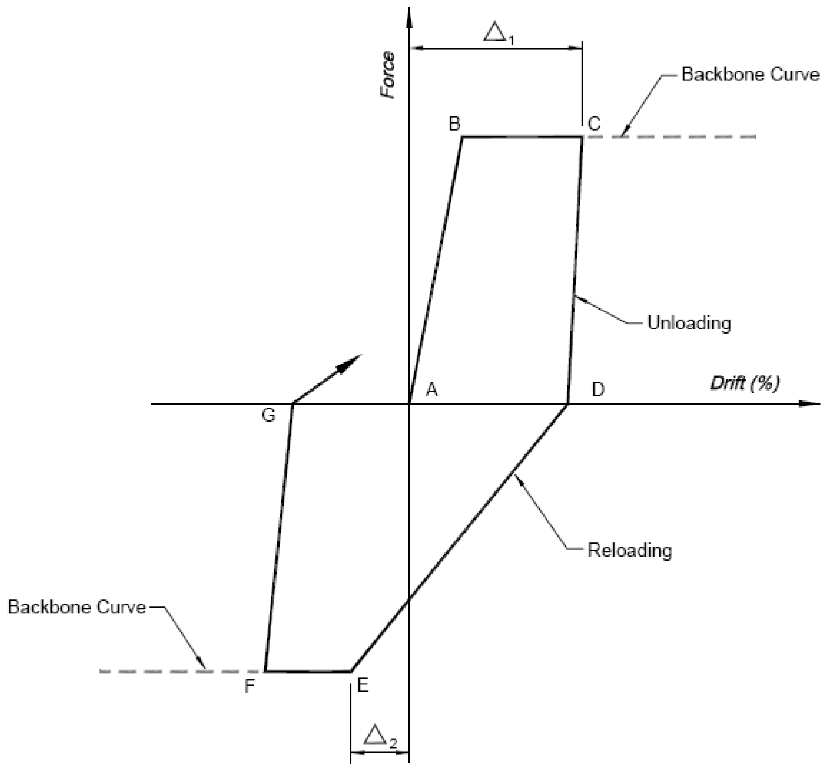


Figure C.5-14 Hysteretic Rules for All Reinforced Concrete Prototypes

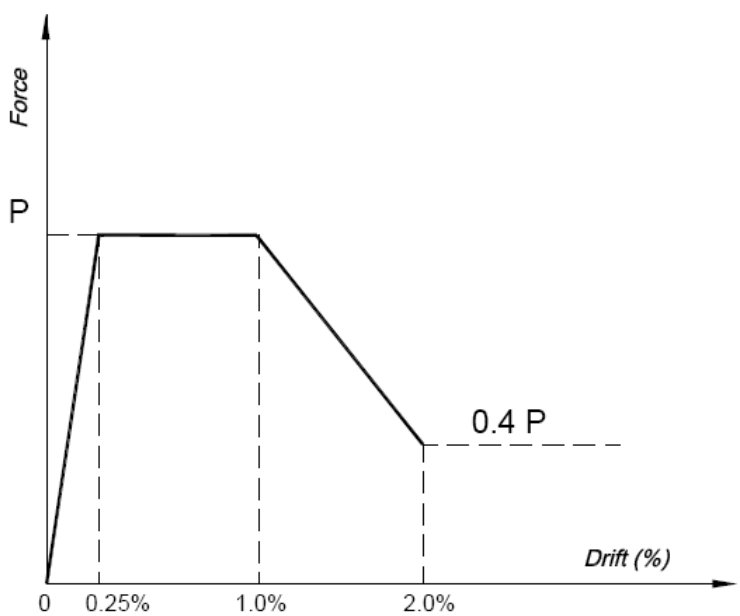


Figure C.5-15 Backbone Curve for Concrete Wall Prototypes C-1 and C-2

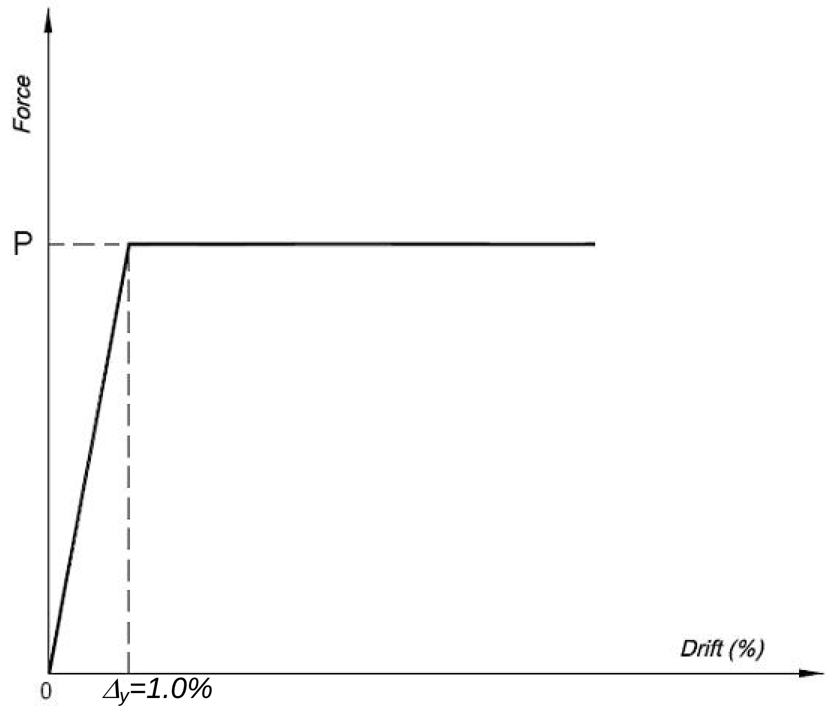


Figure C.5-16 Backbone Curve for Reinforced Concrete Prototype C-3

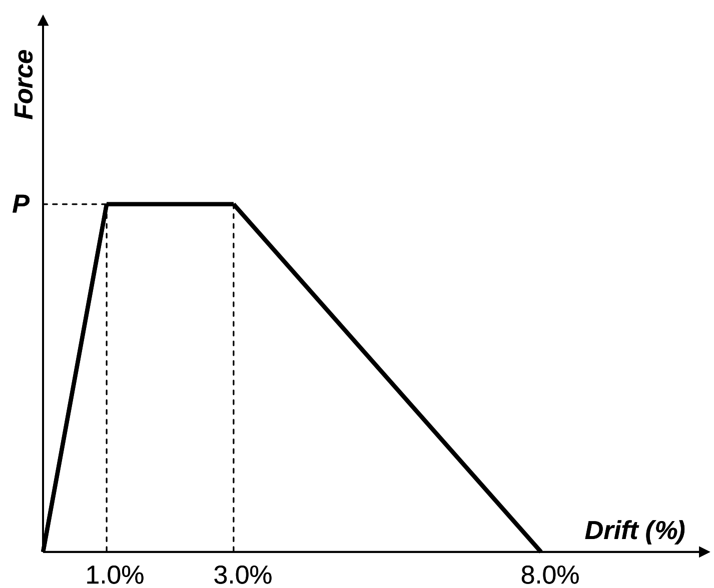


Figure C.5-17 Backbone Curve for Reinforced Concrete Prototype C-4

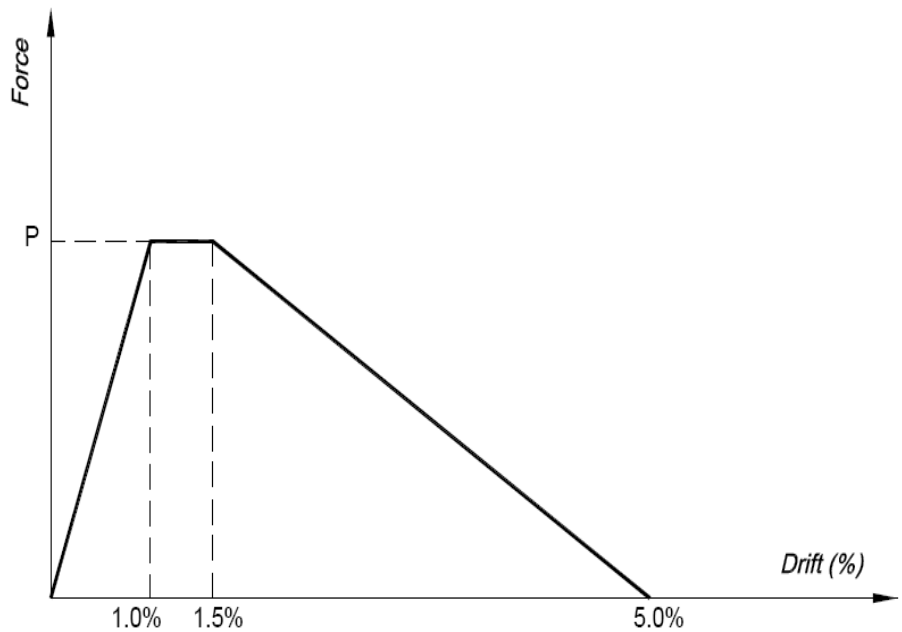


Figure C.5-18 Backbone Curve for Reinforced Concrete Prototype C-5

C5.6 Masonry Prototypes

This section covers the models for both unreinforced masonry (concrete and clay brick) as well as reinforced concrete masonry. Backbone curves are shown in Figures C.5-19 and C.5-20. Hysteretic rules are shown in Figures C.5-21 and C.5-22.

Unreinforced masonry walls can fail in shear, rocking or toe-crushing (FEMA 274 (ATC, 1997)). The exact mode of failure depends on aspect ratio and dead-load. Models for shear (M-1 for concrete masonry and B-1 for clay brick masonry) and rocking (see Section C5.7)) are provided. Toe-crushing is not explicitly modeled. This failure mode is defaulted to a rocking behaviour, which is more conservative.

Prototypes M-1 and B-1 are for the shear failure of unreinforced concrete/clay brick masonry walls, typically with an aspect ratio of 2/3 (H/L) or less. The model is based on the bed-joint sliding model shown in Shing and Klingner (1998). While the prototype specifically models bed-joint sliding, it is also accounts for diagonal tension cracking. For most URM walls, failure by bed-joint sliding governs, as it has a lower capacity than diagonal tension cracking. Calculating the diagonal tension cracking resistance requires a reasonable estimate of the mortar strength and the strength of the concrete masonry units, which would add considerable complication to the assessment of existing masonry walls. For simplicity, only the bed-joint sliding model is used and the calculation of its resistance (Equation 6-2) conservatively does not include any benefit for mortar strength.

M-1 and B-1 have the same model, but are distinguished with different ISDLs in the resistance tables.

Prototype M-2 is for reinforced masonry in both flexure and shear. The model for this prototype is the same as the prototype for concrete shear walls (C-1 and C-2), but has its own overstrength factor (R_o) and ISDL.

The sensitivity study of masonry models is given in Section C6.2.3.

- This sensitivity study looks at the assumption of the “full” hysteretic model used in the bed-joint sliding model (M-1 and B-1). The study shows that using a less robust hysteretic loop does not significantly affect the results. This indicates that there is no significant error in using a bed-joint sliding model (BL2) where diagonal tension cracking (CL2) model would be more appropriate.

The conclusions of the sensitivity study on the limitations of the concrete shear wall models (C-1 and C-2) also apply to the reinforced masonry prototype (M-2). See Section C5.5.

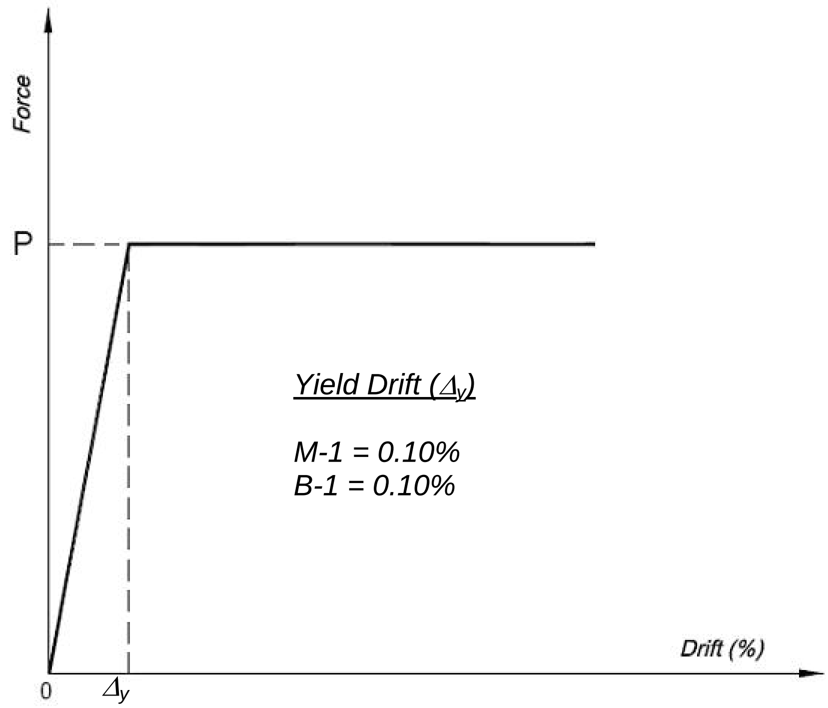


Figure C.5-19 Backbone Curve for Masonry Prototypes M-1 and B-1

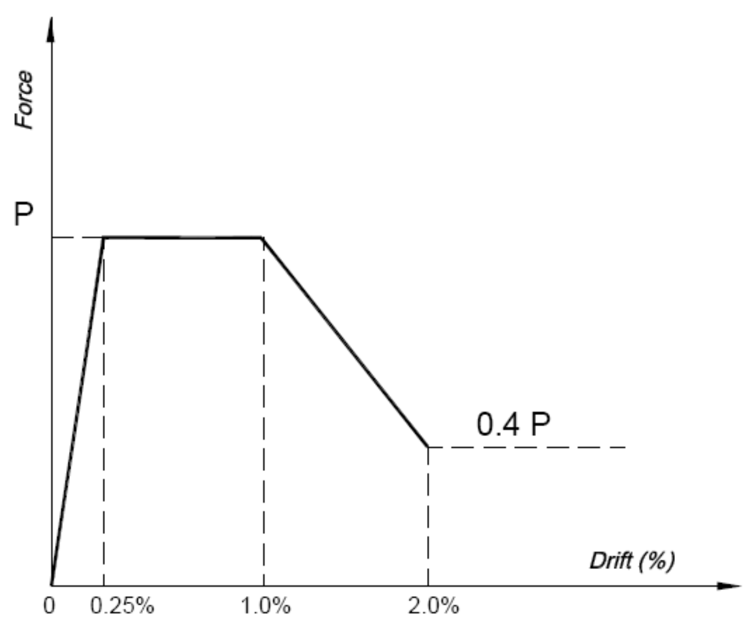


Figure C.5-20 Backbone Curve for Masonry Prototype M-2

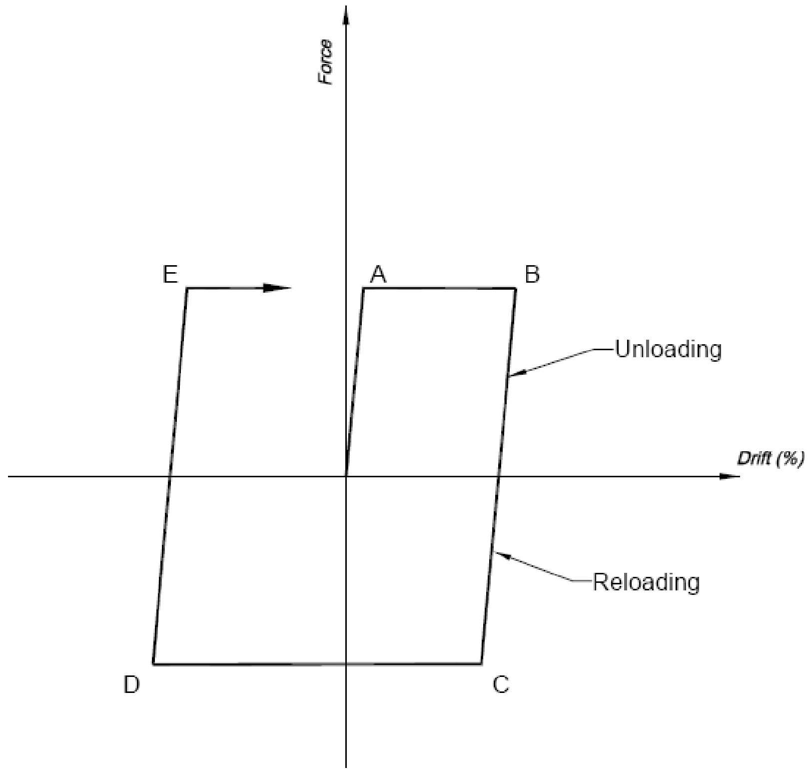


Figure C.5-21 Hysteretic Rules for Masonry Prototypes M-1 and B-1

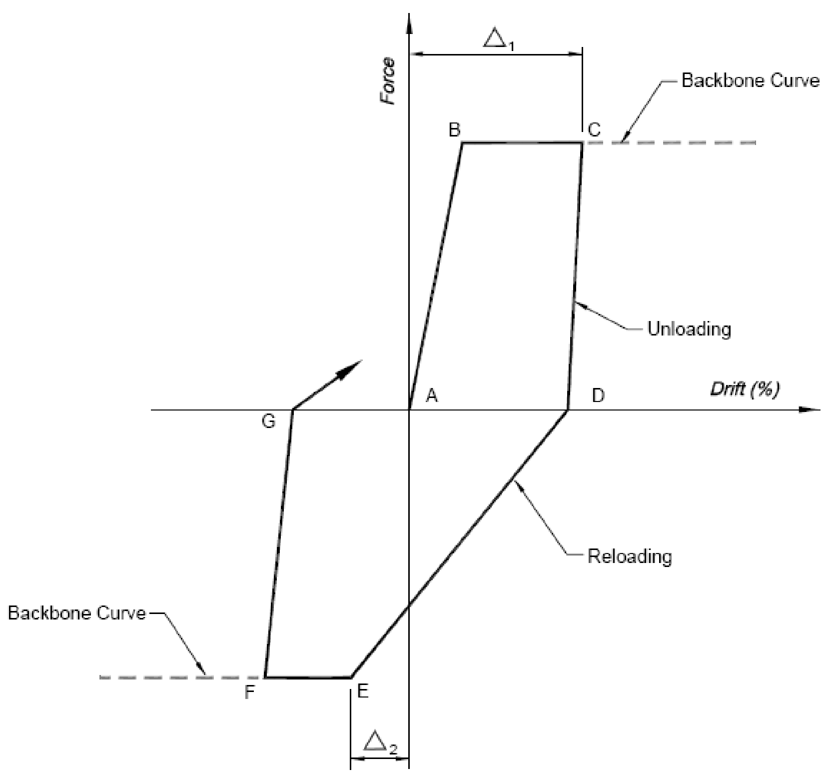


Figure C.5-22 Hysteretic Rules for Masonry Prototype M-2

C5.7 Rocking Element Prototypes

The rocking element models (R-1, R-2 and R-3) can be used for any system that rocks on its foundation. The most common systems that rock are concrete or masonry walls or piers, typically with aspect ratios (H/L) of 1 or more. Rocking can be the sole response of these systems, but often it is a combination with flexure with reinforced concrete or masonry walls. The backbone and hysteretic properties (shown in Figures C.5-23 and C.5-24) are those presented by Erbay and Abrams, 2002.

The assumed "yield drift" of rocking elements is controlled by the aspect ratio (H/L) of the rocking element (see Section B10.1), and the stiffness of that element. Prototype R-1 has a yield drift of 0.15% and is intended to cover the rocking behaviour of stiff walls with an aspect ratio of up to 1 for cantilevers and 2 for piers. Prototype R-2 has a yield drift of 0.6% and is intended to simulate the response of stiff rocking elements with aspect ratios of 2.5 for cantilevers and 5 for piers. Prototype R-3 has a yield drift of 1.2% and is applicable for stiff cantilevers with an aspect ratio of 4 and stiff piers with an aspect ratio of 8. Stiff systems include masonry or concrete walls. More flexible systems have a higher yield drift for a given aspect ratio than a stiff wall. These systems (wood and steel) when governed by rocking, should not use R-1. See Section A8.4 for guidance.

The resistance tables presented for rocking elements are appropriate for 3m tall pier rocking elements in a 1-3 storey building, or single cantilever rocking element with the centre of mass of the building located 3m above the ground. For different a taller storey height (up to 3m) use Equation 8-1 (see Section B3.5). For a taller cantilever wall building with a higher centre of mass use Equation 8-1, and for a lower centre of gravity use Equation 8-2. See Section B10.2 for the influence on the height of the centre of mass.

The sensitivity study for the rocking elements was an integral part of the development of the rocking prototypes. Initially only one rocking element (R-1) was included in the Guidelines, but because the sensitivity study indicated that the rocking model was very sensitive to the assume yield drift, the models R-2 and R-3 were added. See Section C6.2.3 for details on the influence of yield drift on the rocking models.

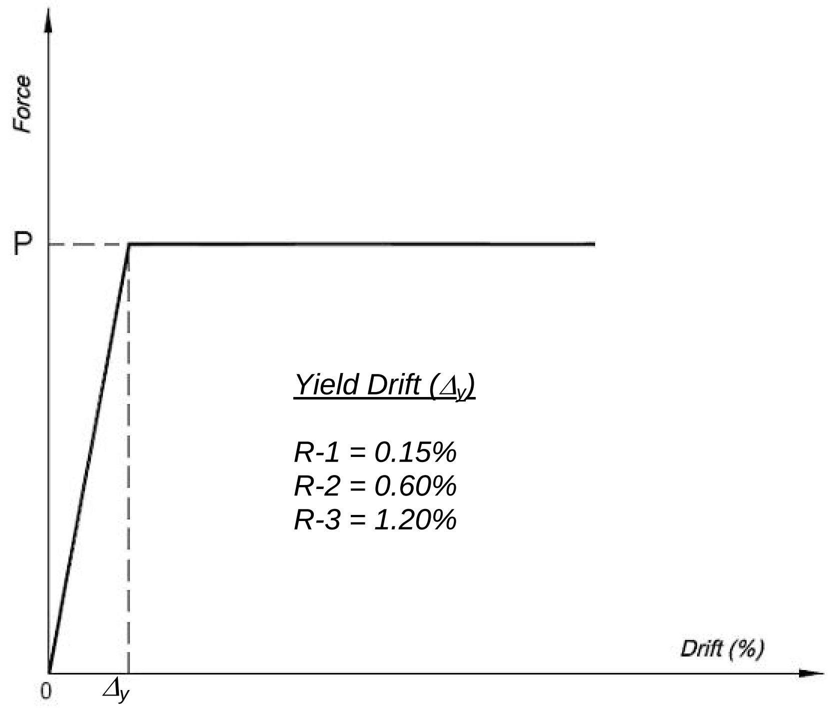


Figure C.5-23 Backbone Curve for Rocking Element Prototypes

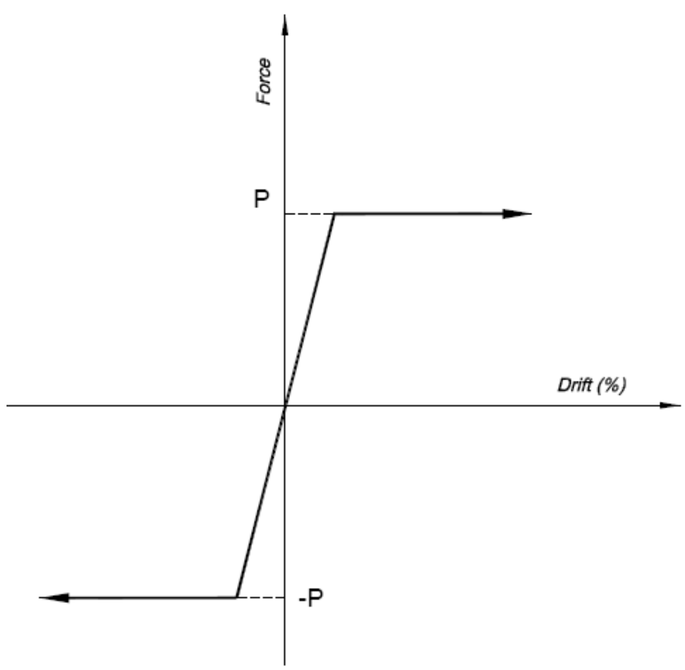


Figure C.5-24 Hysteretic Rules for All Rocking Prototypes

C5.8 Diaphragm Prototypes

Non-linear dynamic analysis was used to generate and validate the values given in the diaphragm resistance tables (Section 10 of the Bridging Guidelines). Quakesoft was used to generate the values, and CANNY was used to validate (see Section C6.1.5). The same suite of ground motions was used in the diaphragm analysis as the LDRS analysis. Non-linear static analysis was not used for the diaphragm analysis.

C5.7.1 Diaphragm Structural Models

The diaphragm structural models were more complex than the models for the LDRS prototypes. The "Class 1" diaphragm model, shown in Figure C.5-25, is comprised of multiple diaphragm elements spanned between two end walls. Both the walls and diaphragm are in plane with the direction of shaking. This model represents a unit strip of the diaphragm and end walls, and not the entire structure. This type of model was used in cases where the end walls were relatively flexible and represented wood or steel construction.

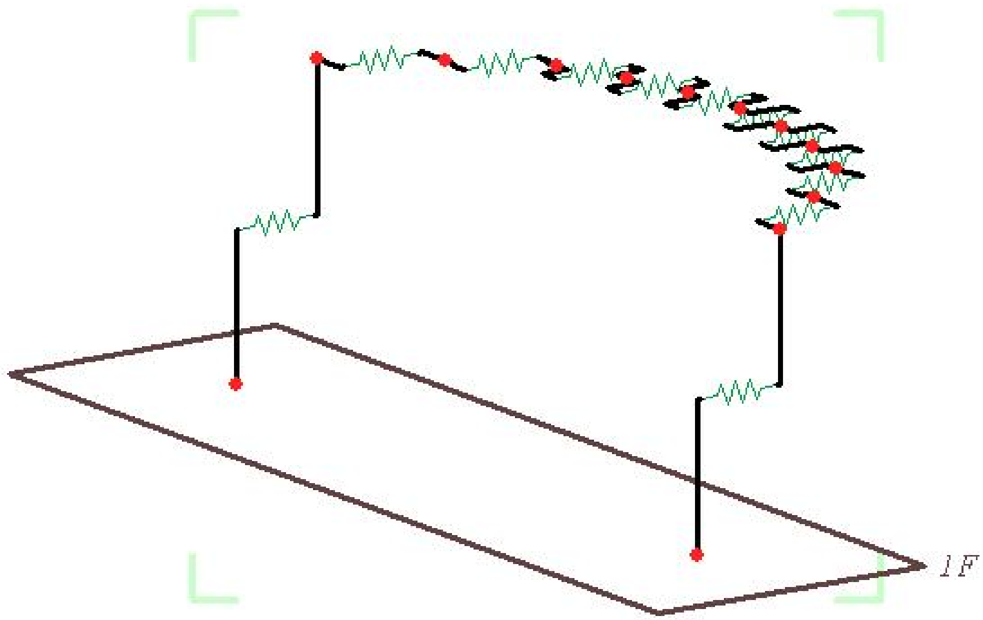


Figure C.5-25 Class 1 Diaphragm Structural Model

The end walls are one-storey LDRS prototypes. Their properties are the same as the LDRS presented in Sections C5.3 through C5.7. The diaphragm elements are non-linear shear spring models (no flexural degree of freedom). Each element was 2.5m in length, which means the number of elements varied with the length of the diaphragm. This length was used as it was a reasonable estimate of the distance between major roof support members. It was also necessary to maintain a constant element length to ensure a consistent estimate of the inelastic shear strains.

The mass was lumped between diaphragm elements, and was proportional to the tributary area of the diaphragm. The mass above the end walls included the mass of the upper half of the wall.

The other diaphragm structural model used was the “Class 2” model, which accounted for the out-of-plane response of the exterior walls running perpendicular to the in-plane end walls. The Class 2 model was used for masonry LDRSs, and is shown in Figure C.5-26.

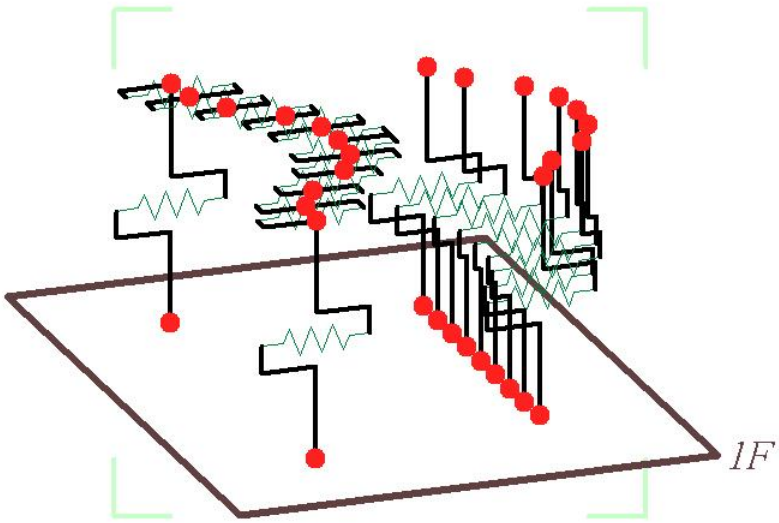


Figure C.5-26 Class 2 Diaphragm Structural Model

This model represents the approximately half a building, accounting for the rocking restraint of only one out-of-plane wall. One rocking element was included for each lumped mass in the diaphragm. Additional mass was added to the top of each rocking element equal to half the weight of the wall segment. Dead load proportional to the tributary area of the diaphragm was applied to each of the rocking elements. The Class 2 model was used to represent relatively stiff end walls related to unreinforced masonry construction.

Preliminary analyses indicated that the demands on the diaphragm prototypes were always higher on the Class 2 model. This is because the stiff walls forced more of the deformation into the diaphragm itself, and not the walls. It was felt that including diaphragm prototypes for both stiff (masonry and concrete) and flexible (wood frame or steel) walls resulted in too many different prototypes; the more conservative of the two was used (i.e. Class 2 – stiff walls with rocking resistance).

A structural model using reinforced concrete and/or reinforced masonry walls was not studied, as it was felt that the out-of-plane restraint of the reinforced walls would be less conservative than the rocking restraint of the Class 2 model.

C5.7.2 Description of Diaphragm Prototypes

Using the Class 2 model above, a total of six diaphragm prototypes were modeled. Each prototype has a unique diaphragm construction. The diaphragm prototypes are summarized on Table C.5-3.

Table C.5-3 Diaphragm Prototype Summary

Diaphragm Prototype	Diaphragm Material/Construction	R_o	Diaphragm Inelatic Strain Limit (%)
D-1	Blocked Plywood/OSB	1.7	2.5%
D-2	Unblocked Plywood/OSB	1.7	2.5%
D-3	Steel Deck - Type A	1.67	1.0%
D-4	Steel Deck - Type B	1.67	0.5%
D-5	Steel Braced Frame (Tension Only)	1.3	3.0%
D-6	Steel Braced Frame (Tension/Compression)	1.3	3.0%

The backbone and hysteretic properties of the different diaphragm material types are presented in Sections C5.7.4 through C5.7.6.

The end wall construction was based on the prototype model M-1 (see Section C5.6). The strength was selected to conform to the level specified in the resistance tables for retrofit. The out-of-plane rocking elements used the R-2 rocking model (see Section C5.7), with a strength based on the overturning capacity of a typical out-of-plane concrete masonry wall with adequate surcharge to meet the out-of-plane requirements of these guidelines (see Section 9.0).

C5.7.3 Performance Criteria for Diaphragms

The resistance tables for the diaphragms list capacity (shear strength) as a portion of the weight of the diaphragm for a given diaphragm span, and diaphragm prototype (D-1 through D-6). Unlike the LDRS resistance tables, the performance criteria (i.e. maximum drift) are not explicitly listed. See Section B11.0 for the performance criteria to which the resistance tables were calculated.

C5.7.4 Wood Diaphragms

Wood diaphragms are modeled in the same way as the wood-frame LDRS prototypes. The blocked plywood or OSB diaphragms are equivalent to LDRS prototype W-1, and the unblocked versions are the same as LDRS prototype W-2. See Section C5.2.

Contributions from other sheathing materials, such as drywall or GWB, are not considered.

C5.7.5 Steel Deck Diaphragms

Two types of steel deck are considered. The distinction is based on the quality of the connection made by the fasteners. Section A10.1 specifies what requirements are needed to qualify the steel deck to use the Type A tables. Type B is the default.

Two sources from the literature were used to develop the models and limits for steel deck diaphragms. These were Essa et al. (2003) and Tremblay et al. (2004).

Steel decking Type A has the superior form of fasteners (screwed side laps and nailed or welded-with-washer deck-to-frame connectors). An inelastic strain limit value of 1.0% is recommended. This type of decking behaves elastically up to 1.0% shear strain. Significant degradation occurs at 2.0% shear strain. The hysteretic behaviour was modeled as shown in Figure C.5-27.

Steel decking Type B has the inferior form of fasteners (button-punched or welded side laps and welded-without-washer deck-to-frame connectors). The inelastic value of 0.5% for this type of steel deck is recommended. This type of decking behaves elastically up to 0.5% shear strain and exhibits total degradation at 1.0% shear strain. The hysteretic behaviour was modeled as shown in Figure C.5-28.

C5.7.6 Steel Braced Frame Diaphragms

Horizontal Steel Braced diaphragms are modeled in the same way as the steel frame LDRS prototype S-1 (tension only) and S-2 (tension/compression steel braced frame). See Section C5.4.

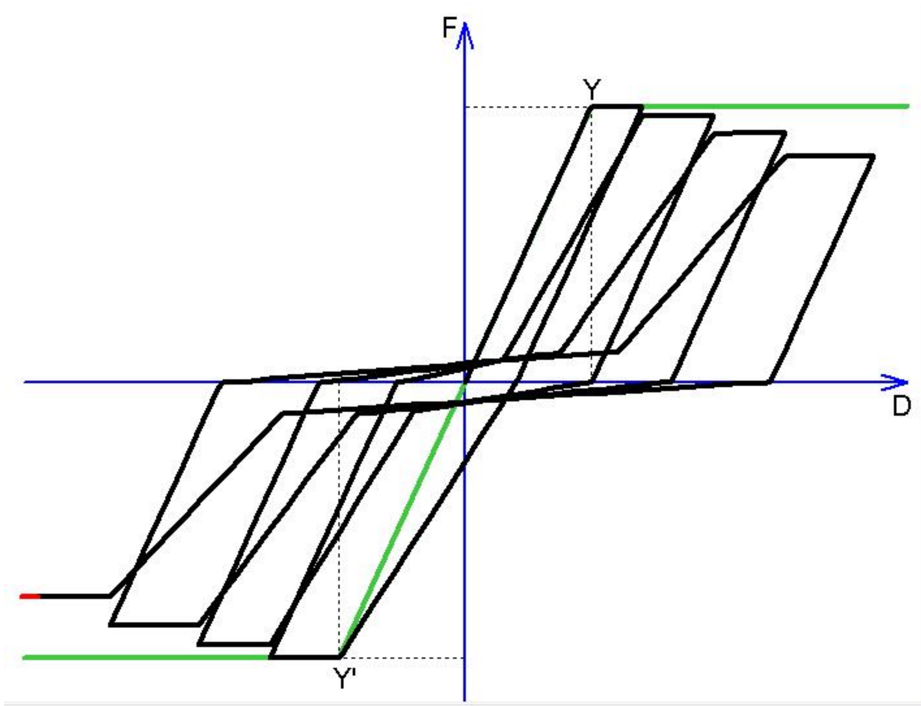


Figure C.5-27 Hysteretic Model for Steel Deck Diaphragm Type A

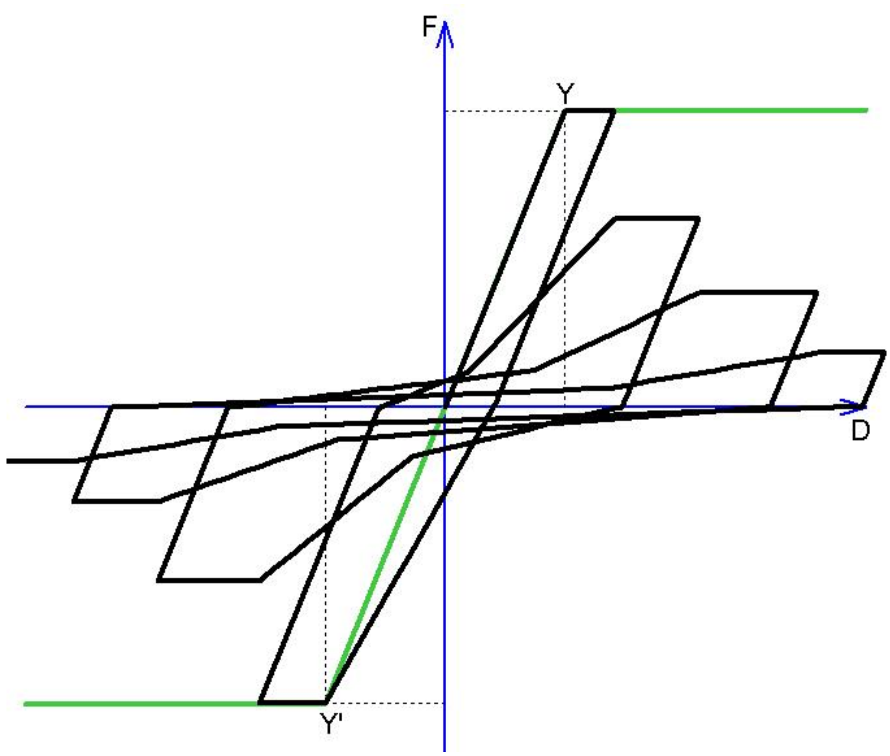


Figure C.5-28 Hysteretic Model for Steel Deck Diaphragm Type B

C6.0 VALIDATION OF RESULTS

C6.1 Comparison of Different Analysis Methods

Results for all of the prototypes, with unique models, for the three types of nonlinear analysis are shown in this section. The values are for Vancouver (Seismic Zone 4) on Site Class C, based on the 2005 National Building Code of Canada (NBCC). Each value shows the strength (as a percentage of the seismic weight of the structure) to limit the drift to the specified level. All calculated strengths were divided by R_o for the system to establish the factored resistances. All resistance tables are for 3 metre storey heights.

The data is provided both in tables and plots. There are two lines shown for both the Quakesoft and CANNY analysis. These are the "Mean + 1s" (the Retrofit level) and the "Mean" (roughly equivalent to the Assessment levels). These are respectively the mean plus one standard deviation and the mean of the results from the suite of 10 ground motions. The Quakesoft analysis at the Mean + 1s level were used as the final numbers in the 2nd Edition Bridging Guidelines.

The FEMA-440-DM analysis was only used to generate values corresponding to the "Mean" value of the ground motion records. The "Mean + 1s" level was not validated with this method, as it was difficult to replicate the standard deviation which was a product of analyzing with a suite of ground motions.

In cases where values are missing from the tables it indicates:

- The drift was in excess of the ISDL, and thus not calculated
- The system was very flexible and determining the value for a low drift was difficult because of a high standard deviation in the results of the suite

The plots also include the 60% and 100% Code line, which is the lower-bound for the retrofit strengths in the 2nd Edition Bridging Guidelines (see Section C2.2). The 50% line used in lieu of the 60% line for prototypes W-2, M-1 and R-1. It should be noted that the equivalent code value is based on the static strength calculation alone, and does not incorporate any check on the drift. The reason why the resistance tables, in the 2nd Edition Bridging Guidelines, show an increase strength demand for lower drifts is because the stiffness in strength are proportional, and more stiffness (and stiffness) is needed to limit the elastic (and inelastic) drifts. Code based designs could need additional stiffening to meet the same drift levels.

For each prototype a statistical comparison of the calculated values is done by observing the coefficient of variation (C_v), also known as the relative standard deviation.

$$C_v = \frac{\sigma}{\mu} \qquad \text{(C6-1)}$$

Where σ is the standard deviation and μ is the mean of a set of values.

Two C_v are reported for each prototype. One is for the analysis methods using the "Mean + 1s" and the other for the values from the "Mean". For this type of analysis C_vs below 10% are considered a good match.

An example of a calculation of the C_v for the "Mean" values of Prototype W-1 are given in Table C.6-0.

Table C.6-0 W-1 Blocked OSB, Seismic Zone 4, Site Class C

Interstory Drift (%h)	**R_m (%W)** *Average*	*Std. Dev.*	C_v
1.0%	26%	1.35%	5%
1.5%	18%	0.63%	4%
2.0%	13%	1.19%	9%
2.5%	9%	0.24%	3%
3.0%	7%	0.69%	10%
4.0%	6%	0.49%	9%
		Average	***7%***

C6.1.1 Wood Frame Prototypes

Table C.6-1 and Figure C.6-1 compare the calculated strengths for prototype W-1. The C_v for the *Mean + 1σ* and *Mean* are both 7%, indicating a good match. The 8%*W* retrofit value at the ISDL is higher than the 60% code level.

Table C.6-1 W-1 Blocked OSB, Seismic Zone 4, Site Class C

Interstory Drift (%h)	Quakesoft *Mean + 1 σ*	Quakesoft *Mean*	CANNY *Mean + 1 σ*	CANNY *Mean*	FEMA-440 *Mean*
1.0%	33%	27%	29%	25%	27%
1.5%	22%	18%	23%	18%	17%
2.0%	16%	12%	17%	14%	12%
2.5%	12%	9%	14%	9%	9%
3.0%	10%	7%	9%	8%	6%
4.0%	8%	6%	7%	5%	6%

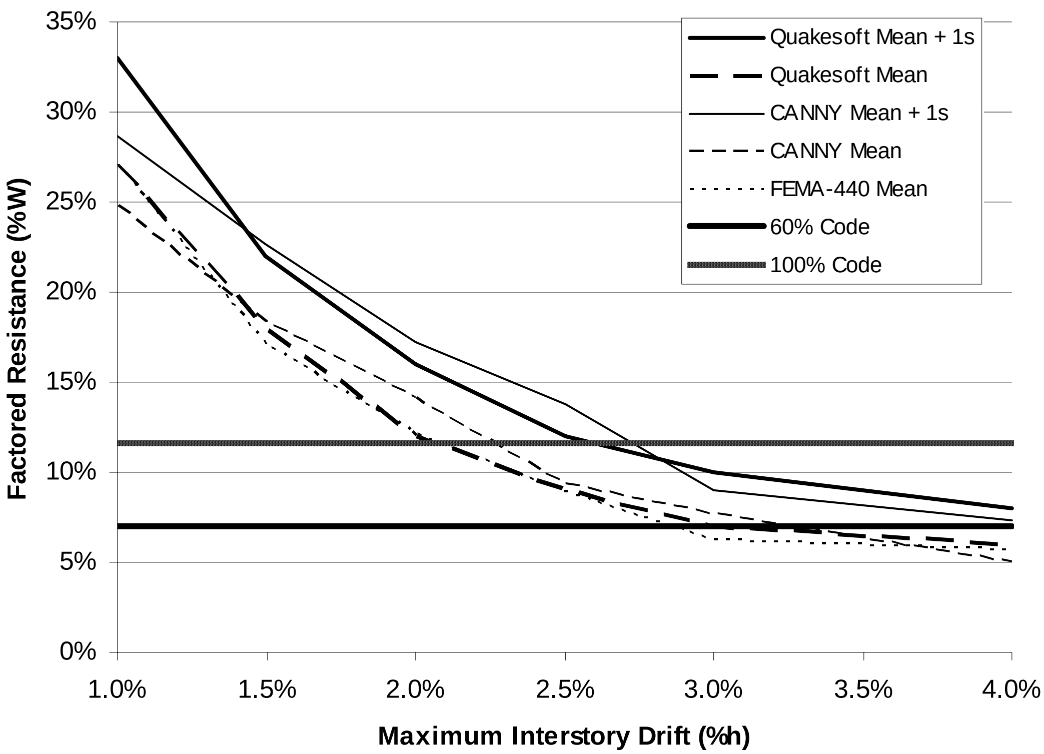


Figure C.6-1 W-1 Blocked OSB, Seismic Zone 4, Site Class C

Table C.6-2 and Figure C.6-2 compare the calculated strengths for prototype W-2. The C_v for the *Mean + 1*σ and *Mean* are 15% and 14% respectively. This is not a particularly good match, with the CANNY results significantly higher than the other results in the drift ranges from 2 to 3%. The retrofit strength of 9%*W* is just equal to the 50% code line.

Table C.6-2 W-2 Unblocked OSB, Seismic Zone 4, Site Class C

Interstory	Quakesoft		CANNY		FEMA-440
Drift (%h)	*Mean + 1 σ*	*Mean*	*Mean + 1 σ*	*Mean*	*Mean*
1.0%					
1.5%	35%	27%	27%	27%	21%
2.0%	18%	14%	23%	20%	15%
2.5%	14%	11%	19%	13%	11%
3.0%	12%	9%	13%	11%	9%
4.0%	9%	7%	11%	9%	9%

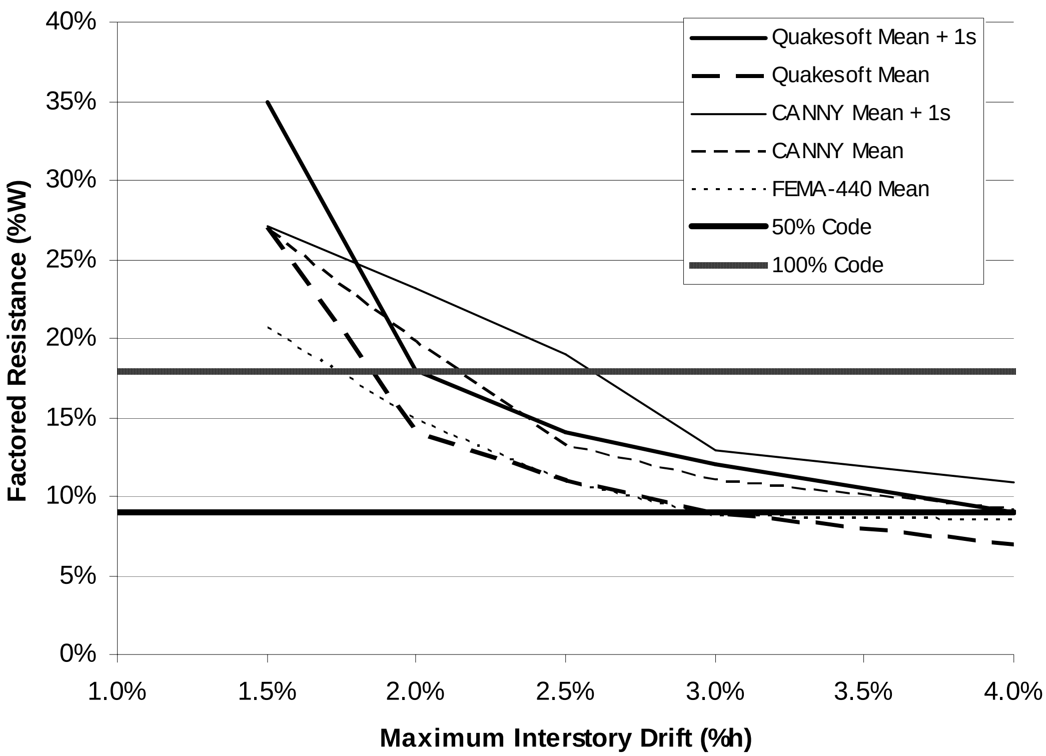


Figure C.6-2 W-2 Unblocked OSB, Seismic Zone 4, Site Class C

C6.1.2 Steel Frame Prototypes

Table C.6-3 and Figure C.6-3 compare the calculated strengths for prototype S-1. The C_v for the *Mean + 1*σ and *Mean* are 5% and 11% respectively. This indicates a good match for the retrofit values, but a high C_v for the mean values. In this case the Quakesoft values are more conservative than the other two methods. The 13%*W* retrofit value at the ISDL is higher than the 60% code level.

Table C.6-3 S-1 Concentric Braced Frame (tension only), Seismic Zone 4, Site Class C

Interstory	Quakesoft		CANNY		FEMA-440
Drift (%h)	*Mean + 1* σ	*Mean*	*Mean + 1* σ	*Mean*	*Mean*
1.0%	46%	37%	46%	34%	33%
1.5%	30%	24%	35%	23%	23%
2.0%	25%	19%	23%	19%	16%
2.5%	21%	16%	21%	14%	13%
3.0%	18%	15%	15%	12%	11%
4.0%	13%	10%	12%	7%	8%

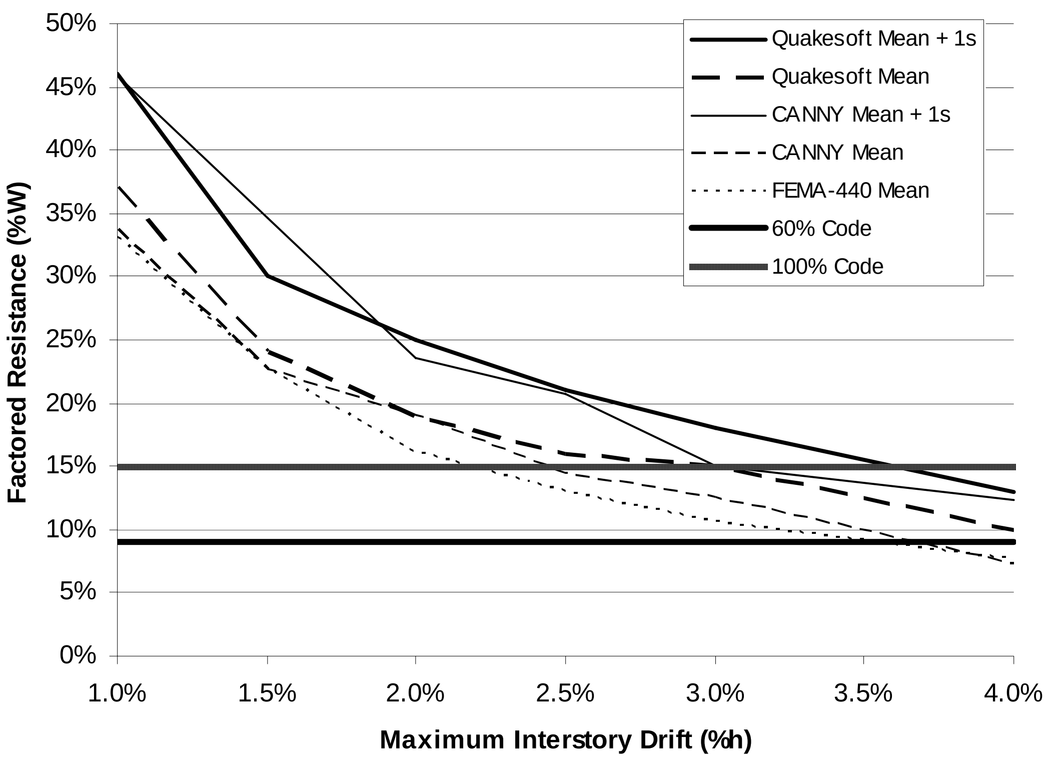


Figure C.6-3 S-1 Concentric Braced Frame (tension only), Seismic Zone 4, Site Class C

Table C.6-4 and Figure C.6-4 compare the calculated strengths for prototype S-2. The C_v for the *Mean + 1σ* and *Mean* were 7% and 8% respectively, indicating a good match. FEMA-440-DM was not used for this model as the backbone curve was too complex to be idealized to a bilinear model. No code comparison is made because the method of calculating the element resistance is significantly different (see Section 4.2(3)).

Table C.6-4 S-2 Concentric Braced Frame (Tension/Compression – C/T=0.6), Seismic Zone 4, Site Class C

Interstory Drift (%h)	Quakesoft		CANNY	
	Mean + 1 σ	*Mean*	*Mean + 1 σ*	*Mean*
1.0%	38%	27%	29%	25%
1.5%	26%	20%	25%	19%
2.0%	21%	16%	21%	14%
2.5%	18%	14%	17%	11%
3.0%				
4.0%				

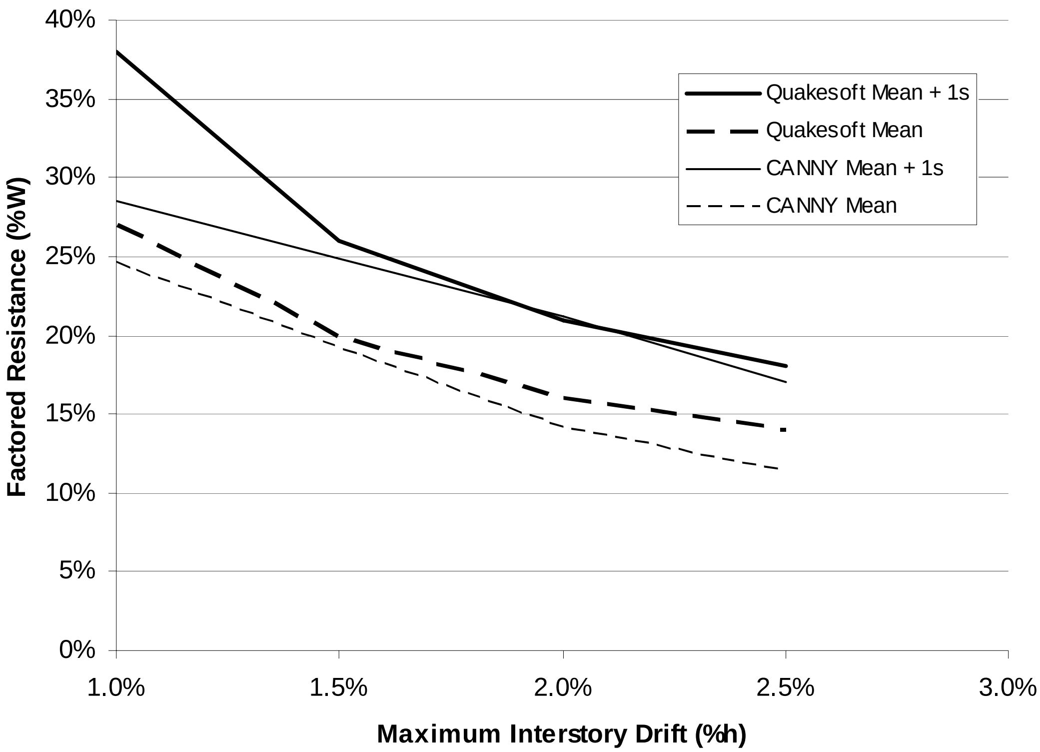


Figure C.6-4 S-2 Concentric Braced Frame (Tension/Compression – C/T=0.6), Seismic Zone 4, Site Class C

Table C.6-5 and Figure C.6-5 compare the calculated strengths for prototype S-3. The C_v for the *Mean + 1σ* and *Mean* were 3% and 10% respectively, indicating an excellent match for the retrofit values. The FEMA-440-DM results were higher than the other two methods at the high and low drift levels, resulting in the higher C_v for the Mean values. The 14%*W* retrofit value at the ISDL is much higher than the 60% code level. This is because the ISDL for Eccentric Braced Frames is conservative. A higher ISDL would significantly lower the strength demands.

Table C.6-5 S-3 Eccentric Braced Frame, Seismic Zone 4, Site Class C

Interstory	**Quakesoft**		**CANNY**		**FEMA-440**
Drift (%h)	*Mean + 1 σ*	*Mean*	*Mean + 1 σ*	*Mean*	*Mean*
1.0%	38%	29%	39%	29%	39%
1.5%	27%	21%	28%	20%	23%
2.0%	21%	15%	21%	16%	16%
2.5%	18%	14%	17%	15%	13%
3.0%	15%	13%	16%	14%	11%
4.0%	14%	10%	14%	10%	8%

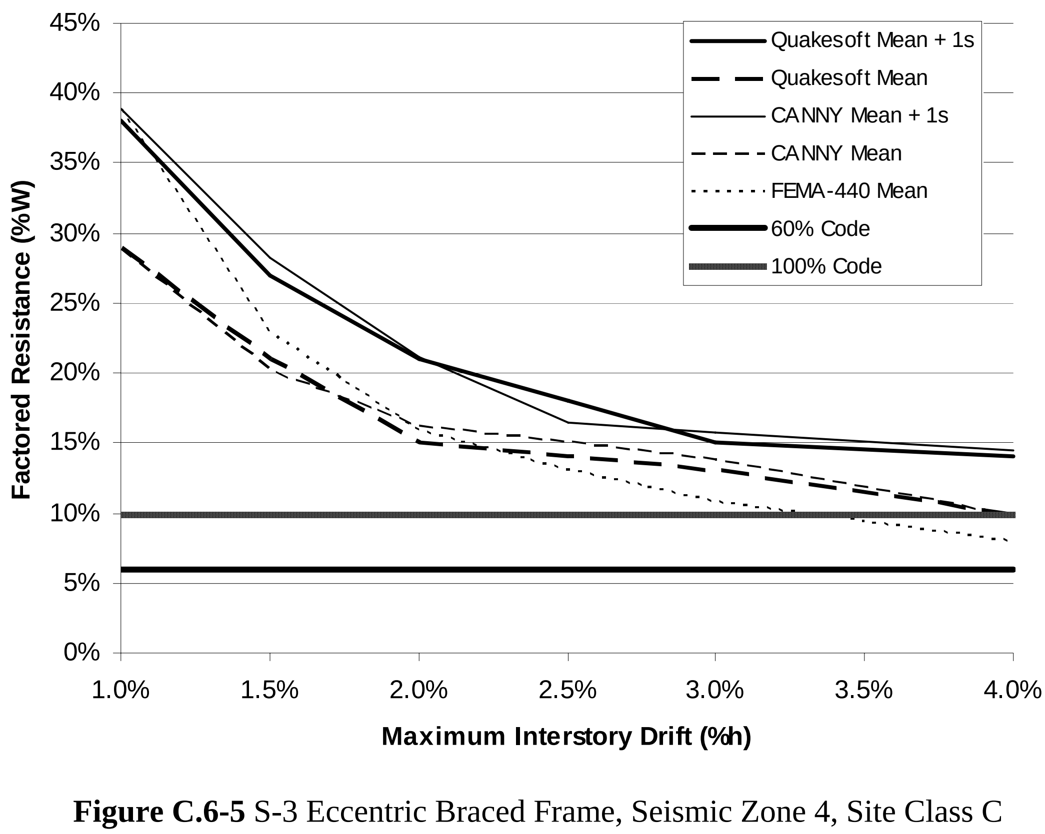


Figure C.6-5 S-3 Eccentric Braced Frame, Seismic Zone 4, Site Class C

Table C.6-6 and Figure C.6-6 compare the calculated strengths for prototype S-4. The C_v for the *Mean + 1*σ and *Mean* were 18% and 25% respectively, indicating a poor match. None of the analysis methods show a good agreement with the others. The 20%*W* retrofit value at the ISDL is much higher than the 60% code level. This is because the ISDL for Moment Frames is very conservative. A higher ISDL would significantly lower the strength demands.

Table C.6-6 S-4 Moment Frame, Seismic Zone 4, Site Class C

Interstory	**Quakesoft**		**CANNY**		**FEMA-440**
Drift (%h)	*Mean + 1* σ	*Mean*	*Mean + 1* σ	*Mean*	*Mean*
1.0%					
1.5%	49%	38%	41%	31%	23%
2.0%	35%	27%	30%	24%	16%
2.5%	29%	21%	26%	17%	13%
3.0%	24%	17%	18%	12%	11%
4.0%	20%	13%	12%	10%	8%

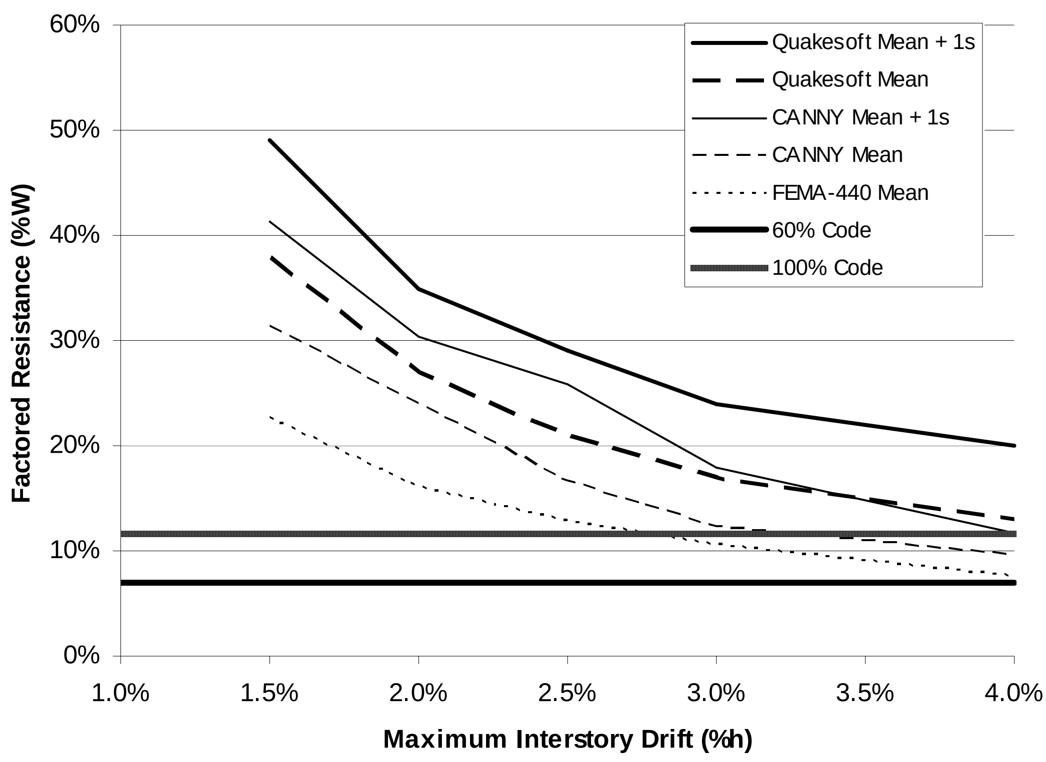


Table C.6-6 S-4 Moment Frame, Seismic Zone 4, Site Class C

C6.1.3 Concrete Prototypes

Concrete prototypes C-1 and C-2 use the same analysis results, which are also shared with prototype M-2, and as such only one data set is presented here for validation. Table C.6-7 and Figure C.6-7 compare the calculated strengths for prototype C-1. The C_v for the *Mean + 1*σ and *Mean* are both 11%. While these values are above 10%, the most conservative values (Quakesoft) were used for the final values. The 19%*W* retrofit value at the ISDL is considerably higher than the 60% code level.

Table C.6-7 C-1 Reinforced Concrete Wall Flexure, Seismic Zone 4, Site Class C

Interstory	Quakesoft		CANNY		FEMA-440
Drift (%h)	*Mean + 1 σ*	*Mean*	*Mean + 1 σ*	*Mean*	*Mean*
1.0%	28%	22%	26%	21%	21%
1.5%	22%	17%	19%	15%	14%
2.0%	19%	15%	15%	11%	10%
2.5%					
3.0%					
4.0%					

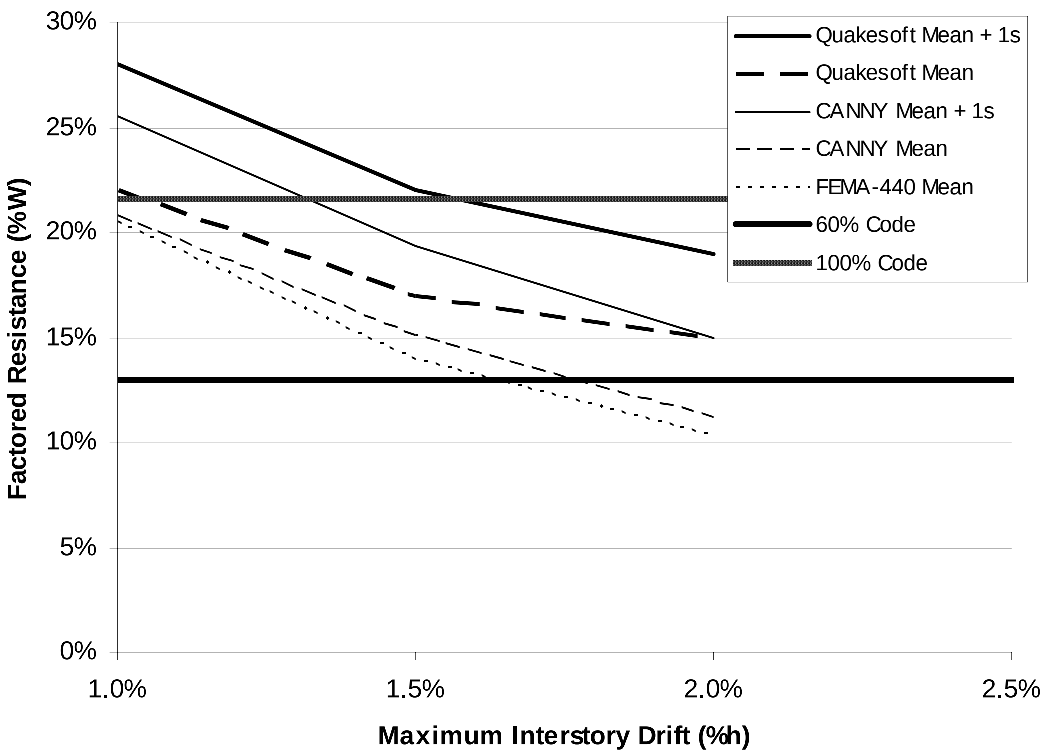


Figure C.6-7 C-1 Reinforced Concrete Wall Flexure, Seismic Zone 4, Site Class C

Table C.6-8 and Figure C.6-8 compare the calculated strengths for prototype C-3. The C_v for the *Mean + 1*σ and *Mean* are 3% and 7% respectively, indicating a good match. The 8%*W* retrofit value at the ISDL is higher than the 60% code level.

Table C.6-8 C-3 Reinforced Concrete Ductile Moment Frame, Seismic Zone 4, Site Class C

Interstory	**Quakesoft**		**CANNY**		**FEMA-440**
Drift (%h)	*Mean + 1* σ	*Mean*	*Mean + 1* σ	*Mean*	*Mean*
1.0%					
1.5%	38%	29%	35%	26%	25%
2.0%	22%	15%	22%	16%	16%
2.5%	15%	11%	15%	11%	12%
3.0%	11%	9%	11%	9%	8%
4.0%	8%	6%	9%	8%	7%

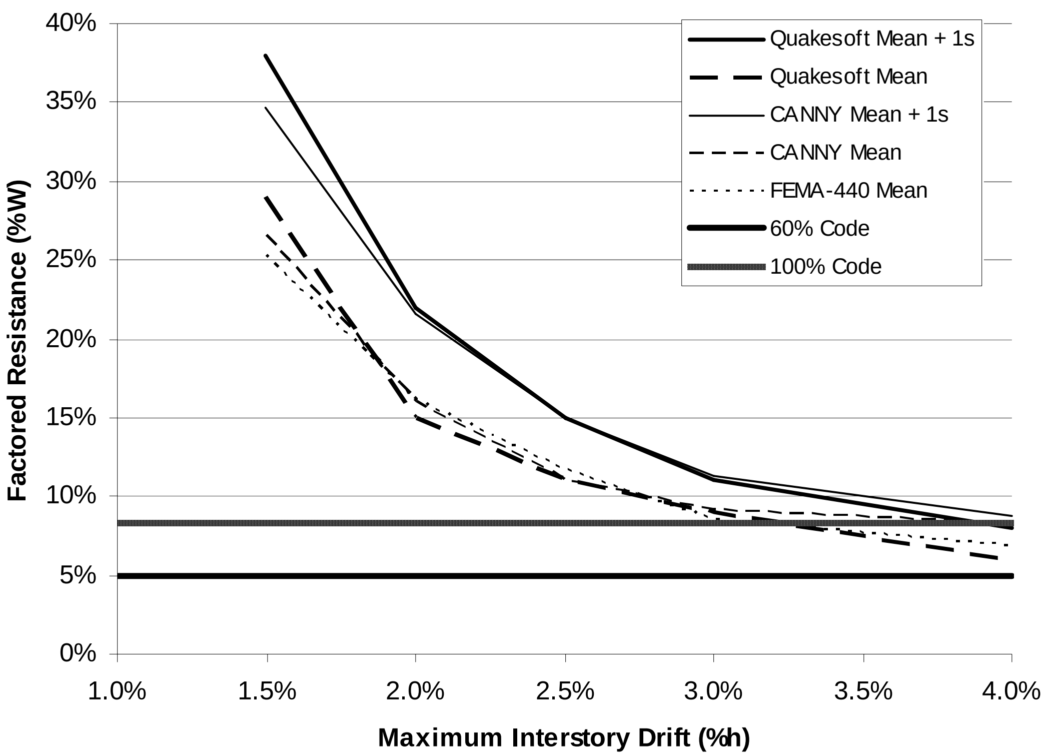


Figure C.6-8 C-3 Reinforced Concrete Ductile Moment Frame, Seismic Zone 4, Site Class C

Table C.6-9 and Figure C.6-9 compare the calculated strengths for prototype C-4. The C_v for the *Mean + 1*σ and *Mean* are 10% and 13% respectively. This indicates a adequate match for the retrofit values, but a high C_v for the mean values. In this case the FEMA-440-DM values are more conservative than the other two methods. The 11%*W* retrofit value at the ISDL is slightly higher than the 60% code level.

Table C.6-9 C-4 Reinforced Concrete Moderately Ductile Moment Frame, Seismic Zone 4, Site Class C

Interstory Drift (%h)	Quakesoft *Mean + 1 σ*	Quakesoft *Mean*	CANNY *Mean + 1 σ*	CANNY *Mean*	FEMA-440 *Mean*
1.0%					
1.5%	40%	29%	48%	35%	33%
2.0%	26%	18%	33%	17%	24%
2.5%	19%	14%	16%	13%	19%
3.0%	13%	10%	13%	10%	10%
4.0%	11%	9%	10%	9%	7%

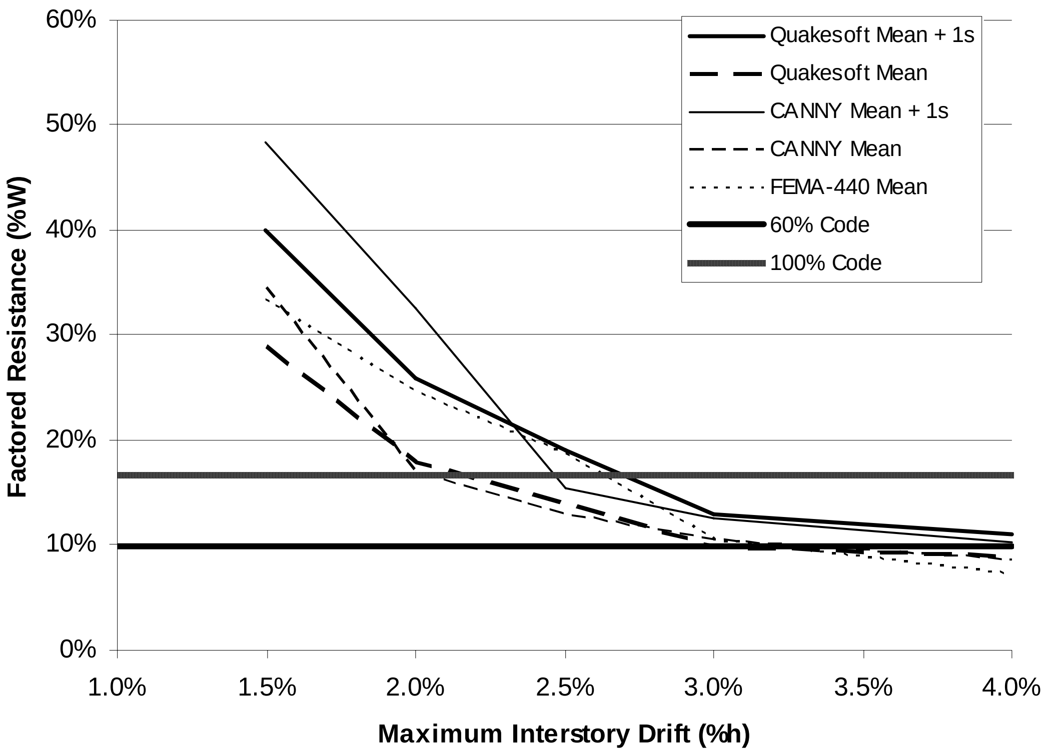


Figure C.6-9 C-4 Reinforced Concrete Moderately Ductile Moment Frame, Seismic Zone 4, Site Class C

Table C.6-10 and Figure C.6-10 compare the calculated strengths for prototype C-5. The C_v for the *Mean + 1σ* and *Mean* are 18% and 14% respectively, indicating a poor match. None of the analysis methods show a good agreement with the others in the drift range of 1.5 to 2.5%. The 20%*W* retrofit value at the ISDL is higher than the 60% code level.

Table C.6-10 C-5 Reinforced Concrete Conventional Construction Moment Frame, Seismic Zone 4, Site Class C

Interstory	Quakesoft		CANNY		FEMA-440
Drift (%h)	*Mean + 1 σ*	*Mean*	*Mean + 1 σ*	*Mean*	*Mean*
1.0%					
1.5%	35%	27%	50%	38%	37%
2.0%	28%	21%	37%	19%	28%
2.5%	23%	17%	19%	17%	22%
3.0%	21%	16%	18%	15%	15%
4.0%	20%	14%	15%	12%	15%

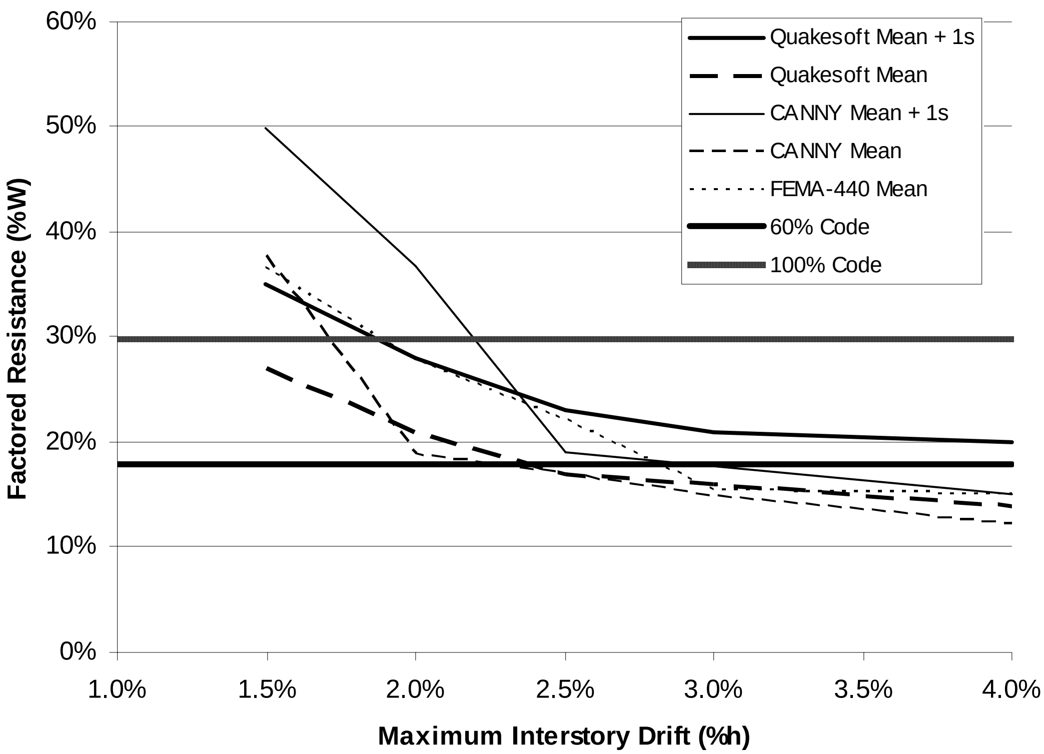


Figure C.6-10 C-5 Reinforced Concrete Conventional Construction Moment Frame, Seismic Zone 4, Site Class C

C6.1.4 Masonry and Rocking Prototypes

This section shows both the masonry and rocking prototypes. Prototype M-2 uses the same model as C-1, so is not shown here. Additionally, prototype B-1 is the same as prototype M-1, so only the results for M-1 are shown.

Table C.6-11 and Figure C.6-11 compare the calculated strengths for prototype M-1. The C_v for the *Mean + 1*σ and *Mean* are 9% and 8% respectively, indicating an adequate match. The 15%*W* retrofit value at the ISDL is higher than the 50% code level.

Table C.6-11 M-1 Unreinforced Masonry Shear, Seismic Zone 4, Site Class C

Interstory	Quakesoft		CANNY		FEMA-440
Drift (%h)	*Mean + 1 σ*	*Mean*	*Mean + 1 σ*	*Mean*	*Mean*
1.0%	22%	16%	20%	14%	14%
1.5%	15%	11%	13%	9%	10%
2.0%					
2.5%					
3.0%					
4.0%					

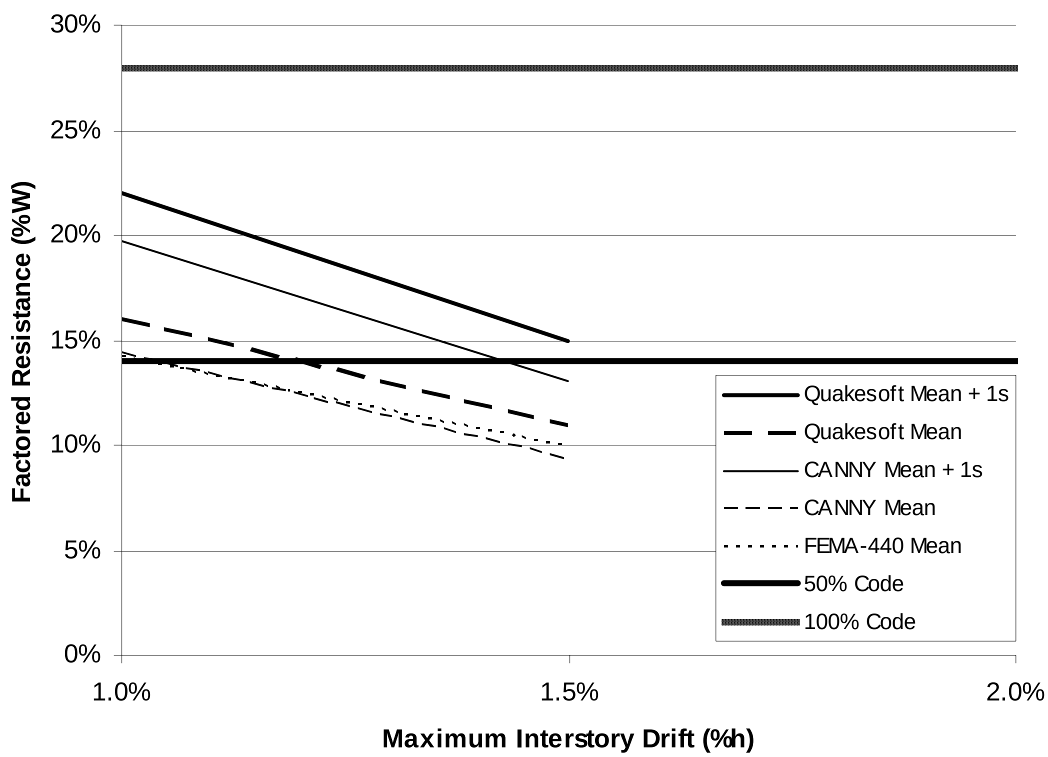


Figure C.6-11 M-1 Unreinforced Masonry Shear, Seismic Zone 4, Site Class C

Table C.6-12 and Figure C.6-12 compare the calculated strengths for prototype R-1. The C_v for the *Mean + 1*σ and *Mean* were 26% and 28% respectively, indicating a poor match. Neither of the analysis methods show a good agreement with the other. FEMA-440-DM analysis was not included as it cannot simulate a non-linear elastic hysteretic curve. The 12%*W* retrofit value at the ISDL is much lower than the 50% code level.

Table C.6-12 R-1 Low Aspect Ratio Rocking Elements, Seismic Zone 4, Site Class C

Interstory	Quakesoft		CANNY	
Drift (%h)	*Mean + 1 σ*	*Mean*	*Mean + 1 σ*	*Mean*
1.0%	51%	32%	63%	41%
1.5%	33%	25%	47%	35%
2.0%	22%	16%	38%	24%
2.5%	19%	14%	27%	21%
3.0%	16%	12%	23%	19%
4.0%	12%	9%	19%	15%

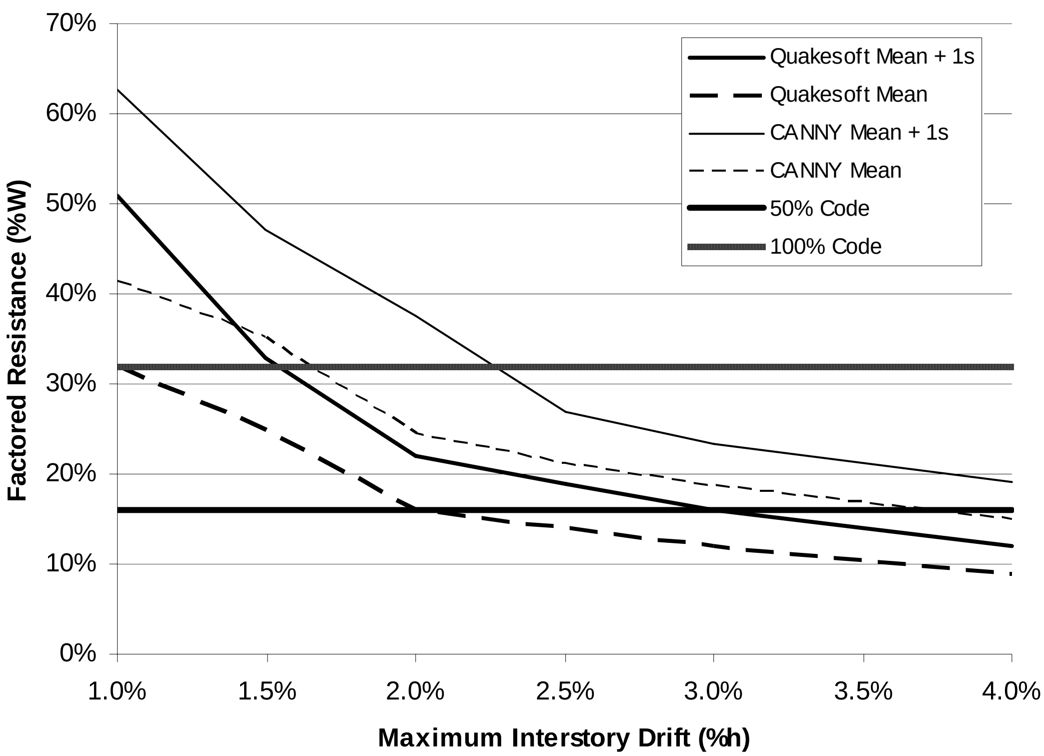


Figure C.6-12 R-1 Low Aspect Ratio Rocking Elements, Seismic Zone 4, Site Class C

Table C.6-13 and Figure C.6-13 compare the calculated strengths for prototype R-2. The C_v for the *Mean + 1σ* and *Mean* are 17% and 9% respectively, indicating a poor match. Like R-1 the FEMA-440-DM method was not used. This prototype requires further investigation. The 19%*W* retrofit value at the ISDL is higher than the 60% code level.

Table C.6-13 R-2 Medium Aspect Ratio Rocking Elements, Seismic Zone 4, Site Class C

Interstory Drift (%h)	Quakesoft		CANNY	
	Mean + 1 σ	*Mean*	*Mean + 1 σ*	*Mean*
1.0%	81%	55%	64%	60%
1.5%	58%	44%	59%	49%
2.0%	38%	31%	52%	35%
2.5%	34%	26%	47%	31%
3.0%	25%	22%	34%	23%
4.0%	19%	15%	23%	18%

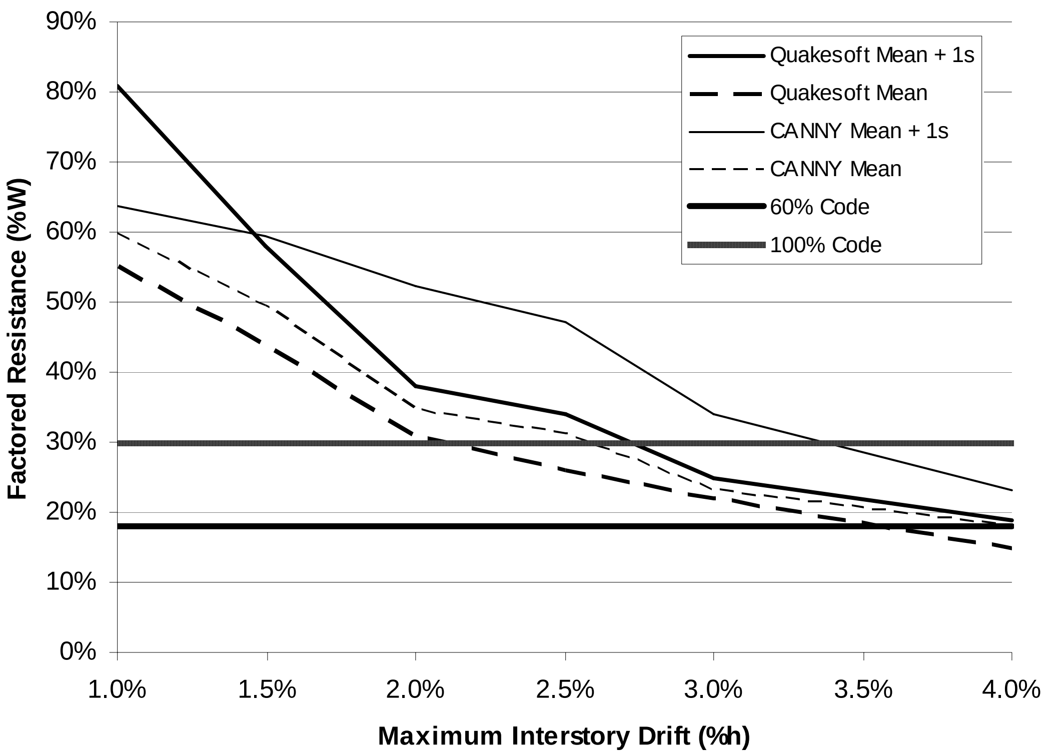


Figure C.6-13 R-2 Medium Aspect Ratio Rocking Elements, Seismic Zone 4, Site Class C

Table C.6-14 and Figure C.6-14 compare the calculated strengths for prototype R-3. The C_v for the *Mean + 1σ* and *Mean* are 11% and 9% respectively, indicating a poor match. Like R-2 the FEMA-440-DM method was not used. This prototype requires further investigation. The 24%*W* retrofit value at the ISDL is higher than the 60% code level.

Table C.6-14 R-3 High Aspect Ratio Rocking Elements, Seismic Zone 4, Site Class C

Interstory Drift (%h)	Quakesoft		CANNY	
	Mean + 1 σ	*Mean*	*Mean + 1 σ*	*Mean*
1.0%				
1.5%	84%	64%	71%	65%
2.0%	65%	50%	67%	58%
2.5%	49%	38%	62%	30%
3.0%	42%	31%	58%	26%
4.0%	24%	19%	25%	20%

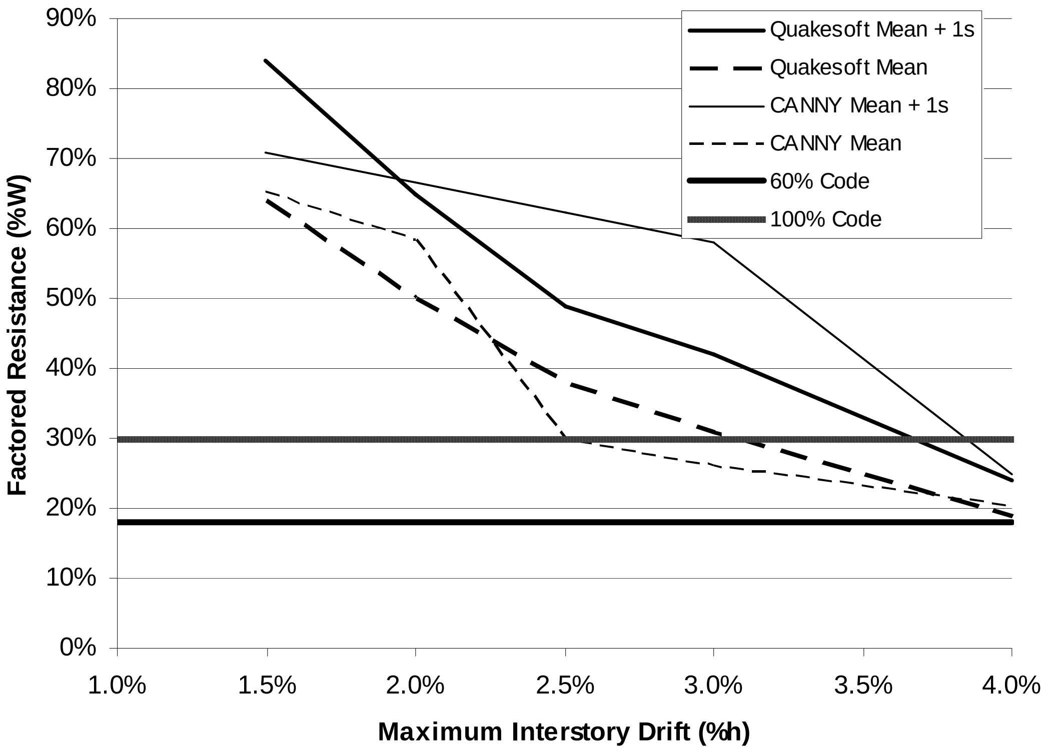


Figure C.6-14 R-3 High Aspect Ratio Rocking Elements, Seismic Zone 4, Site Class C

C6.1.5 Diaphragm Prototypes

The results of the six diaphragm prototypes are given below. Unlike the LDRS prototypes, there is only one set of results from each analysis method, in this case Quakesoft and CANNY. The results given are the design values based on both a diaphragm shear strain limit (DISL) and a displacement limit. See Section B11.0 for more details on the diaphragm performance objectives.

Table C.6-15 and Figure C.6-15 compare the calculated strengths for prototype D-1. The C_v for the results is 30%. While these values are above 10%, the strengths are very small, thus exaggerating the C_v. The more conservative Quakesoft values were used in the D-1 resistance tables.

Table C.6-16 and Figure C.6-16 compare the calculated strengths for prototype D-2. The C_v for the results is 17%. While this is above the 10% mark, the actually differences between the values are within 4%.

Table C.6-17 and Figure C.6-17 compare the calculated strengths for prototype D-3. The C_v for the results is 19%. The trend is similar but there is an average offset of about 6%. This prototype will undergo further investigation by the EPR.

Table C.6-18 and Figure C.6-18 compare the calculated strengths for prototype D-4. The C_v for the results is 29%. These values are poor and need further investigation. The more conservative values for Quakesoft are used in the resistance tables.

Table C.6-19 and Figure C.6-19 compare the calculated strengths for prototype D-5. The C_v for the results is 32%. These values are poor and need further investigation.

Table C.6-20 and Figure C.6-20 compare the calculated strengths for prototype D-6. The C_v for the results is 6%, which is an excellent match.

Table C.6-15 D-1 Blocked Plywood/OSB Diaphragm, Seismic Zone 4, Site Class C

Prototype D-1, Zone 4, Site Clas

Span	**Strength (%W)**	
metres	*Quakesoft*	*CANNY*
10	8%	9%
15	8%	8%
20	8%	6%
30	8%	5%
40	7%	4%
50	7%	3%

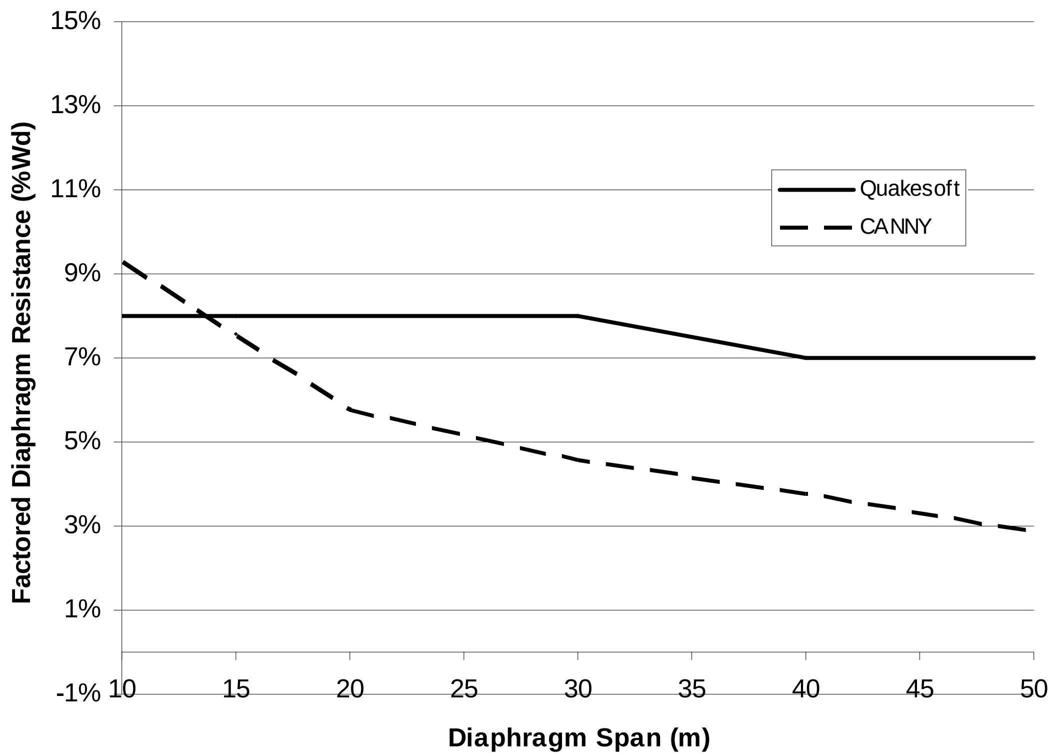


Figure C.6-15 D-1 Blocked Plywood/OSB Diaphragm, Seismic Zone 4, Site Class C

Table C.6-16 D-2 Unblocked Plywood/OSB Diaphragm, Seismic Zone 4, Site Class C

Prototype D-2, Zone 4, Site Clas

Span	**Strength (%W)**	
metres	*Quakesoft*	*CANNY*
10	8%	11%
15	8%	9%
20	6%	8%
30	6%	4%
40	6%	5%
50	11%	9%

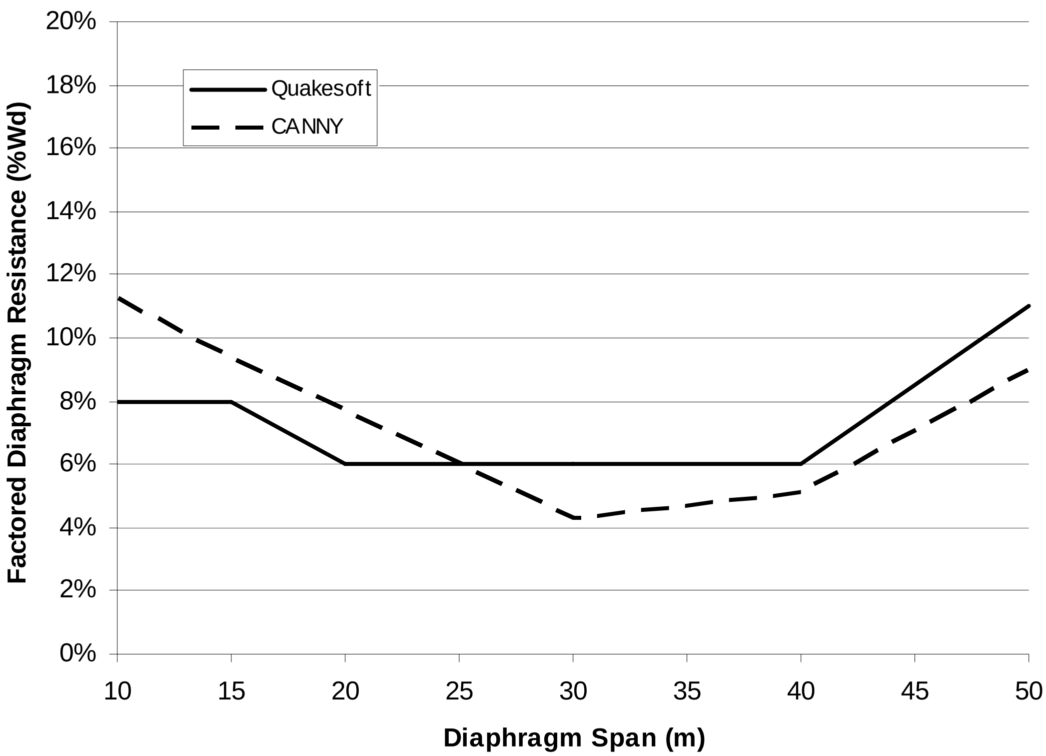


Figure C.6-16 D-2 Unblocked Plywood/OSB Diaphragm, Seismic Zone 4, Site Class C

Table C.6-17 D-3 Steel Deck Diaphragm Type A, Seismic Zone 4, Site Class C

Prototype D-3, Zone 4, Site Clas

Span	**Strength (%W)**	
metres	*Quakesoft*	*CANNY*
10	18%	25%
15	19%	27%
20	22%	25%
30	18%	23%
40	16%	21%
50	17%	22%

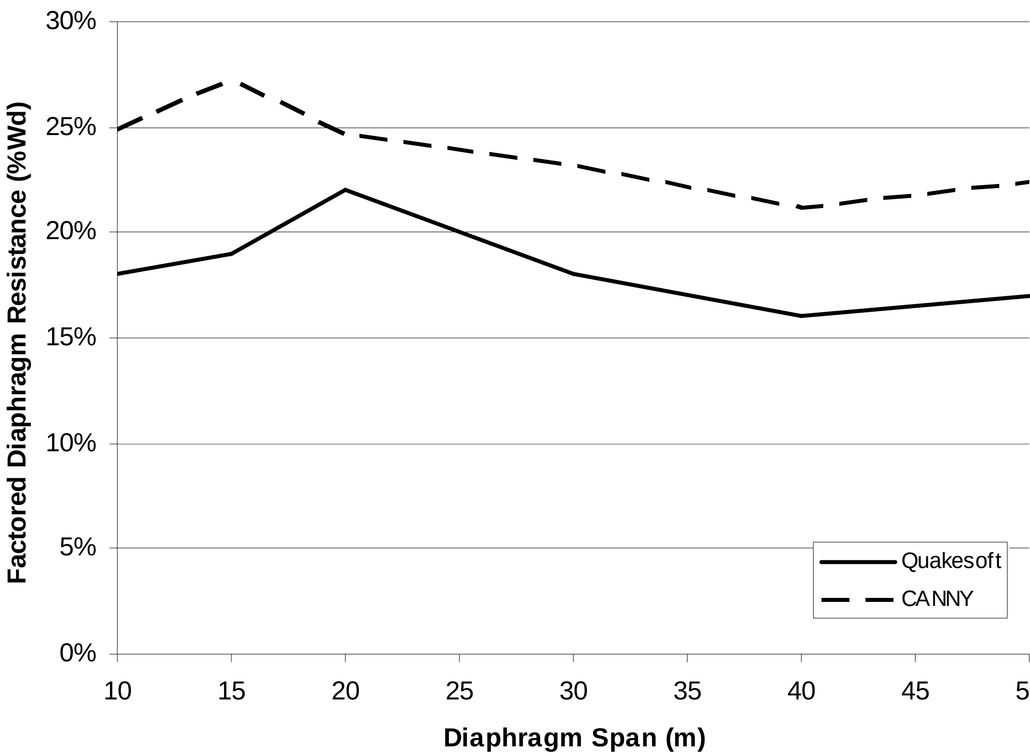


Figure C.6-17 D-3 Steel Deck Diaphragm Type A, Seismic Zone 4, Site Class C

Table C.6-18 D-4 Steel Deck Diaphragm Type B, Seismic Zone 4, Site Class C

Span	**Strength (%W)**	
metres	*Quakesoft*	*CANNY*
10	30%	35%
15	41%	35%
20	45%	35%
30	55%	35%
40	55%	30%
50	73%	29%

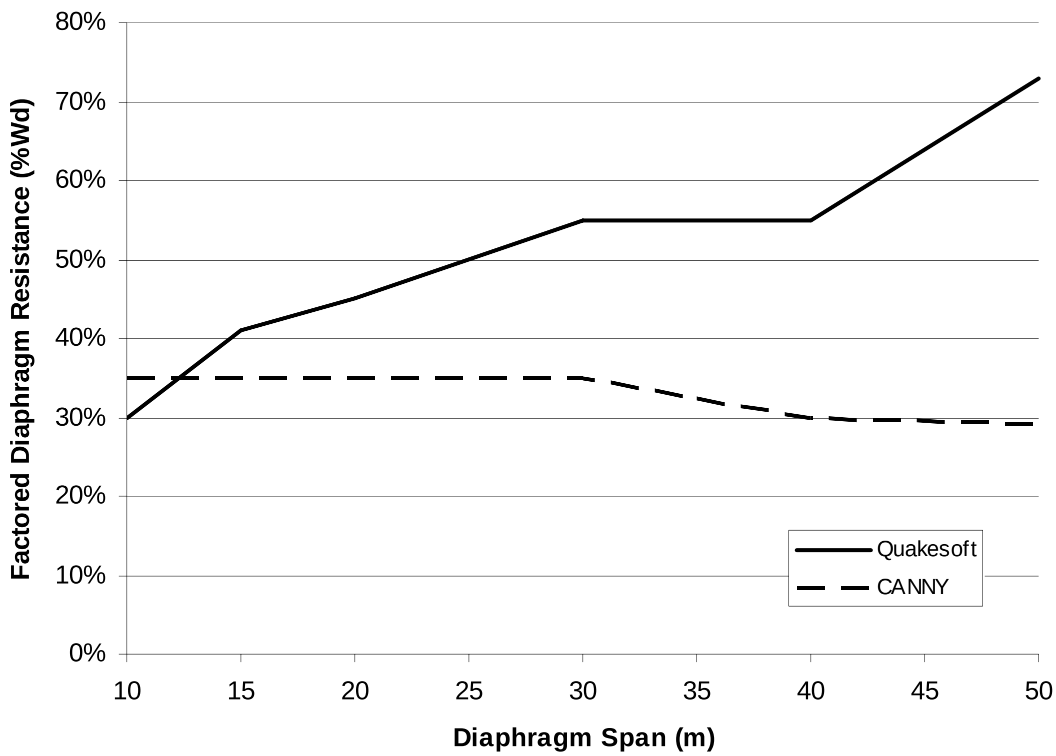


Figure C.6-18 D-4 Steel Deck Diaphragm Type B, Seismic Zone 4, Site Class C

Table C.6-19 D-5 Steel Braced-frame Diaphragm (Tension only), Seismic Zone 4, Site Class C

Prototype D-5, Zone 4, Site Clas

Span	**Strength (%W)**	
metres	*Quakesoft*	*CANNY*
10	12%	11%
15	14%	10%
20	14%	7%
30	15%	7%
40	13%	5%
50		

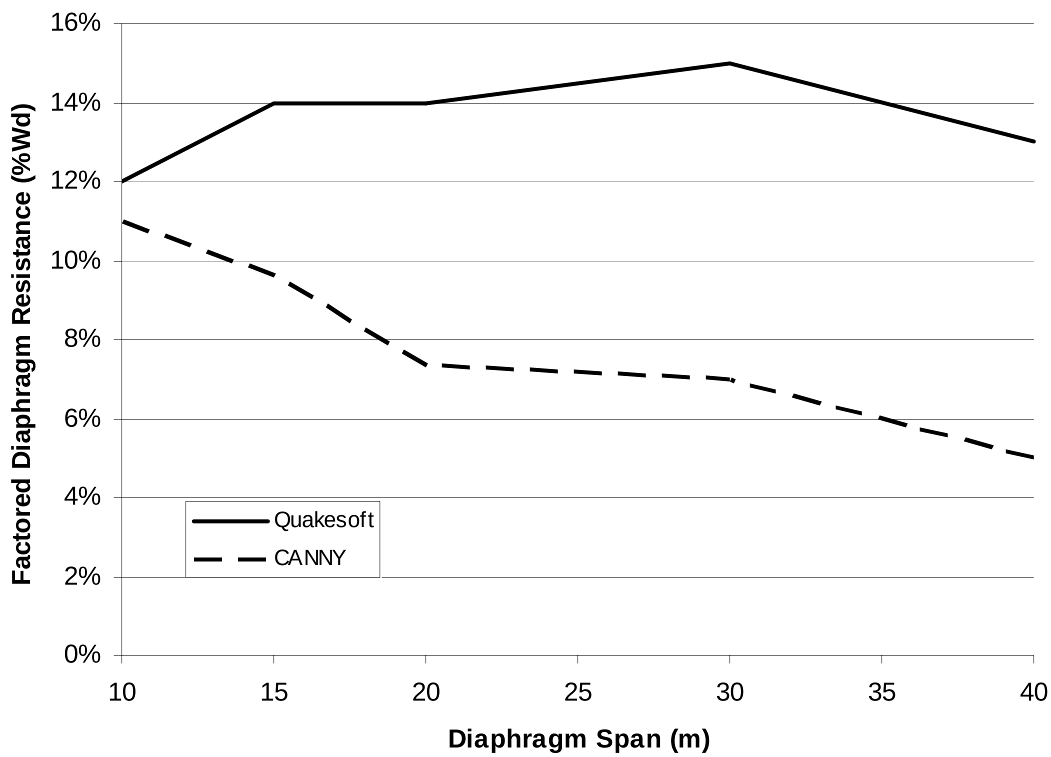


Figure C.6-19 D-5 Steel Braced-frame Diaphragm (Tension only), Seismic Zone 4, Site Class C

Table C.6-20 D-6 Steel Braced-frame Diaphragm (Tension/Compression), Seismic Zone 4, Site Class C

Span	**Strength (%W)**	
metres	*Quakesoft*	*CANNY*
10	9%	12%
15	11%	12%
20	12%	12%
30	12%	12%
40	12%	11%
50	11%	12%

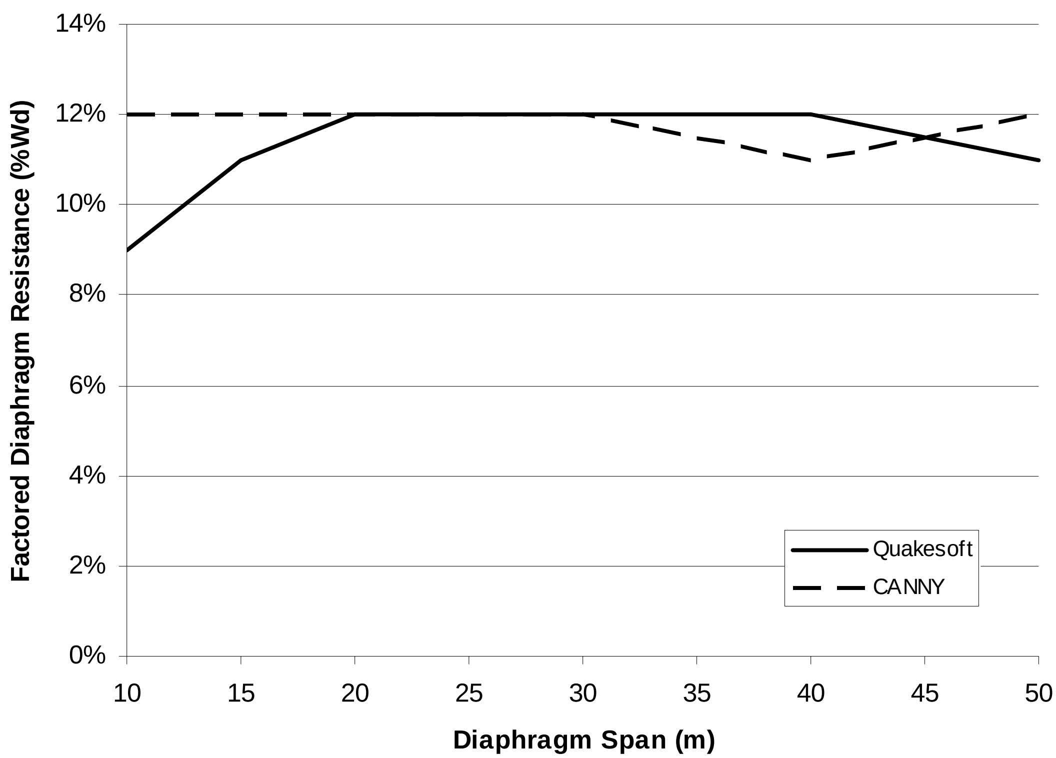


Figure C.6-20 D-6 Steel Braced-frame Diaphragm (Tension/Compression), Seismic Zone 4, Site Class C

C6.1.6 Conclusions of Validation

The majority of the prototype models had an average C_v equal to or lower than 10%, indicating an acceptable level of correlation. However, several prototypes did not show as strong an agreement, and need further investigation into their modeling methods. These prototypes were:

- W-2 Unblocked OSB/Plywood
- S-4 Steel Moment Frames
- C-1 Concrete Shearwalls
- C-5 Conventional Construction Concrete Moment Frames
- R-1 Low Aspect Ratio Rocking Elements
- R-2 Medium Aspect Ratio Rocking Elements
- R-3 High Aspect Ratio Rocking Elements
- D-1 Blocked OSB/Plywood
- D-2 Unblocked OSB/Plywood Diaphragms
- D-3 Steel Deck Diaphragm Type A
- D-4 Steel Deck Diaphragm Type B
- D-5 Steel Brace-frame Diaphragm (Tension Only)

A few prototypes have very conservative strength demands, in some cases near or above 100% of the 2005 NBCC. These prototypes will be further investigated in future editions. These prototypes were:

- S-3 Eccentric Braced Frames
- S-4 Steel Moment Frames
- C-1 Concrete Shearwalls
- R-1 Low Aspect Ratio Rocking Elements (really below code)
- D-5 Steel Deck Diaphragm Type B

It should be noted that the reduction in strength demands due to an increased storey height (see Section B3.5) do mitigate the seemingly high demands for the above prototypes.

C6.2 Sensitivity Analysis of Prototype Models

Sensitivity analysis was done to investigate the influence of particular parameters on the behaviour of a prototype. This analysis also helped to define the level of variation in these parameters which could still be adequately defined by the prototypes.
The sensitivity analysis was carried out by altering a given parameter of a prototype model and running it again for the suite of ground motions. This establish a new factored resistance vs. max. drift relationship.

Like the resistance tables, the mean +1 standard deviation was used. The results of the sensitivity study are plotted in a similar fashion to the resistance tables. However instead of having multiple lines representing different site classes, there are different lines representing variations in the given parameter. The variations in the parameters are associated with a physical meaning within a given structural system.

All of the analysis for the Sensitivity Study was conducted using CANNY (see Section C3.1). The full suite of ground motions were used and they were scaled to Site Class C in Seismic Zone 4.

C6.2.1 Steel Frame Prototypes

The sensitivity studies for the steel prototypes are shown in Figures C.6-21 through C.6-25. Each of the figures and the prototype parameters they are investigating are discussed below.

Figure C.6-21 show the response of prototype S-1 for different yield drifts. The default yield drift for S-1 is 0.3%. Variations in the yield drift can be associated with a difference in the angle of the brace, as well as the relative stiffness of the rest of the frame. The plot indicates that the response of the steel braced frames (tension only) is mildly sensitive to the variation in stiffness. For stiffer frames (0.2% yield drift) the resistance tables are conservative, as these frames have lower strength demands. For more flexible frames (up to 0.4% yield drift) their response is about the same as the default 0.3% yield drift. Brace frames more flexible that 0.4% should not use the resistance tables for S-1, as they may be unconservative. It is assumed that this behaviour would also be similar for prototype S-2, as they have similar characteristics and the same stiffness (yield drift).

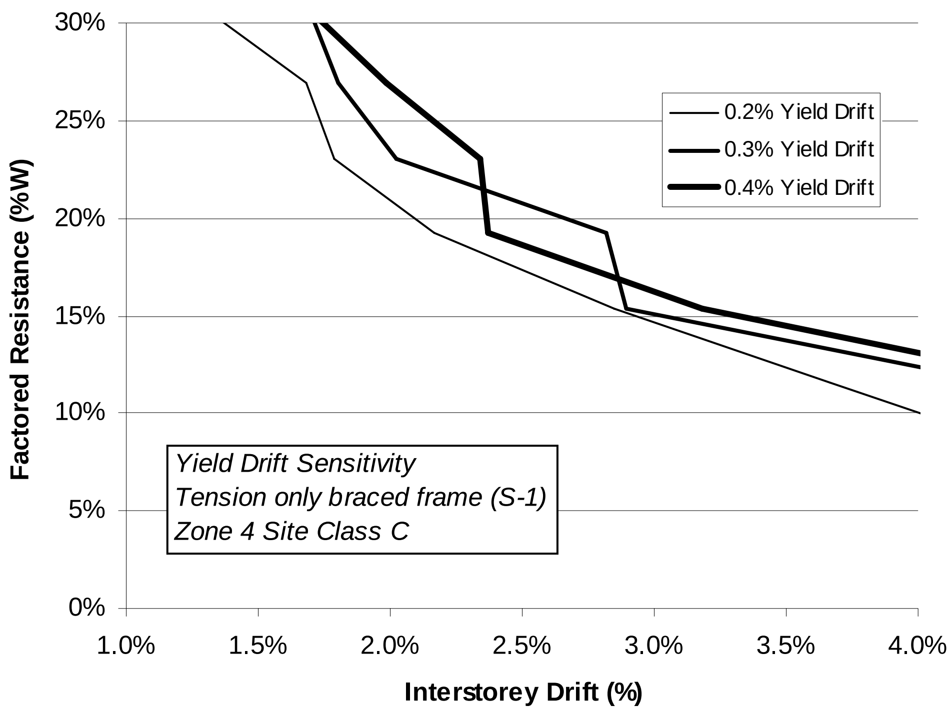


Figure C.6-21 Sensitivity to Yield Drift (brace angle) for Prototype S-1 Concentric Braced Frame (Tension Only)

Figure C.6-22 show the response of prototype S-1 for different levels of strain hardening. The default strain hardening for S-1 is 0.1%. Modeling a low rate of strain hardening is undoubtedly conservative. This sensitivity study investigates the degree of conservatism. The plot shows that there is a small sensitivity in the response of prototype S-1 to the level of strain hardening, ranging from 0.1% to 2%. The default value of 0.1% results in a slightly conservative response compared to the higher levels of strain hardening, but it is not significant enough to modify the existing model.

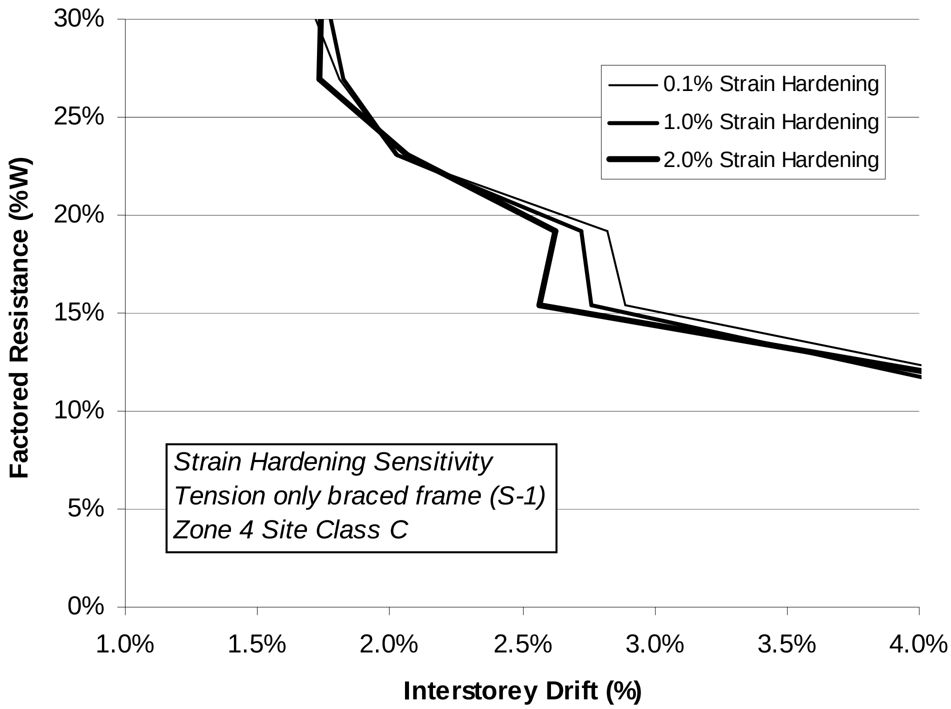


Figure C.6-22 Sensitivity to Strain Hardening for Prototype S-1 Concentric Braced Frame (Tension Only)

Figure C.6-23 show the response of prototype S-2 for different ratios of strength of the compression brace to the tension brace. The default yield drift for S-2 is 60%, which means the strength of the compression brace is 60% of the strength of the tension brace. This value was taken as it was felt that it was the most representative of tension/compression braced frames. Note that the factored resistance is based solely on the strength of the tension brace (i.e. the strength of the compression brace is not included in the factored resistance or demand). The plot indicates that there is some variation in the response of the system based on the relative strength of the compression brace, as one might expect. However, this variation is small. Braced frame systems with a high relative compression strength will be conservative if designed to the resistance tables. Those with a low percentage (but not lower than 30%) will be slightly unconservative, but still within an acceptable range. For braced frames with a relative strength lower than 30% prototype S-1 should be used.

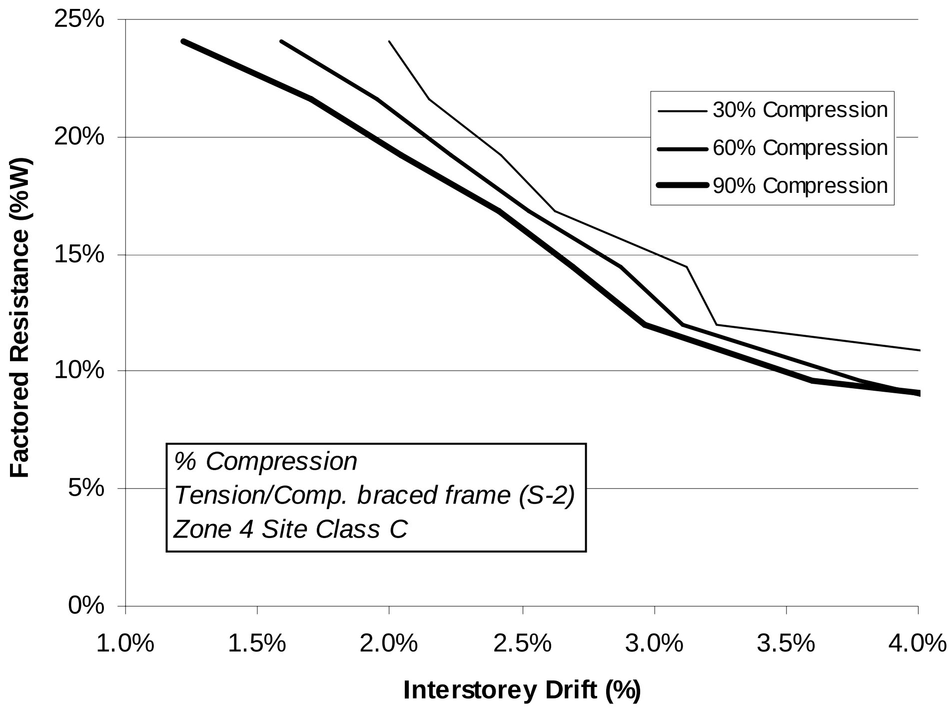


Figure C.6-23 Sensitivity to Relative Strength of Brace in Compression to Tension for Prototype S-2 Concentric Braced Frame (Tension/Compression)

Figure C.6-24 show the response of prototype S-4 for different yield drifts (stiffness). The default yield drift for S-4 is 1%. Yield drifts from 0.5% to 1.2% were investigated. The results show that there is a significant amount of scatter for the yield drifts ranging from 0.8% to 1.2%. There is no clear trend, but it appears that the default yield drift (1%) is a reasonable estimate for this range of stiffnesses. Frames more flexible than the 1.2% yield drift should not use the resistance tables for S-4, as they would most likely be unconservative. Stiffer frames would be conservative using the S-4 prototype. Very stiff frames with a 0.5% yield drift or less, should use the resistance tables for eccentrically braced frames (S-3).

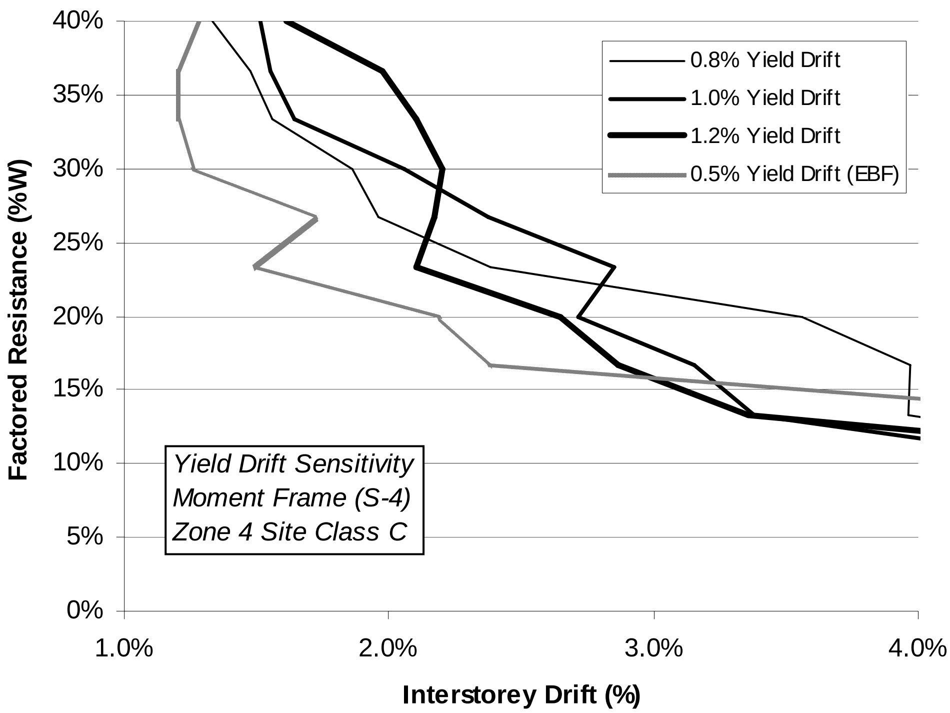


Figure C.6-24 Sensitivity to Yield Drift for Prototype S-4 Moment Frame

Figure C.6-25 show the response of prototype S-4 for two levels of strain hardening. The default strain hardening for S-4 is 0.1%. Modeling a low rate of strain hardening is conservative. The plot shows that there is a small sensitivity in the response of prototype S-4 to the level of strain hardening, ranging from 0.1% to 5%. The default value of 0.1% results in a conservative response compared to the higher level of strain hardening.

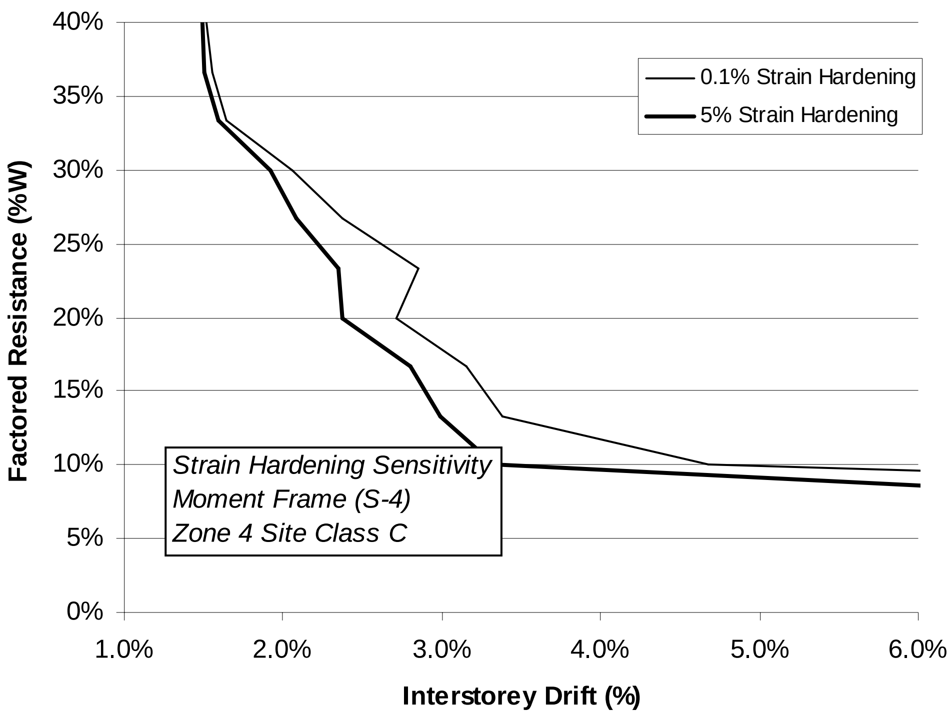


Figure C.6-25 Sensitivity to Strain Hardening for Prototype S-4 Moment Frame

C6.2.2 Concrete Prototypes

The sensitivity studies for the reinforced concrete prototypes are shown in Figures C.6-26 through C.6-28. Each of the figures and the prototype parameters they are investigating are discussed below.

Figure C.5-26 shows the sensitivity of the concrete shear wall prototypes (C-1, C-2 and even M-2) to the variation in yield drift (relative stiffness). The default yield drift for C-1 is 0.25%. The variation in relative stiffness can be related to aspect ratio (see Section B7.4). The range of yield drifts (0.0075% to 0.4%) account for aspect ratios (H/L) of cantilever walls from 1 to 8. The 1.0% yield drift is representative of a concrete moment frame. The plot indicates that shearwalls are sensitive to their stiffnesses (yield drift). Walls stiffer than used for the tabular values (0.25% yield drift) would require less strength, thus the tabular values are conservative. Walls more flexible than used for the tabular values would require more strength, thus the tabular values are unconservative. For very flexible shear walls, a moment frame prototype (C-3, C-4 or C-5) might be more appropriate.

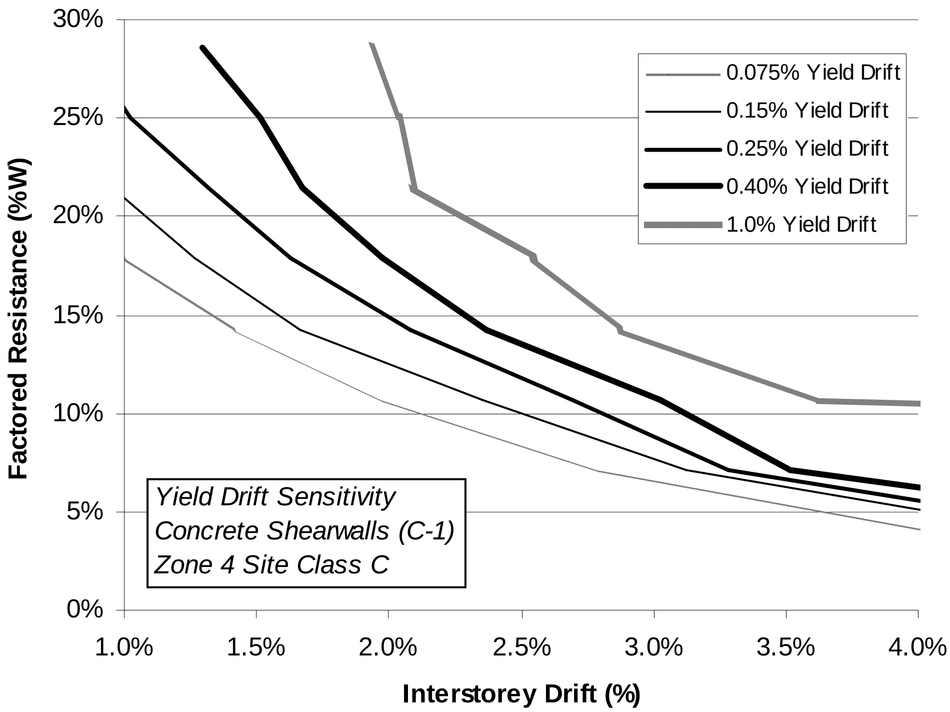


Figure C.6-26 Sensitivity to Yield Drift for Prototype C-1 and C-2 Concrete Walls

Figure C.6-27 show the response of prototype C-3 for two levels of strain hardening. The default strain hardening for C-3 is 0%. Modeling no strain hardening is conservative. The plot shows that there is a small sensitivity in the response of prototype C-3 to the level of strain hardening, ranging from 0% to 8%. The default value of 0% results in a conservative response compared to the higher level of strain hardening.

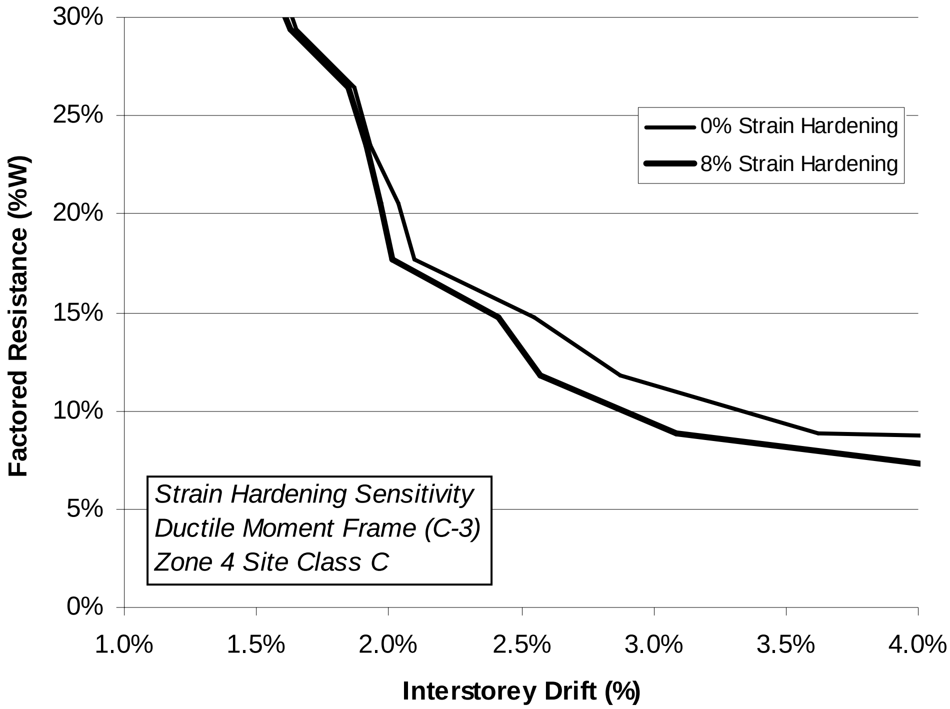


Figure C.6-27 Sensitivity to Strain Hardening for Prototype C-3 Ductile Moment Frame

Figure C.5-28 shows the sensitivity of the concrete moment frame (conventional construction) prototype (C-5) to the variation in yield drift (relative stiffness). The default yield drift for C-5 is 1%. The plot shows that prototype C-5 is sensitive to the yield drift. Stiffer frames (i.e. yield drifts below 1%) require less strength than specified on the resistance tables. Very stiff frames (with yield drifts of 0.25% or less) could use the shearwall resistance tables (C-1 and C-2). More flexible frames (with yield drifts greater than 1%) would be unconservative if assessed/retrofitted with the resistance tables and are thus are outside the scope of these guidelines. It is assumed that the other concrete moment frame models (C-3 and C-4) would show a similar pattern to C-5.

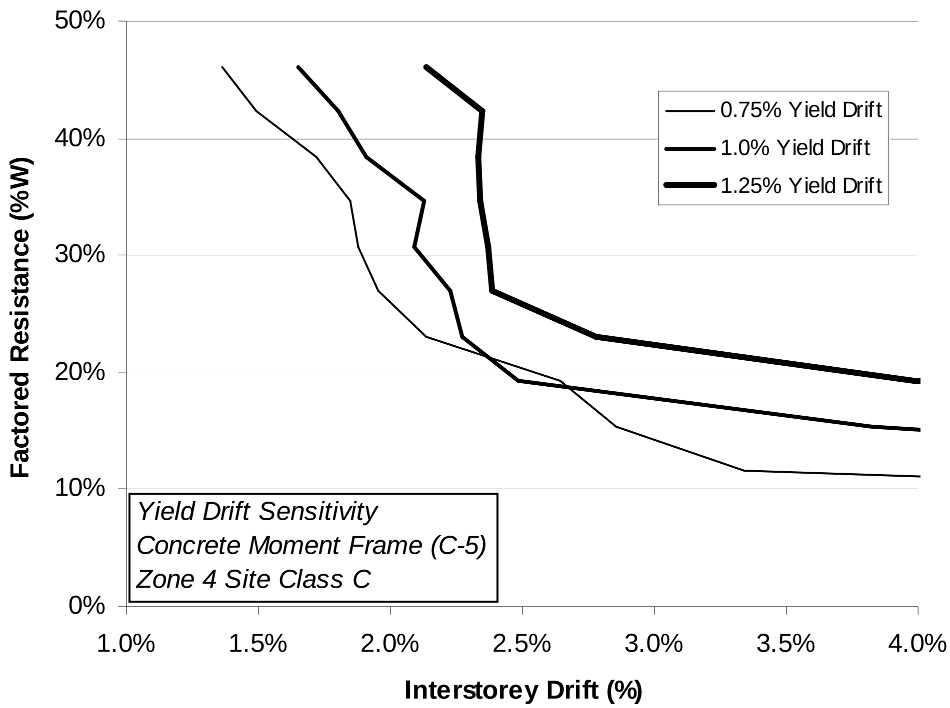


Figure C.6-28 Sensitivity to Yield Drift for Prototype C-5 Conventional Construction Moment Frame

C6.2.3 Masonry and Rocking Prototypes

The sensitivity studies for the masonry and rocking prototypes are shown in Figures C.6-29 and C.6-30. Each of the figures and the prototype parameters they are investigating are discussed below.

The bed-joint sliding model uses a very robust hysteretic model (BL elastic-perfectly plastic)(see Figure C.5-21) and this sensitivity study was performed to examine what influence this has on the response of the prototypes. One of the major differences between bed-joint sliding and toe-crushing failure is the shape of the hysteretic curve. Figure C.5-22 shows the CL (modified Clough) hysteretic curve, which is more appropriate for a toe-crushing failure than the BL hysteretic curve. Figure C.6-27 indicates that there appears to be no significant change in behaviour for M-1 (and B-1) between the BL and the CL hysteretic behaviours. The final hysteretic behaviour is the peak-oriented model (PO). This model has much less hysteretic damping than the modified Clough. This model shows a significant change from the other two. This model is very extreme, and is the type expected from a diagonal tension failure, which typically has a much higher “capacity” than bed-joint sliding.

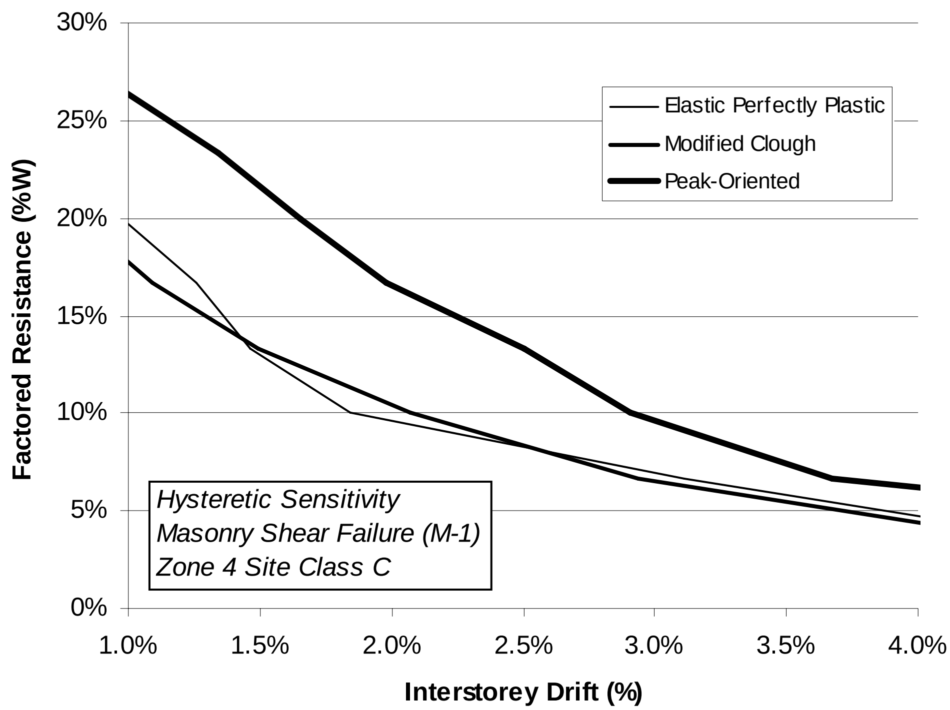


Figure C.6-29 Sensitivity to Hysteretic Properties for Prototype M-1 Unreinforced Masonry Shear

Figure C.6-30 shows the sensitivity of the rocking elements (R-1 to R-3) to the variation in yield drift (relative stiffness). The default yield drifts are 0.15% (R-1), 0.6% (R-2) and 1.2% (R-3). The variation in relative stiffness can be related to aspect ratio (see Section B10.1). The range of yield drifts (0.15% to 1.2%) account for aspect ratios (H/L) of stiff cantilever rocking elements from 1 to 3.5. The plot indicates that rocking elements are sensitive to their stiffnesses (yield drift). This resulted in the development of the 3 separate rocking models. Rocking elements with yield drifts significantly higher than 1.2% (maximum aspect ratio of 4 for cantilever walls) are outside the scope of these guidelines.

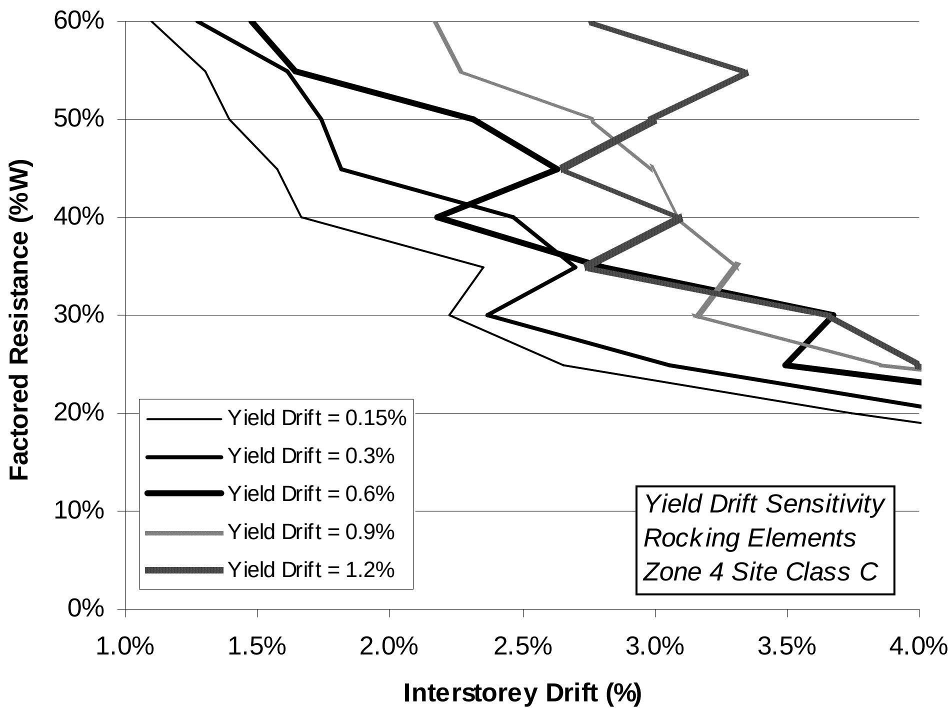


Figure C.6-24 Sensitivity to Yield Drift for Rocking Prototypes

C6.3 Toolbox Check

The Toolbox Method (see Section B3.2) allows the strength of different Lateral Deformation Resisting Systems (LDRSs) to be combined to limit the drift of the building to the Governing Drift Limit (GDL).

While this seems logical, the resistance tables were calculated based on the response of each prototype by itself. The response of the system may be different when combined with other systems.

Table C.6-21 shows results of analyses carried out to verify the validity of the toolbox assumption. A wide range of combinations and GDL were investigated, and the results indicate that the majority of the results were conservative (i.e. the combination of materials resulted in a lower maximum drift than the GDL).

In one case (C-5/R-1) the max. drift of one of the ground motions was larger than the GDL, and resulted in the mean +1s to be larger than the GDL. While the drift was greater than the GDL, it did not exceed it by a large amount. However, more work will have to be done to determine which combinations of systems work and which do not.

Table C.6-21 Toolbox Check Results

Prototypes	Weight Distribution	GDL	Total Strength (Unfactored)	Strength Distribution	Max. Drift Mean + 1σ
S1 W1 M1	0.3 - 0.3 - 0.4	2.0%	23.7%	9 - 8.67 - 6	1.46%
S1 W1 C1	0.3 - 0.3 - 0.4	3.0%	19.5%	5.85 - 4.6 - 9	2.31%
M2 R1	0.5 - 0.5	2.0%	33.3%	14.3 - 19	1.33%
W2 R1	0.5 - 0.5	4.0%	18.9%	9.35 - 9.5	2.83%
W2 W3	0.5 - 0.5	4.0%	16.2%	9.35 - 6.8	2.99%
C5 R1	0.5 - 0.5	1.5%	45.5%	25 - 20.5	1.60%
S2 M1	0.5 - 0.5	1.5%	26.0%	16.25 - 9.75	1.07%

C6.4 Comparison of Analysis with External Peer Review

The External Peer Reviews (EPR) for this project analyzed five of the prototypes independently. These analyses were conducted using the same suite of ground motions, but used a different computer program, unfactored resistances, and a different storey height (3.81m) than the results shown in Section C6.2. See the External Peer Review Report on the Bridging Guidelines CD for more details.

Quakesoft was used to analyze the same prototypes, with the same storey height as the EPR used, to determine the unfactored strengths, so that a direct comparison could be made. Table C.6-22 shows the results of the five prototypes and lists the EPR and UBC results.

Table C.6-22 Analysis Comparison

Prototype No.	**Prototype**	**Analysis Generator**	**Drift for Lateral Resistance**			
			30%W	*25%W*	*20%W*	*15%W*
W-1	Blocked OSB/Plywood	EPR	1.58%	1.53%	1.97%	3.35%
		UBC	1.48%	1.73%	1.84%	2.35%
		EPR/UBC	1.07	0.88	1.07	1.43
S-1	Braced Steel Frame (Tension Only)	EPR	1.77%	2.05%	2.66%	3.75%
		UBC	2.08%	2.07%	2.96%	3.88%
		EPR/UBC	0.85	0.99	0.90	0.97
M-1	Unreinforced Masonry	EPR	0.81%	0.85%	1.04%	1.44%
		UBC	0.79%	1.11%	1.27%	1.60%
		EPR/UBC	1.03	0.77	0.82	0.90
C-2	Concrete Shear Wall	EPR	0.80%	0.91%	U	U
		UBC	0.86%	1.24%	U	U
		EPR/UBC	0.93	0.73	-	-
R-1	Rocking	EPR	1.45%	1.52%	2.33%	2.74%
		UBC	1.35%	1.78%	2.14%	3.06%
		EPR/UBC	1.07	0.85	1.08	0.9

Notes: Drift results represented by "U" indicate LDRS instability.
Lateral resistances are unfactored.