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

## *ERRATA AND CRITIQUE*

MARCH, 2006

BRITISH COLUMBIA
Ministry of Education

APEG

## 1.0 INTRODUCTION

This document contains errata to the October version of the Bridging Guidelines Second Edition.

## 2.0 TECHNICAL ERRATA

The following errata affect the technical aspects of the Bridging Guidelines Second Edition.

**(1) Base Moment (Section 1.11)**

Replace Equation (1-2) with:

$$M_b = \sum K_{bm} \cdot R_{ei} \cdot h_s \qquad (1\text{-}2)$$

Where $h_s$ = inter-storey height

**(2) Combustible Non-load Bearing LDRSs (Section 1.1)**

Add the at the end of Section 1.1:

11) As part of a seismic retrofit, combustible non-load bearing LDRSs are permitted in non-combustible buildings.

**(3) Adjacency (Section 1.13)**

Revise Sentence 1.13(1) to read:

1) Two adjacent buildings with floors at different elevations (more than twice floor thickness) and separated by a horizontal distance …”.

**(4) Design of Foundations**

The following section should be added:

**Section 1.17 Foundations**

1) Foundations shall be designed in accordance with one of the following two options:

   (a) Strong foundations that develop the maximum lateral resistance of all supported LDRSs
   (b) Weaker foundations that rock before developing the maximum lateral resistance of all supported LDRSs

2) Foundations that are designed in accordance with Sentence 1.17(1)(a) shall have an overtstrength that is equal to or greater than the overstrength of all supported LDRSs.

3) Foundations that are designed in accordance with Sentence 1.17(1)(b) shall be designed as rocking elements.

4) The overstrength factor $R_o$ for fundations is 1.3.

**(5) Strength of Existing Wood-frame Materials**

Table 3.1A is an addendum to Table 3.1. Table 3.1A is located at the end of this document.

**(6) Short Columns (Sentence 5.1(5))**

The drift calculated in accordance with Sentence 5.5(1) should be prorated by $h_{sc}/h_s$, not $h_s/h_{sc}$.

**(7) Factored Resistance of Rocking Footing (Section 8.3)**

Replace Equation (8-4) with the following:

$$V_{rr} = \frac{\phi_r \cdot W_r \cdot l_r}{H} \tag{8-4}$$

where $W_r$ = Total weight of footing and building elements that are connected to and that rock with the footing (kN)

$l_r$ = Horizontal distance from toe of footing to centre of mass of $W_r$ (m)

**(8) Diaphragm Required Strength (Section 10.2)**

Sentence 10.2(1) is to be capitalized and reads:

1) THE DEFINITION OF $W_D$ IN SECTION 1.4 IS FOR THE WEIGHT OF THE COMPLETE DIAPHRAGM (INCLUDING 25% OF SNOW LOADING FOR ROOF DIAPHRAGM) AND THE WEIGHT OF THE TOTAL LENGTH OF WALLS SUPPORTED BY THE DIAPHRAGM ALONG THE EDGES OF THE DIAPHRAGM PERPENDICULAR TO THE DIRECTION OF SHAKING. The height of the supported wall is to the mid-height of the wall for each storey above and below the diaphragm. For a roof diaphragm, the height of the supported wall is from the mid-height of the top storey wall to the top of the wall or parapet.

Note that $R_{md}W_d$ is the required strength of the diaphragm, and not the seismic force acting on the diaphragm.

**(9) Diaphragm Connections (Section 11.3)**

Equation (11-1) is to be replaced with the following:

$R_{mc}$ is equal to the lesser of: (11-1)

$$R_{mc} = R_{ed} \div n_c \tag{11-1a}$$

$$R_{mc} = R_m \div n_c \tag{11-1b}$$

Where $R_m$ is the Minimum required factored resistance of LDRS that supports the diaphragm parallel to the direction of shaking.

**(10) Foundation Connections for Rocking (Section 11)**

Replace Section 11.6 and add Section 11.7 below:

**11.6 Minimum Factored Resistance – Rocking Foundation Connections**

1) The factored resistance of foundation connections, where thee foundation is permitted to rock, need not exceed the maximum factored forces due to rocking.

**11.7 Calculation of Connection Resistance**

1) The factored resistance of a connection is to be calculated in accordance with the manufacturer's recommendations and best current practice.

**(11) John Wallace's November Seminar Diaphragm Presentation (Page 5)**

On page 5 the total weight of the diaphragm, $W_d$, was calculated to be 1500 kN. On the next slide the end wall force is calculated with only half the diaphragm weight. This is incorrect, the load on the end walls is 270 kN, and the required shear strength of the diaphragm is 12.3 kN/m.

**(12) Short Columns (Sentence A5.1(5))**

Revise second sentence to read ".. and prorated by $h_{sc}/h_s$, are …". Add a third sentence "Secondary vertical support elements should be added to prevent loss of vertical support when short columns can fail in a brittle manner."

**(13) Development Length (Section A5.3)**

Revise third and fourth sentences to read:

"The tensile strength of all reinforcing bars that do not meet the lap splice requirements of CSA/A23.3-04 shall be prorated accordingly. This reduction in strength could result in shearwall rocking."

**(14) Cantilever Diaphragms (Section A10.2)**

Revise Section A10.2 to read:

For clarification, the definition of $W_d$ is illustrated in Figure A.10-2. For cantilever roof diaphragms, $W_d$ is twice the combined weight of the cantilever portion of the roof and the wall supported out- of-plane by the roof cantilever. Also for cantilever diaphragms, use double the span length when referencing the appropriate resistance table.

## 3.0 EDITORIAL ERRATA

The following errata are editorial in nature.

Page 1-4 – Definition of *Pier*: change to “that is free *to* rock”

Page 1-6 - Notations $F_c$, $R_{mc}$ and $R_{md}$ should all be "minimum required factored resistance".

Page 1-7 – Sentence 1.8(1) For clarity, this Section, Section 2.3 (Toolbox) and the corresponding sections in the material sections should be amended to be consistent with a revised second sentence that reads "A building poses an acceptable level of risk if the assessment procedure given in Section 2.3 determines that $R_{rt}$, the sum of the factored resistance ratios ($R_r$) for all LDRSs, equals or exceeds 80%.". With this change in wording, the engineer only uses 100% of the values in the resistance tables in a calculation whose end result is used for both assessment (>=80%) and retrofit (>=100%).

Page 1-8 - Sentence should be revised to read "..the recommended minimum required factored resistance of the second storey is equal to the average of the lateral factored resistances of the first and top storeys".

Page 2-1 – Section 2.2(1)(b) Should read “LDRSs spaced *no* more…”

Page 4-1(2) – Sentence 4.1(2) Change first sentence to “For prototype S-2, a single tension/compression brace is not permitted in a single bay or in a multiply bay line.”

Page 4-8 – Table 4-2(a) – Change Legend to C, D/E

Page 6-2 – Section 6.5(4)(a) “Walls fully *confined* …”

Page 10-2 – Sentence 10.4(1) Add a second sentence to this part: “The calculation of the lateral resistance of an existing wood diaphragm for materials not addressed in CAN/CSA-O86-05 but listed in Table 3.1 shall be calculated using the appropriate value from Table 3.1”.

Page 10-4 – Section 10.9 – Rename Section to “Global Inelastic Strain Limitation.”

Page 10-10 – Table 10-6(c) – Graph does not include values for 40 and 50m spans. Used the tabulated values.

Page A-42 – Section A10.0 – Rename Section to “Global Inelastic Strain Limitations”. Rename Table A.10-1 to “Maximum Diaphragm Local Inelastic Strain Limits”

## 4.0 DIAPHRAGM RESISTANCE TABLES

Diaphragm resistance tables for Seismic Zones 2, 3 and 5 are attached at the end of this document.

**END OF ERRATA**

**Table 3.1A** Lateral Factored Resistance of Additional Selected Existing Materials

| | Existing Material and Construction | Factored Capacity (kN/m) |
|---|---|---|
| 1. See note 1. | Transverse board (shiplap) sheathing<br>6" wide boards on framing at 400 c/c<br>6" wide boards on framing at 600 c/c | <br>2.0<br>1.4 |
| 2. See note 2. | Diagonal board (shiplap) sheathing<br>with 2 – 64mm common nails to supports<br>6" wide boards<br>8" wide boards | <br><br>3.5 (8.9 upgraded)<br>2.7 (6.7 upgraded) |
| 3. See note 3. | 38mm T & G decking | 0.6 |
| 4. See note 4. | 64/89m T & G decking with no side spiking | 0.9 |
| 5. See note 5. | 64/89m T & G decking with 6mm Ø x 200mm long side spikes at 750mm c/c | 5.2 |
| 6. See note 6. | Shiplap or T & G floors with ply sub-floor and/or hardwood flooring | add 1-3.0kN/m to applicable valves above |
| 7. See note 7. | ½" drywall sheathing<br>on framing at 400mm<br>on framing at 600mm | <br>0.8<br>0.5 |

*Notes:*

General

Values for some existing materials utilized as diaphragms and shearwalls are tabulated in a wide variety of FEMA, ATC, NRC and other references. The most consistent observation about these various documents is that the values provided are inconsistent. Values are given in a variety of manners that do not correlate well for use by the Bridging Guidelines and current wood design code. In particular, avoid using 'strength values' provided in the 1992 NBC Guidelines for Seismic Evaluation of existing buildings and other sources.

*Notes continued on next page.*

1. Values are based on the methodology and results provided in the 1980 Timber Design Manual (others also). The unfactored shear values given havebeen increased prorate to reflect the factored nail values provided in the 2005 Wood Design Manual. The values assume 2-64mm common nails per 6” nominal wide boards to support framing. Values for 8” nominal boards can be taken as 15 percent larger than those provided for 6” boards.

2. Values are based on the methodology and results provided in the 1980 Timber Design Manual. The unfactored shear values given have been increased prorate to reflect the factored nail values provided in the 2005 Wood Design Manual. The values assume 2-64mm common nails for both 6” and 8” nominal board widths to support framing. The lower values given are reasonable for assessment purposes. They are approximately 40 percent of the upgraded values, reflecting inadequate nail end distance and detailing typically expected at the diaphragm/shearwall boundaries. The upgrade values are for use where the boundary conditions are provided to develop full nail capacity. Note that per the 1980 Timber Design Manual, further increases in shear capacity are possible with more nailing throughout the diaphragm/shearwall areas, including special attention to boundaries and butt ends of boards. All diaphragms/shearwalls must be upgraded to provide tension/compression chords in accordance with good engineering practice.

3. Value is based on the same methodology as described for transverse board sheathing, assuming a 1.8m span. The value is directly proportional to span and may be adjusted accordingly. For example a 1.2m span increases the capacity by 50 percent while a 2.4m span reduces the capacity to 75 percent of that tabulated.

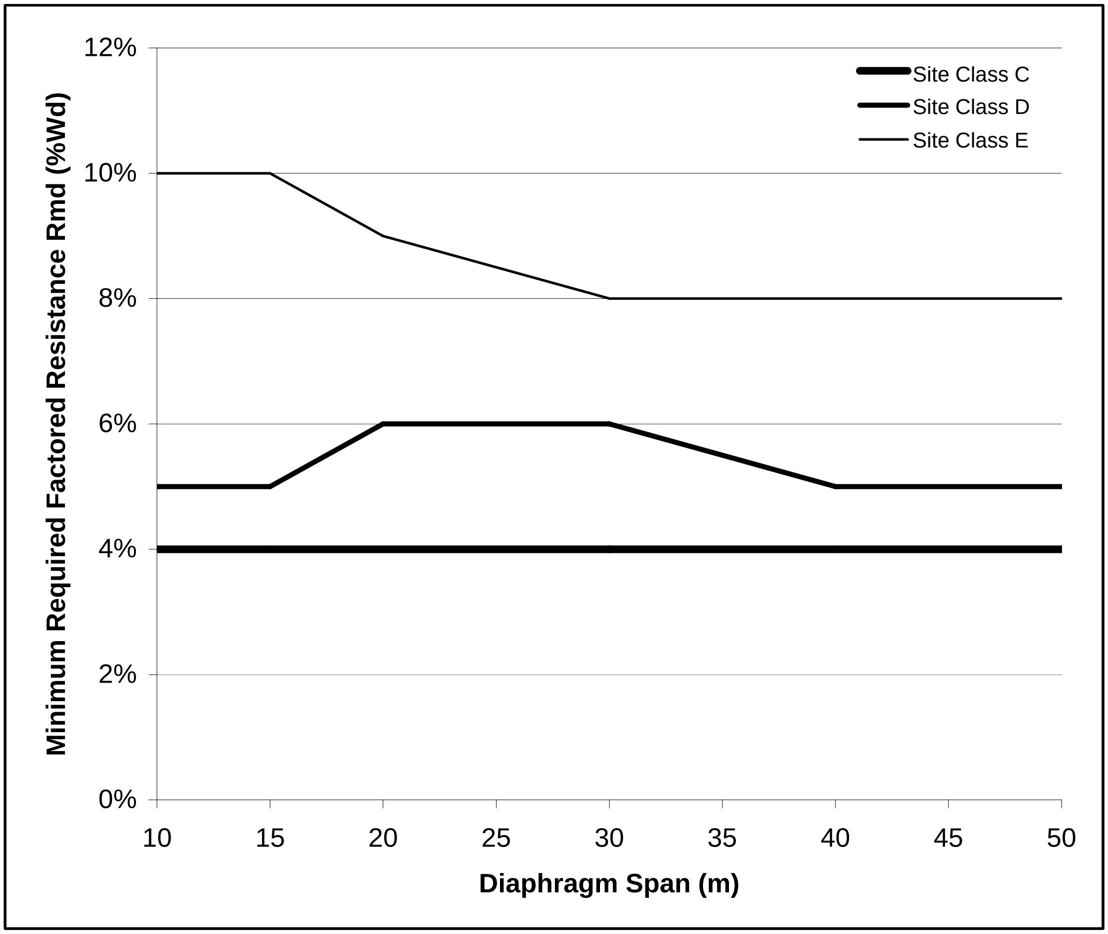
4. Similar to point 3 above assuming a 3.0m span.

5. Value is the sum of point 4.0 above and the lateral capacity of the side spikes. Both old and newer timber reference manuals specify 200mm long by 6mm diameter of side spikes between decking boards at 750mm c/c through predrilled holes provided in the decking. Although the presence of side spikes is expected (required usually to obtain tight and straight joints during installation) some decks may only be partially spiked. The strength of decks with less than complete side spiking can be obtained prorate based on 4.3kN/m for side spikes at 750mm c/c.

6. Value needs to be assessed on an individual basis to reflect variability of floor systems.

7. Values taken from the 2005 Wood Design Manual for unblocked drywall with nails at panel edge at 200mm c/c. Values for 16mm drywall are also provided in the Manual. Suggest using a value for 19mm plaster walls as for 16mm drywall assuming fastening at 200mm c/c on panel edges. i.e. 1.1 kN/m for framing at 400 c/c.

**D-1 Zone 2**

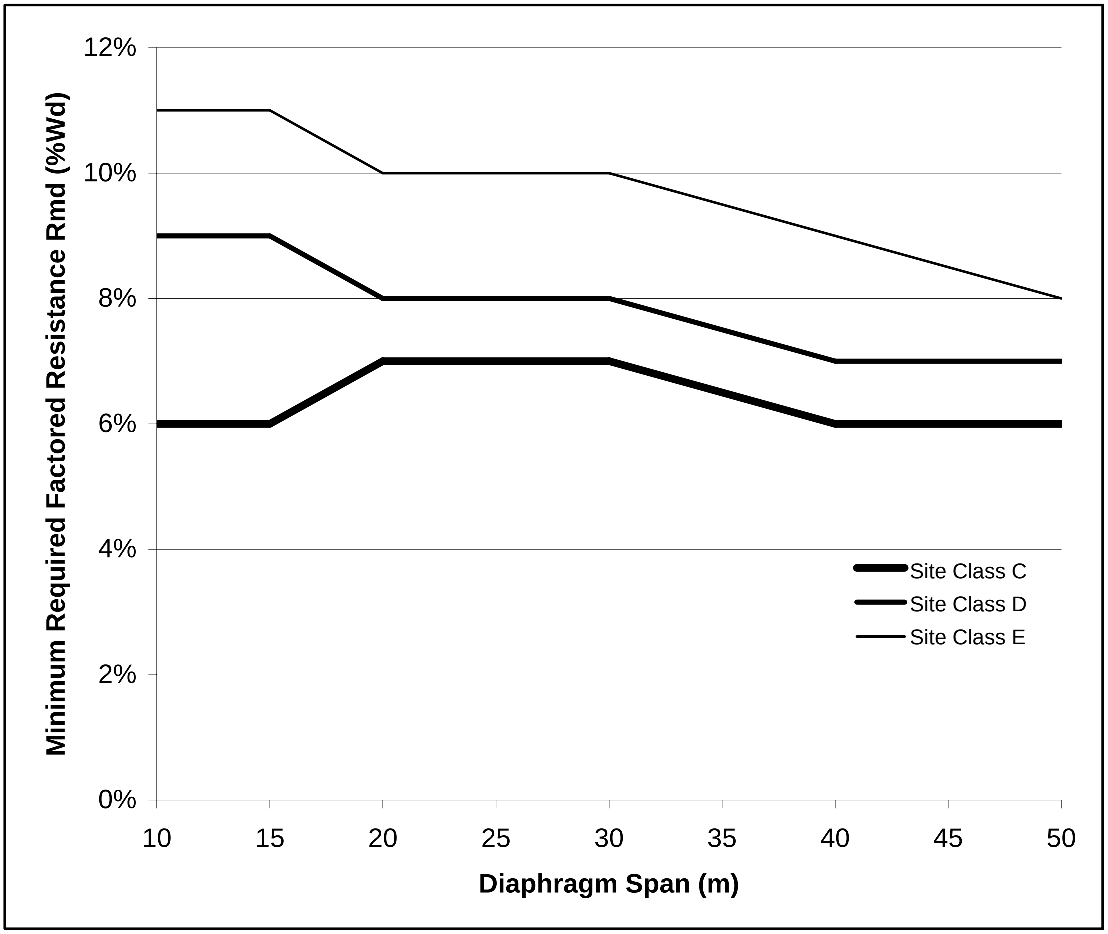



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 4% | 4% | 4% | 4% | 4% | 4% |
| *D* | 5% | 5% | 6% | 6% | 5% | 5% |
| *E* | 10% | 10% | 9% | 8% | 8% | 8% |

**Figure 10-1(a)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-1 Blocked Plywood/OSB*
*Seismic Zone 2 (Princeton)*

**D-1 Zone 3**

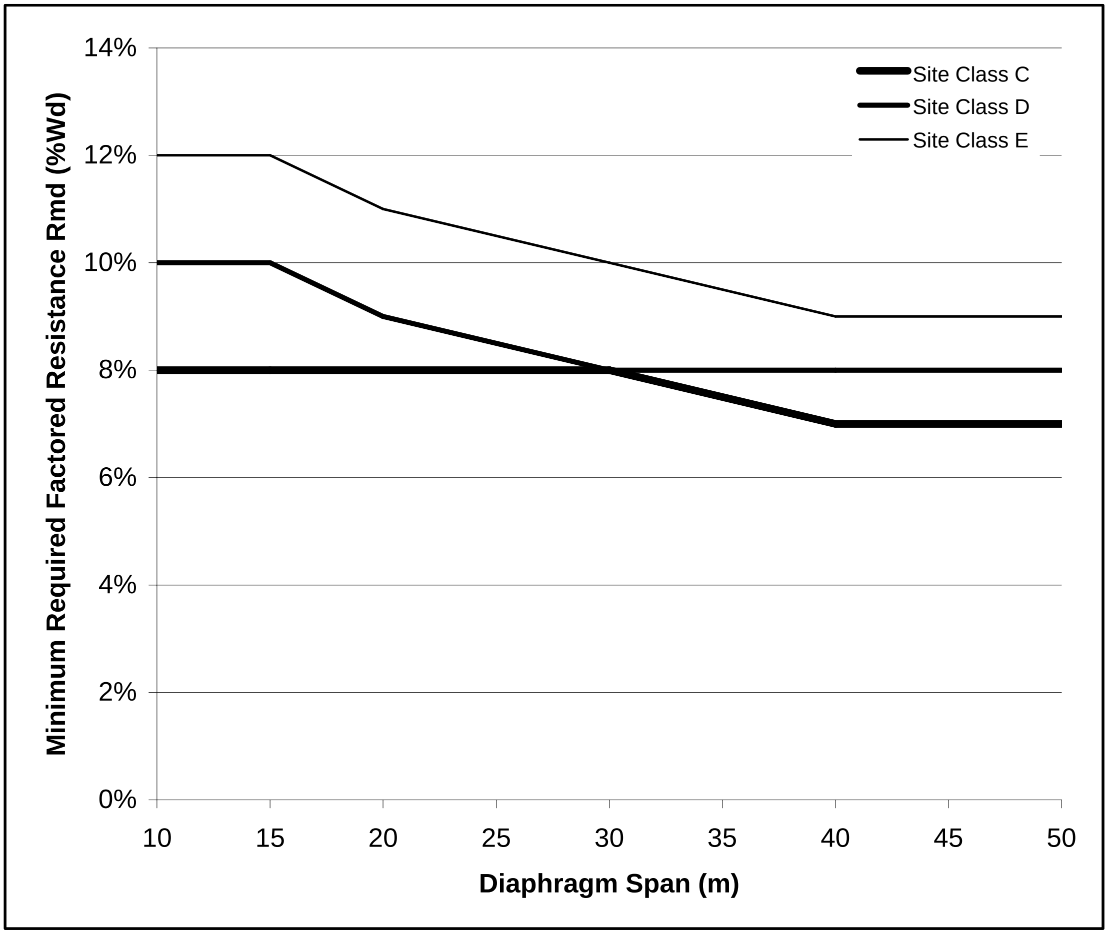



| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 6% | 6% | 7% | 7% | 6% | 6% |
| *D* | 9% | 9% | 8% | 8% | 7% | 7% |
| *E* | 11% | 11% | 10% | 10% | 9% | 8% |

**Figure 10-1(b)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (% $W_d$)*
*Diaphragm Prototype D-1 Blocked Plywood/OSB*
*Seismic Zone 3 (Chilliwack)*

**D-1 Zone 4**

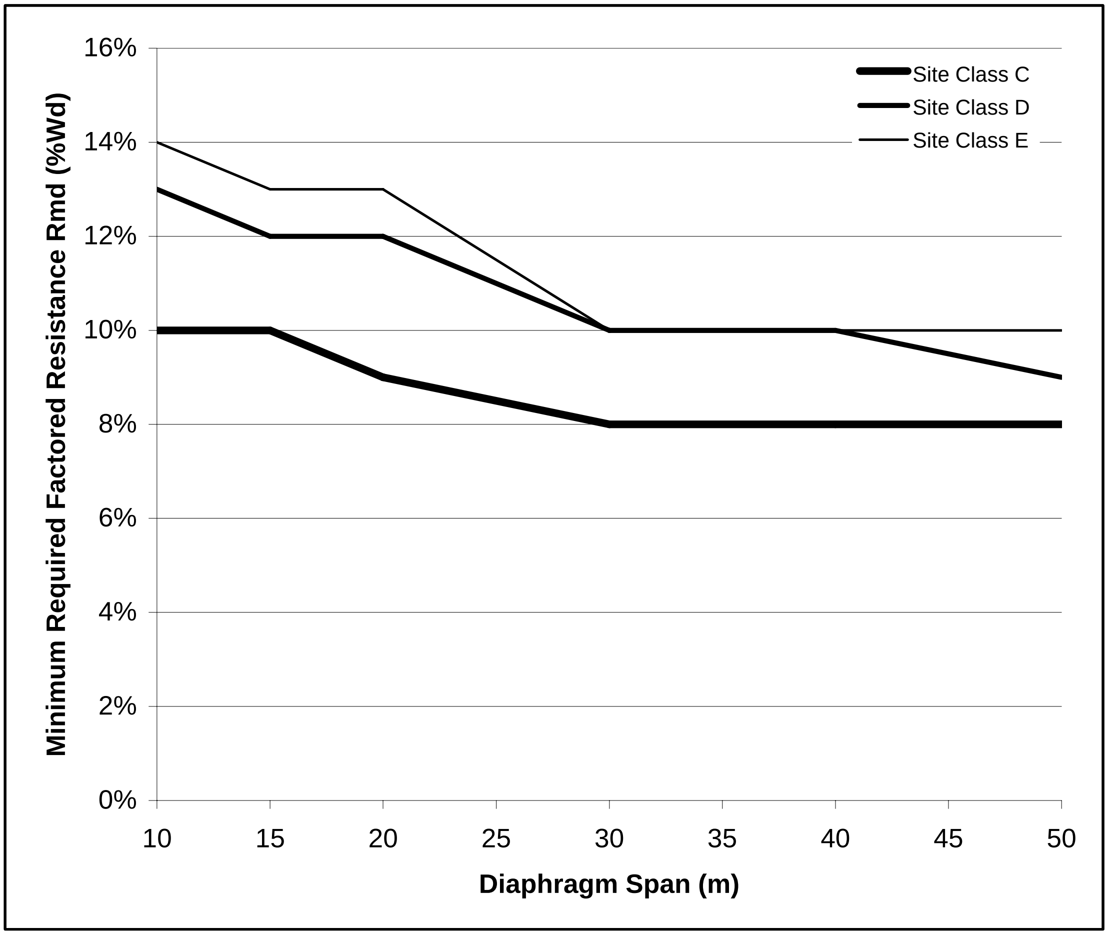



| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 8% | 8% | 8% | 8% | 7% | 7% |
| *D* | 10% | 10% | 9% | 8% | 8% | 8% |
| *E* | 12% | 12% | 11% | 10% | 9% | 9% |

**Figure 10-1(c)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-1 Blocked Plywood/OSB*
*Seismic Zone 4 (Vancouver)*

**D-1 Zone 5**

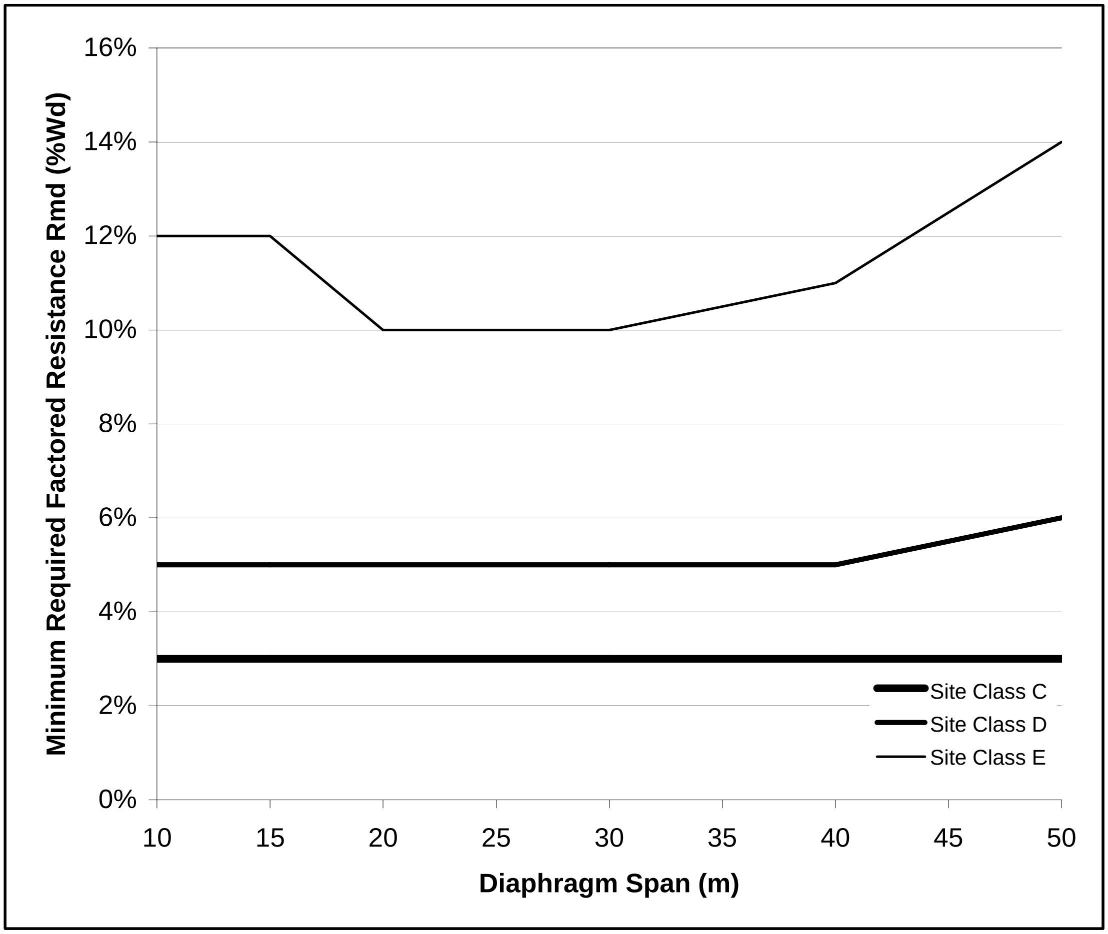



| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 10% | 10% | 9% | 8% | 8% | 8% |
| *D* | 13% | 12% | 12% | 10% | 10% | 9% |
| *E* | 14% | 13% | 13% | 10% | 10% | 10% |

**Figure 10-1(d)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-1 Blocked Plywood/OSB*
*Seismic Zone 5 (Victoria)*

**D-2 Zone 2**

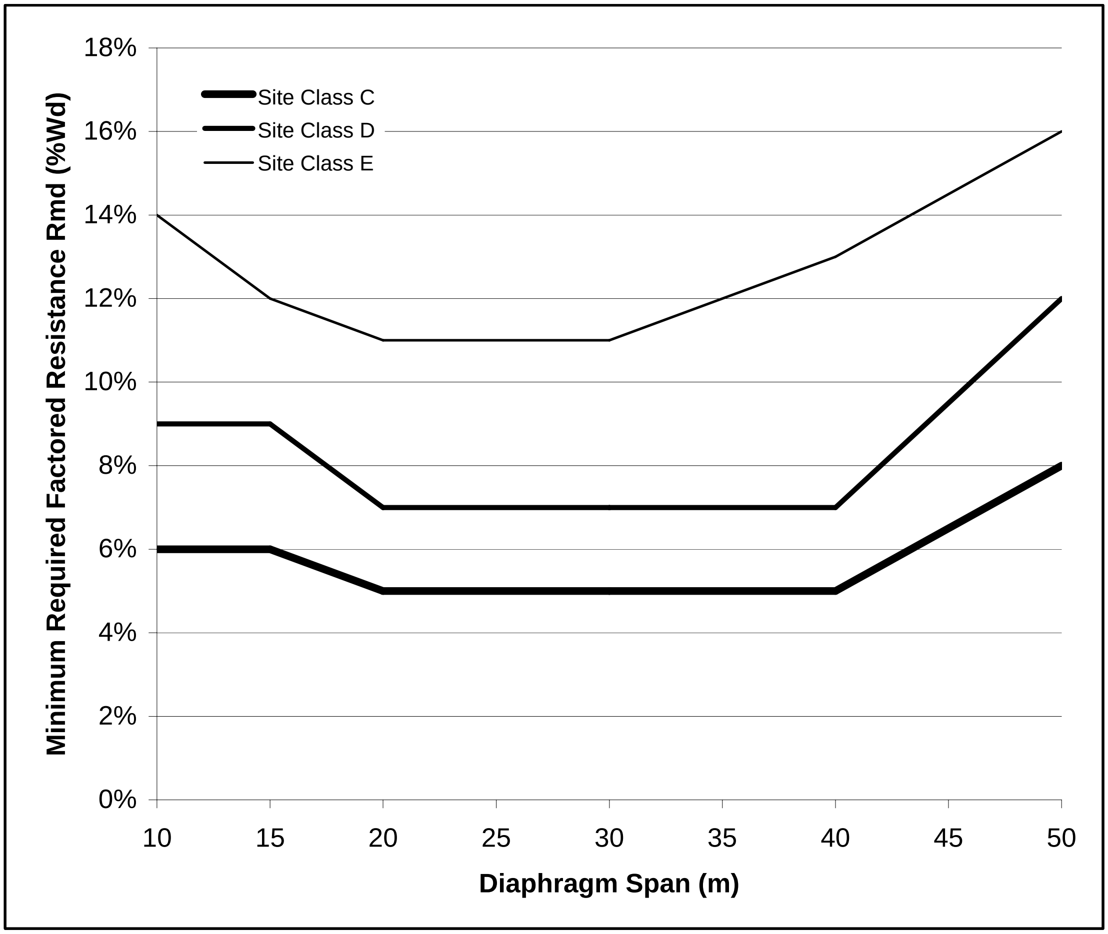



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 3% | 3% | 3% | 3% | 3% | 3% |
| *D* | 5% | 5% | 5% | 5% | 5% | 6% |
| *E* | 12% | 12% | 10% | 10% | 11% | 14% |

**Figure 10-2(a)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-2 Unblocked Plywood/OSB*
*Seismic Zone 2 (Princeton)*

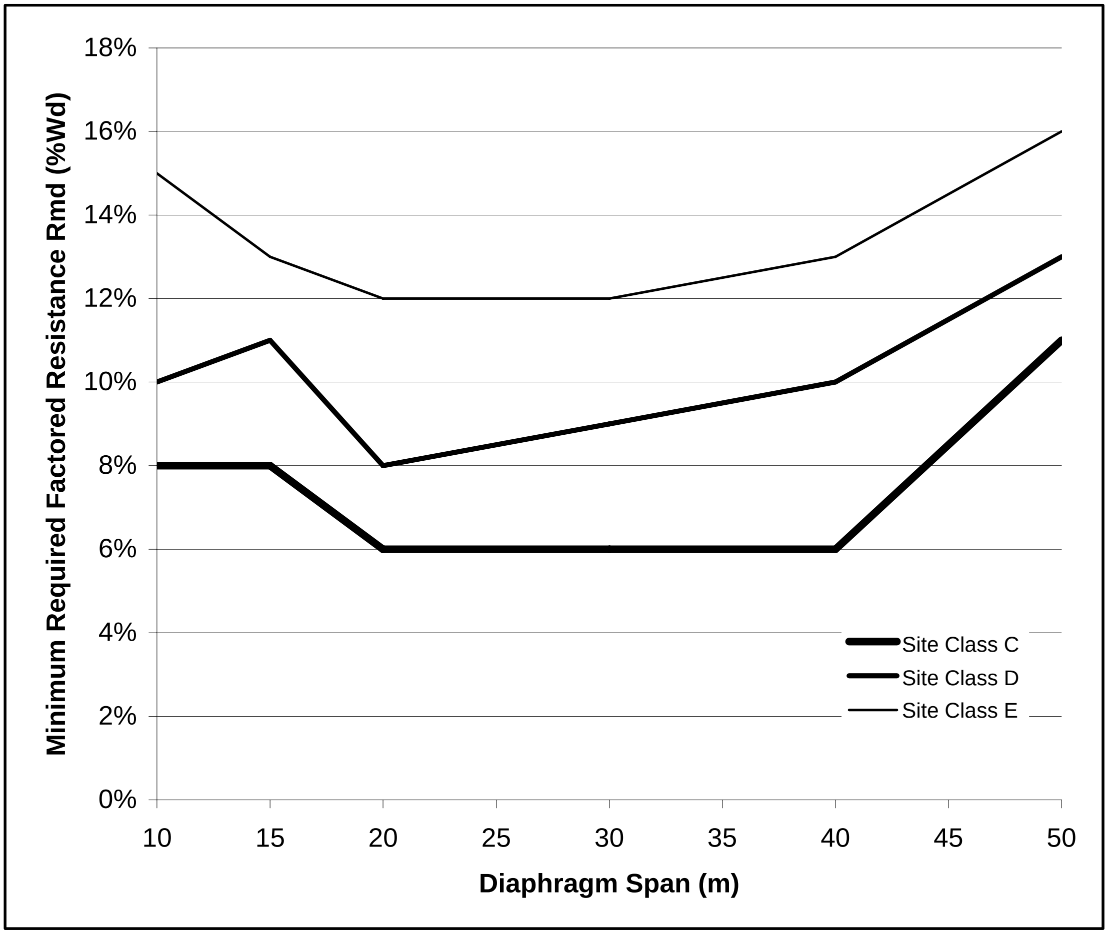
**D-2 Zone 3**




| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 6% | 6% | 5% | 5% | 5% | 8% |
| *D* | 9% | 9% | 7% | 7% | 7% | 12% |
| *E* | 14% | 12% | 11% | 11% | 13% | 16% |

**Figure 10-2(b)** *Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-2 Unblocked Plywood/OSB*
*Seismic Zone 3 (Chilliwack)*

**D-2 Zone 4**

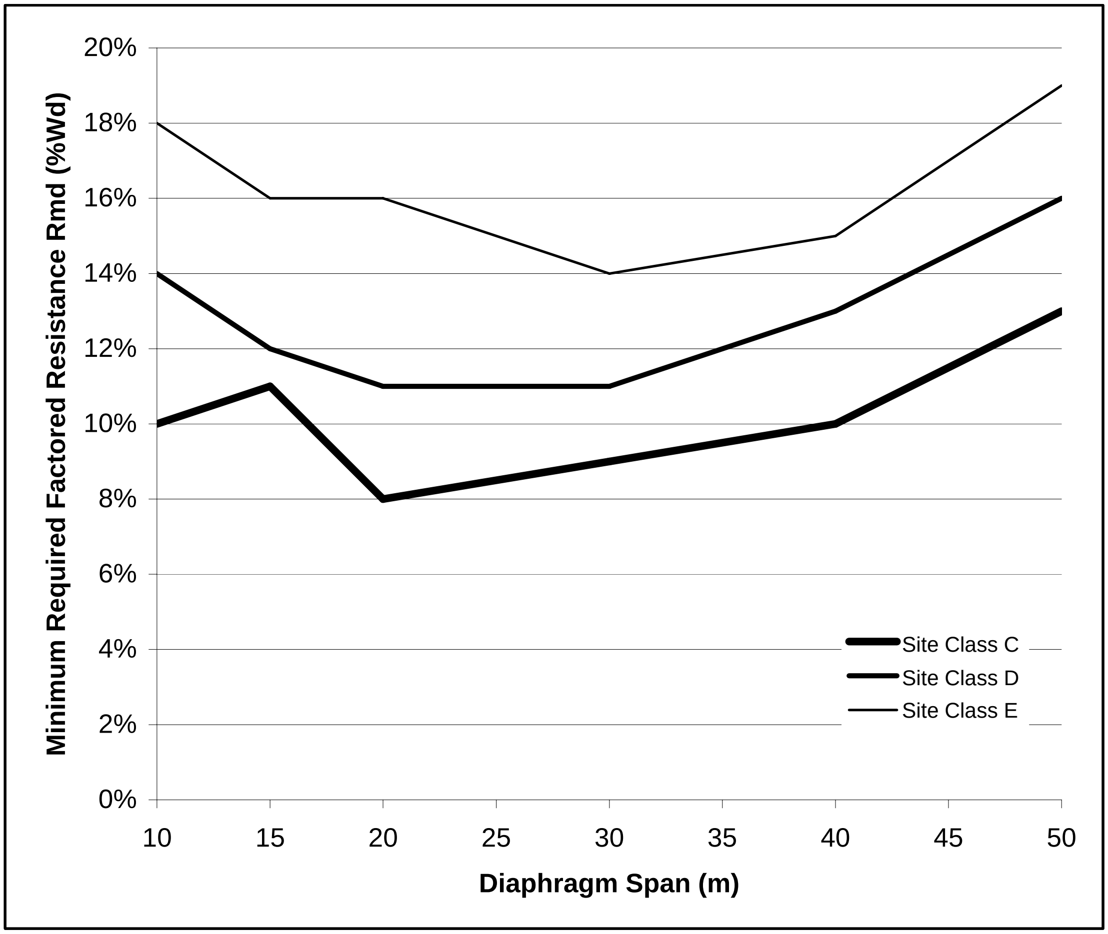



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 8% | 8% | 6% | 6% | 6% | 11% |
| *D* | 10% | 11% | 8% | 9% | 10% | 13% |
| *E* | 15% | 13% | 12% | 12% | 13% | 16% |

**Figure 10-2(c)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-2 Unblocked Plywood/OSB*
*Seismic Zone 4 (Vancouver)*

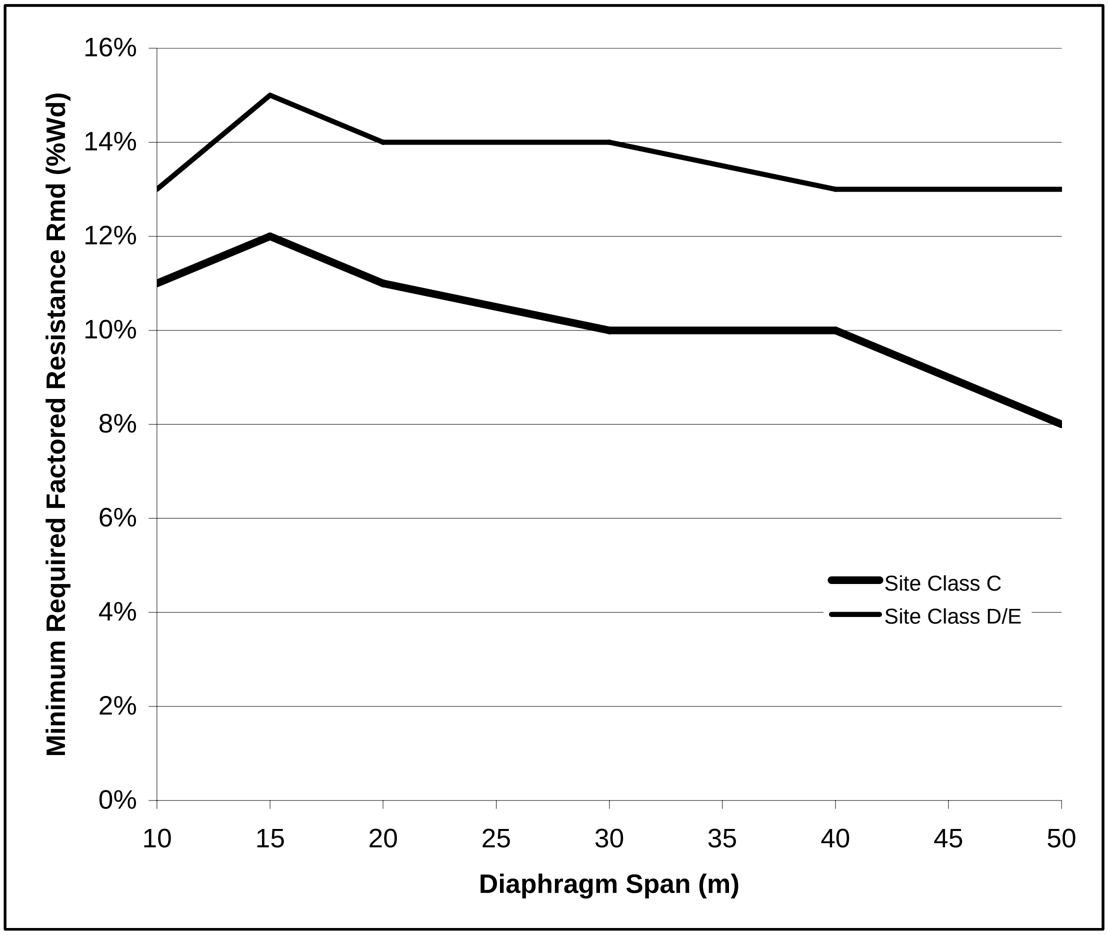
**D-2 Zone 5**




| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 10% | 11% | 8% | 9% | 10% | 13% |
| *D* | 14% | 12% | 11% | 11% | 13% | 16% |
| *E* | 18% | 16% | 16% | 14% | 15% | 19% |

**Figure 10-2(d)** *Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-2 Unblocked Plywood/OSB*
*Seismic Zone 5 (Victoria)*

**D-3 Zone 2**

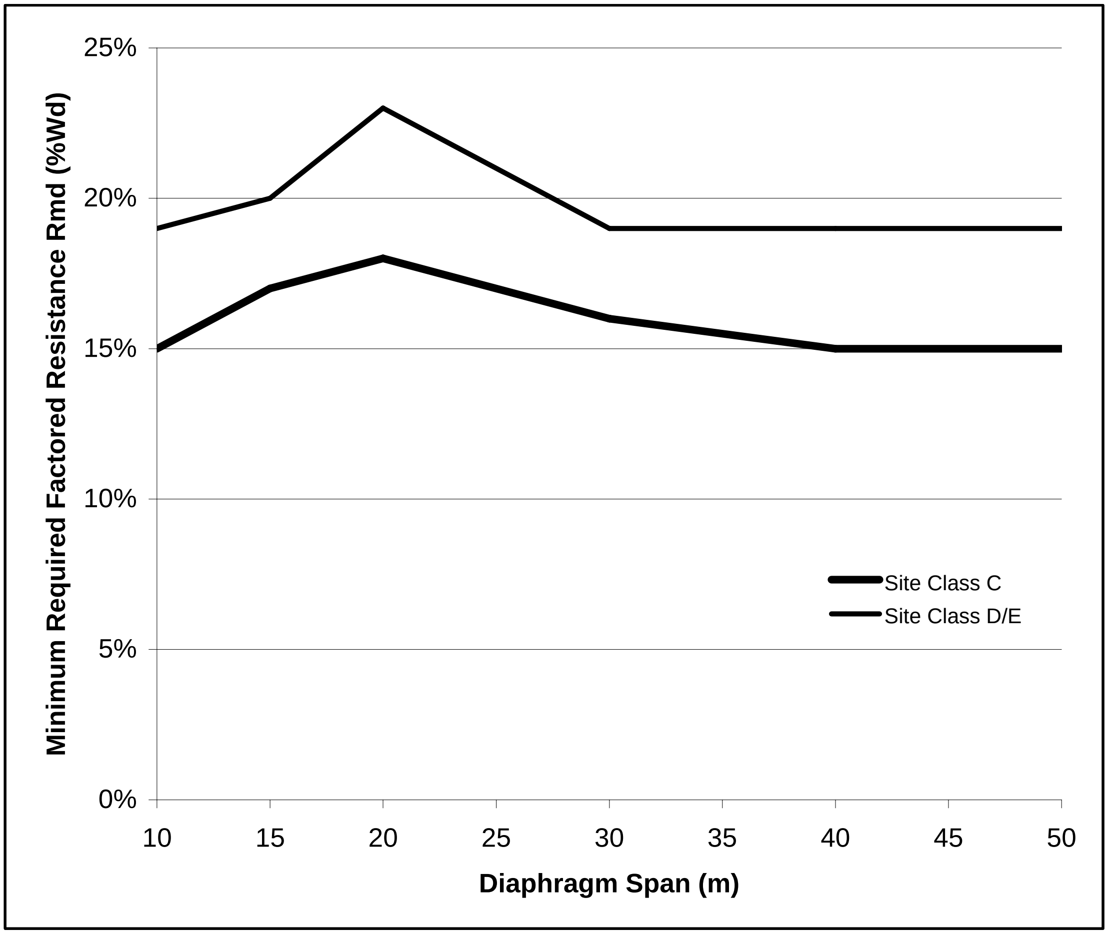



| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 11% | 12% | 11% | 10% | 10% | 8% |
| *D* | 13% | 15% | 14% | 14% | 13% | 13% |
| *E* | 13% | 15% | 14% | 14% | 13% | 13% |

**Figure 10-3(a)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-3 Steel Deck Type A*
*Seismic Zone 2 (Princeton)*

## D-3 Zone 3

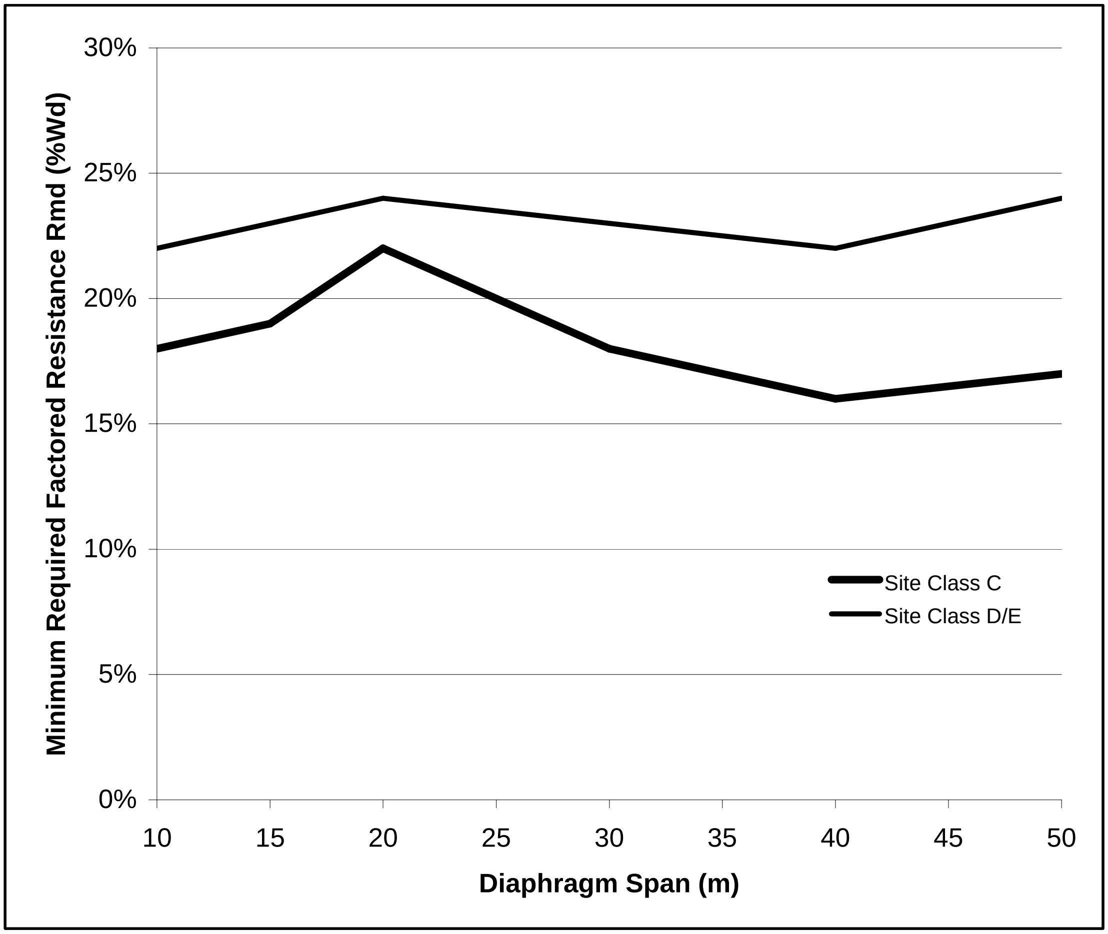



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 15% | 17% | 18% | 16% | 15% | 15% |
| *D* | 19% | 20% | 23% | 19% | 19% | 19% |
| *E* | 19% | 20% | 23% | 19% | 19% | 19% |

**Figure 10-3(b)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-3 Steel Deck Type A*
*Seismic Zone 3 (Chilliwack)*

**D-3 Zone 4**

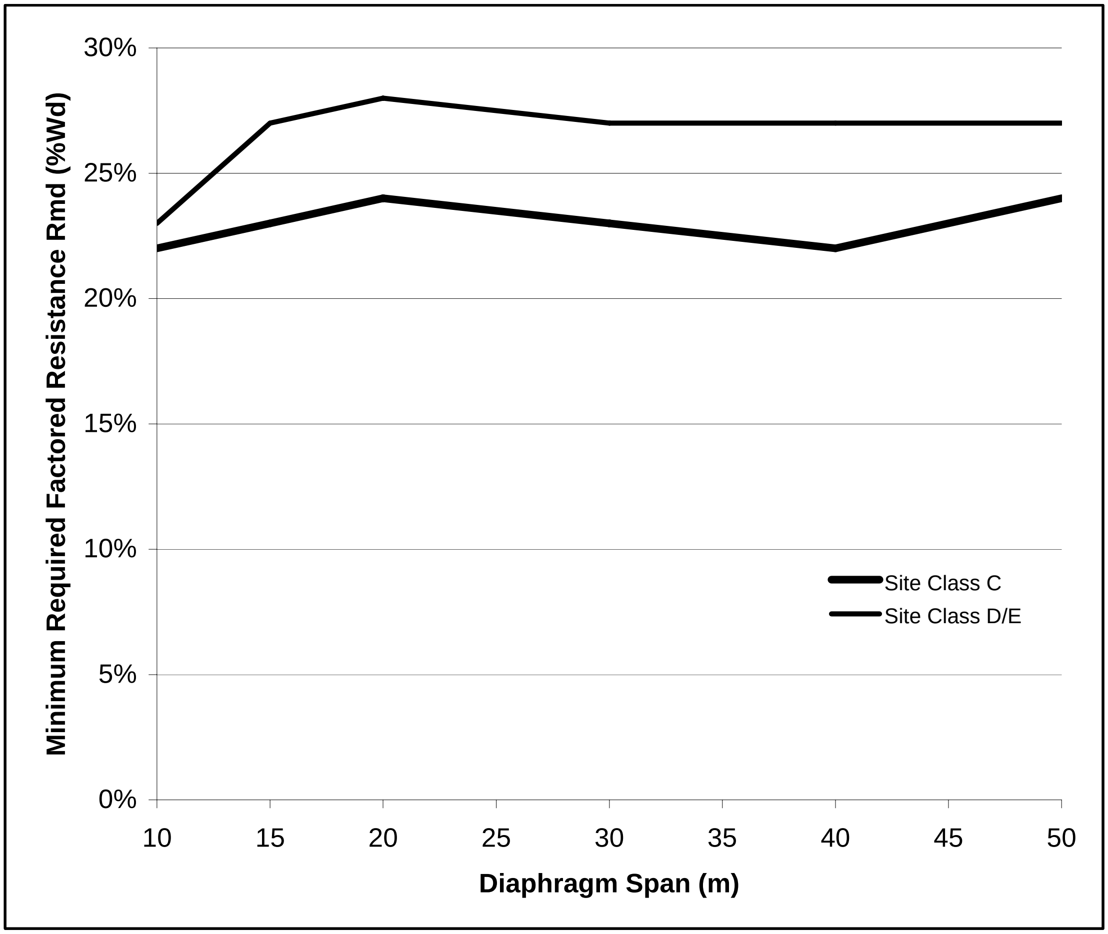



| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 18% | 19% | 22% | 18% | 16% | 17% |
| *D* | 22% | 23% | 24% | 23% | 22% | 24% |
| *E* | 22% | 23% | 24% | 23% | 22% | 24% |

**Figure 10-3(c)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-3 Steel Deck Type A*
*Seismic Zone 4 (Vancouver)*

**D-3 Zone 5**

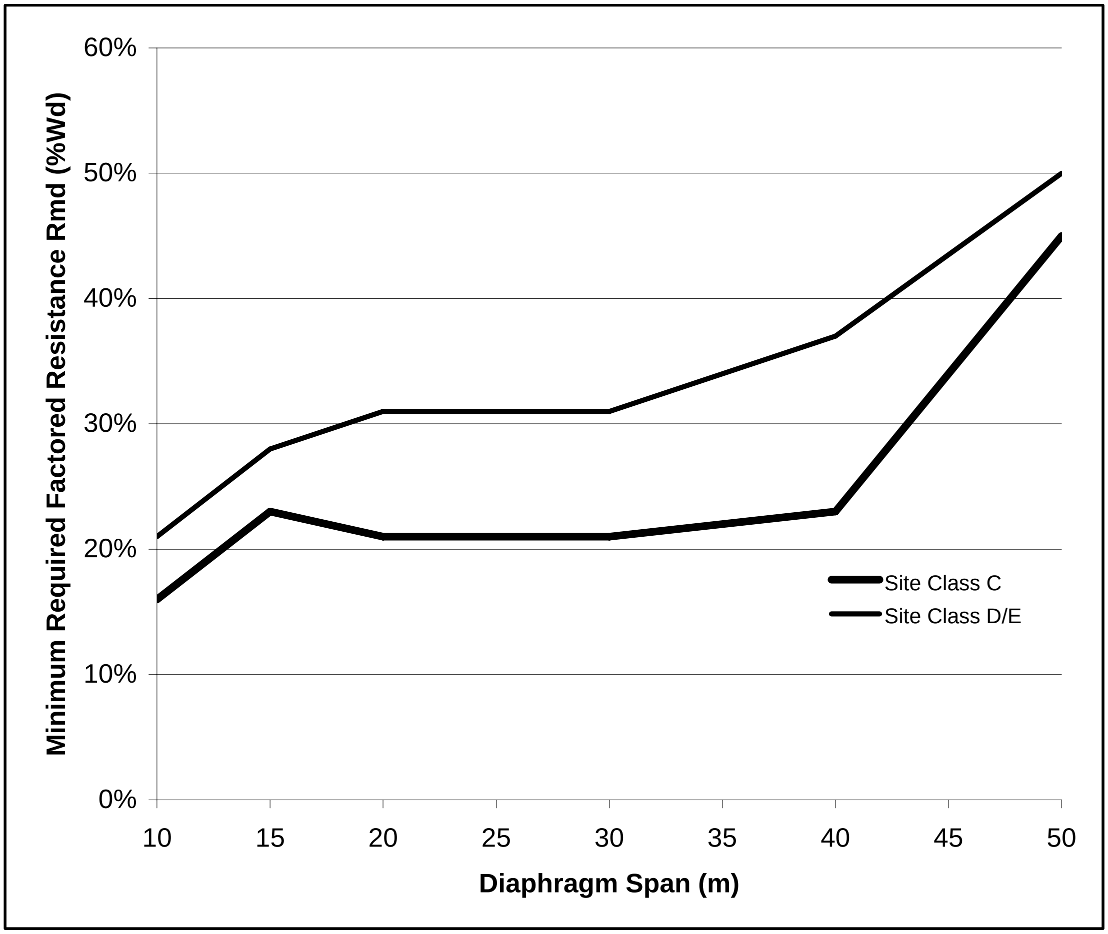



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 22% | 23% | 24% | 23% | 22% | 24% |
| *D* | 23% | 27% | 28% | 27% | 27% | 27% |
| *E* | 23% | 27% | 28% | 27% | 27% | 27% |

**Figure 10-3(d)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-3 Steel Deck Type A*
*Seismic Zone 5 (Victoria)*

**D-4 Zone 2**

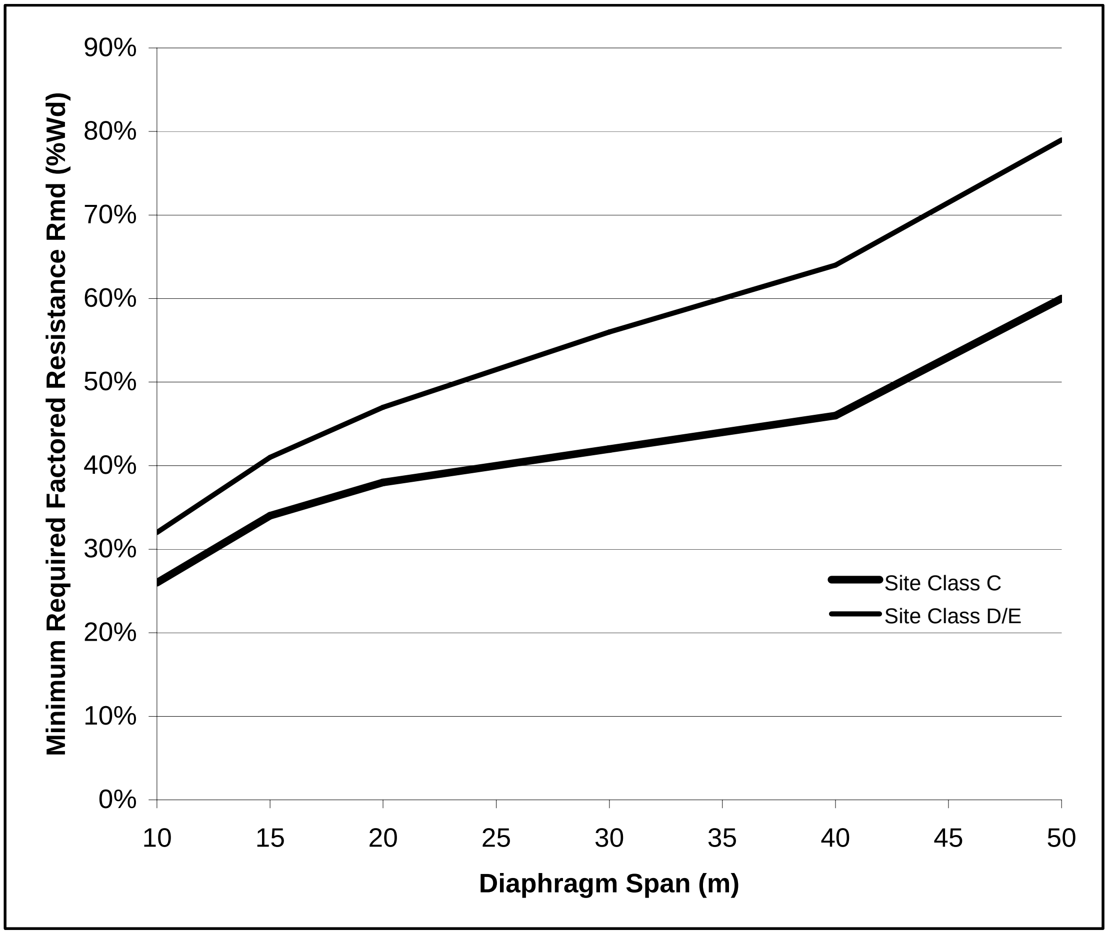



| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 16% | 23% | 21% | 21% | 23% | 45% |
| *D* | 21% | 28% | 31% | 31% | 37% | 50% |
| *E* | 21% | 28% | 31% | 31% | 37% | 50% |

**Figure 10-4(a)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-4 Steel Deck Type B*
*Seismic Zone 2 (Princeton)*

## D-4 Zone 3

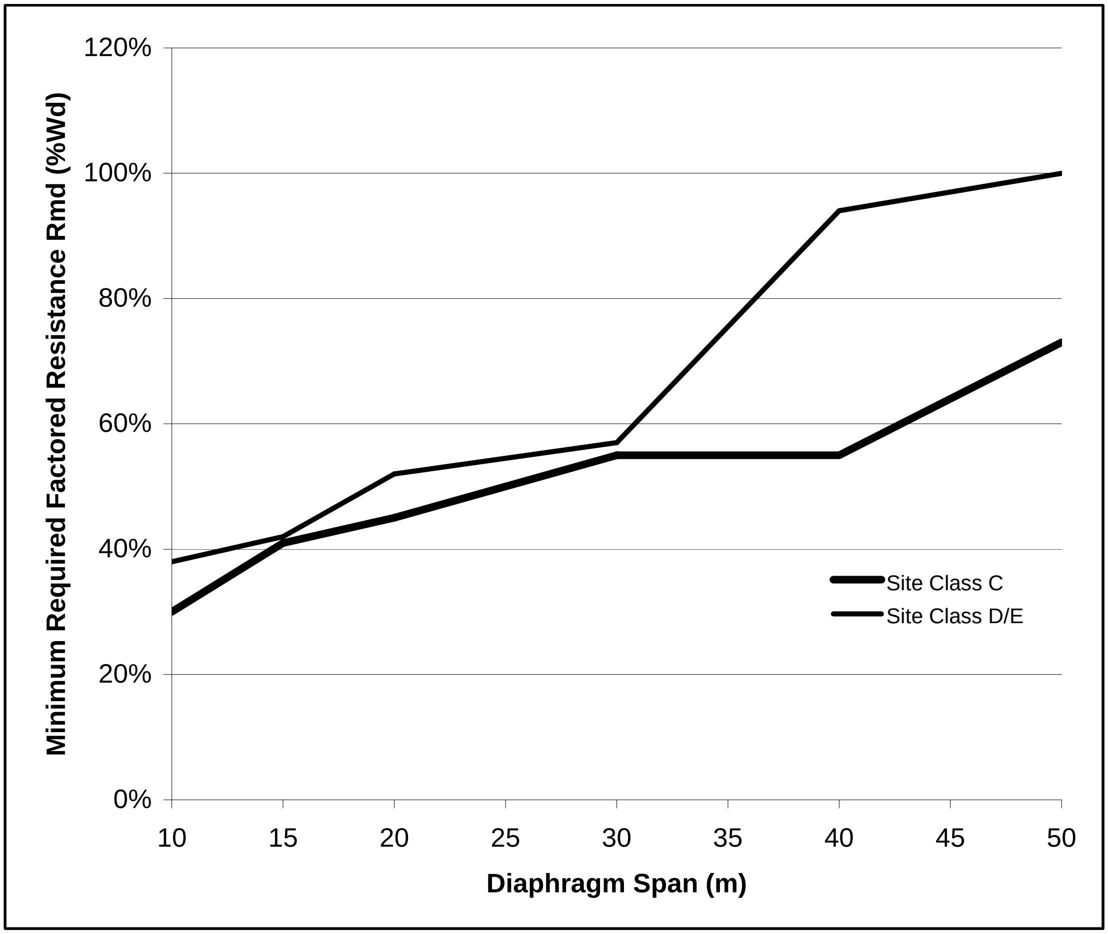



| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 26% | 34% | 38% | 42% | 46% | 60% |
| *D* | 32% | 41% | 47% | 56% | 64% | 79% |
| *E* | 32% | 41% | 47% | 56% | 64% | 79% |

**Figure 10-4(b)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-4 Steel Deck Type B*
*Seismic Zone 3 (Chilliwack)*

## D-4 Zone 4




| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 30% | 41% | 45% | 55% | 55% | 73% |
| *D* | 38% | 42% | 52% | 57% | 94% | 100% |
| *E* | 38% | 42% | 52% | 57% | 94% | 100% |

**Figure 10-4(c)**

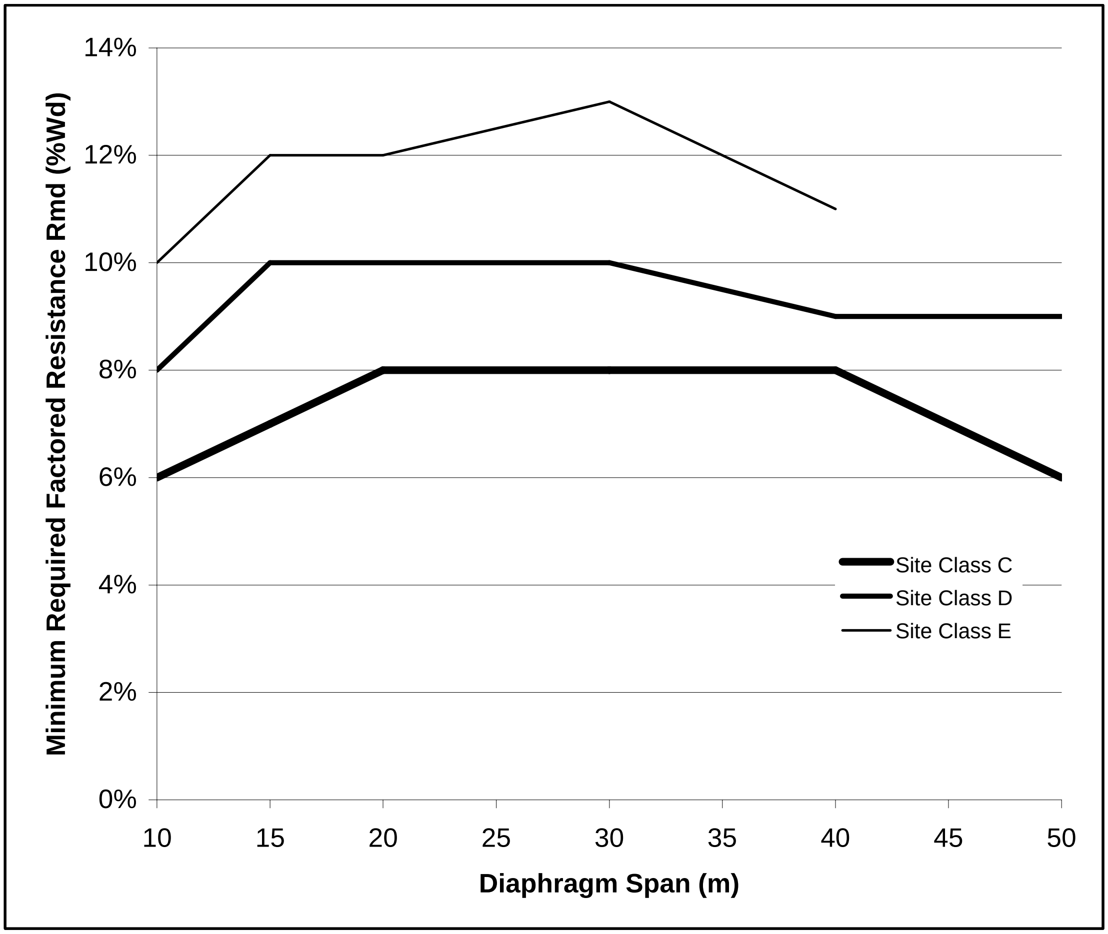
*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-4 Steel Deck Type B*
*Seismic Zone 4 (Vancouver)*


| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 38% | 42% | 52% | 57% | 94% | 100% |
| *D* | 38% | 54% | 50% | 66% | 116% | 133% |
| *E* | 38% | 54% | 50% | 66% | 116% | 133% |

**Figure 10-4(d)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-4 Steel Deck Type B*
*Seismic Zone 5 (Victoria)*

**D-5 Zone 2**

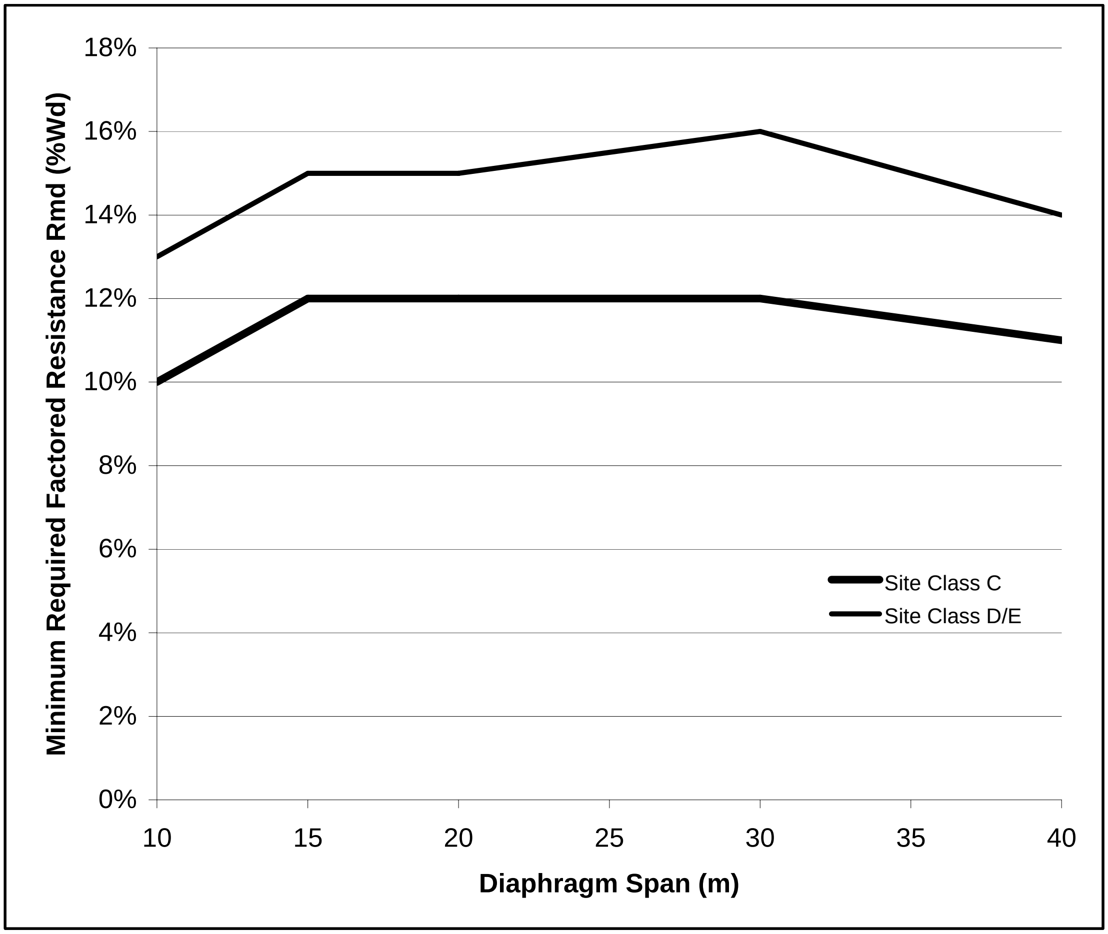



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 6% | 7% | 8% | 8% | 8% | 6% |
| *D* | 8% | 10% | 10% | 10% | 9% | 9% |
| *E* | 10% | 12% | 12% | 13% | 11% | NR |

**Figure 10-5(a)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ ($\%W_d$)*
*Diaphragm Prototype D-5 Steel Bracing (Tension Only)*
*Seismic Zone 2 (Princeton)*

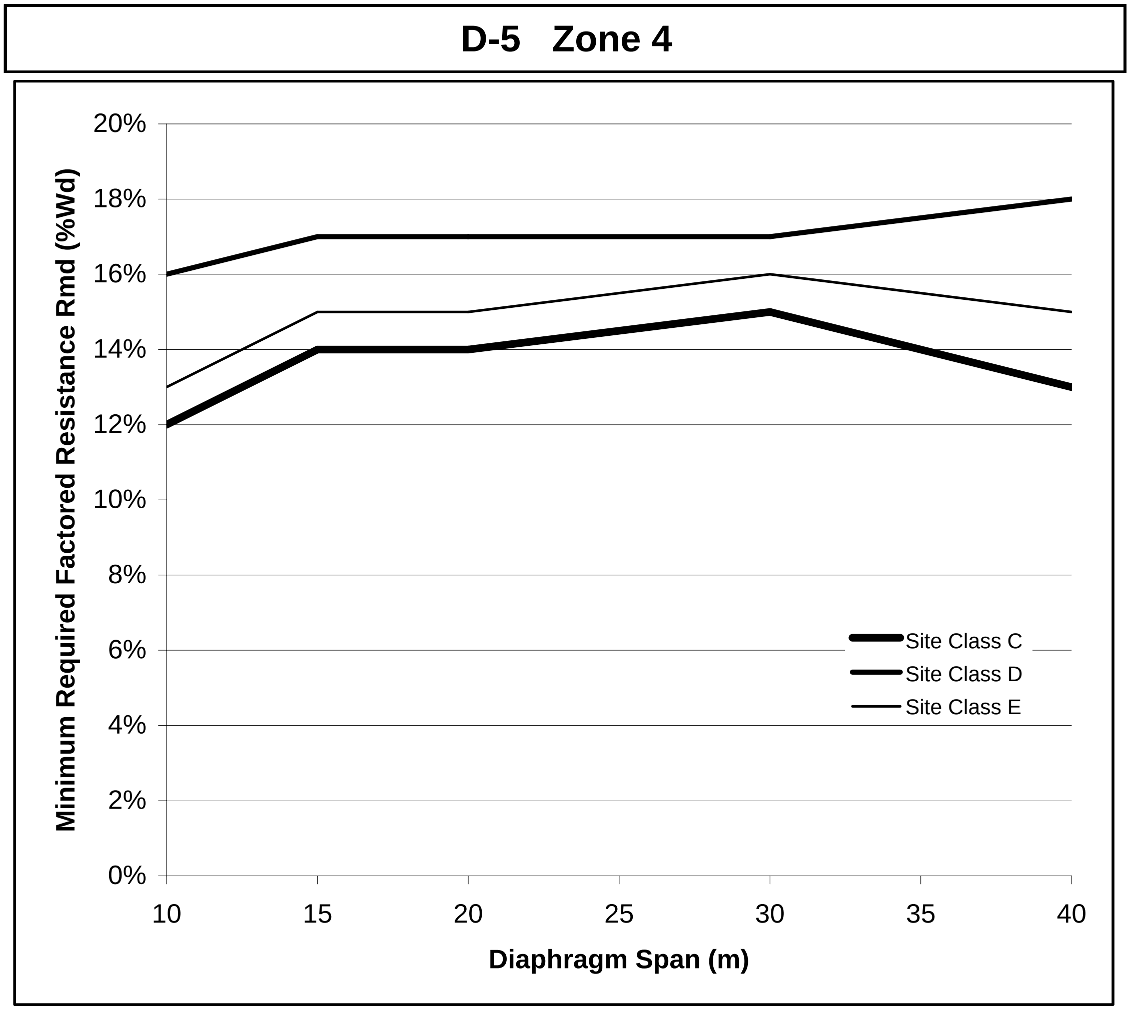
**D-5 Zone 3**




| **Site Class** | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 10% | 12% | 12% | 12% | 11% | NR |
| *D* | 13% | 15% | 15% | 16% | 14% | NR |
| *E* | 13% | 15% | 15% | 16% | 14% | NR |

**Figure 10-5(b)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-5 Steel Bracing (Tension Only)*
*Seismic Zone 3 (Chilliwack)*

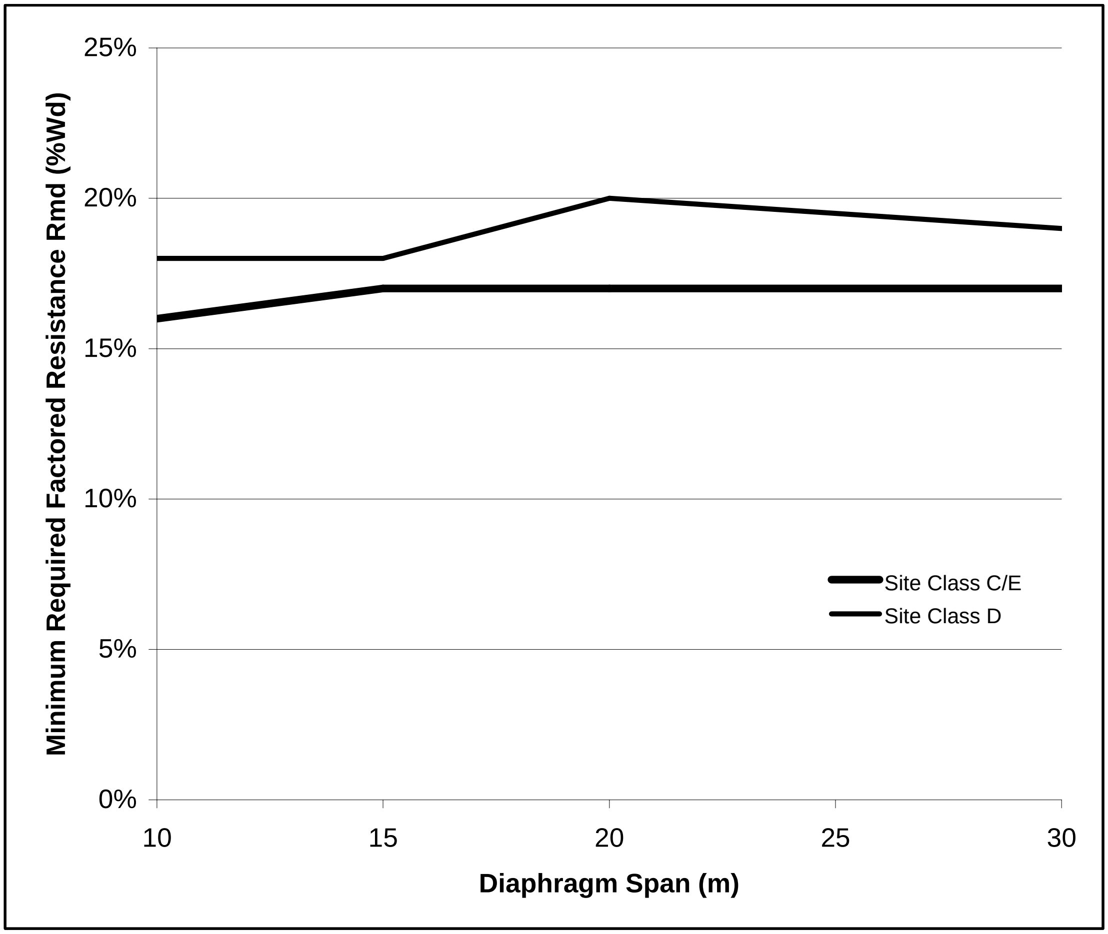



| Site Class | 10m | 15m | 20m | 30m | 40m | 50m |
|---|---|---|---|---|---|---|
| C | 12% | 14% | 14% | 15% | 13% | NR |
| D | 16% | 17% | 17% | 17% | 18% | NR |
| E | 13% | 15% | 15% | 16% | 15% | NR |

**Figure 10-5(c)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-5 Steel Bracing (Tension Only)*
*Seismic Zone 4 (Vancouver)*

**D-5 Zone 5**

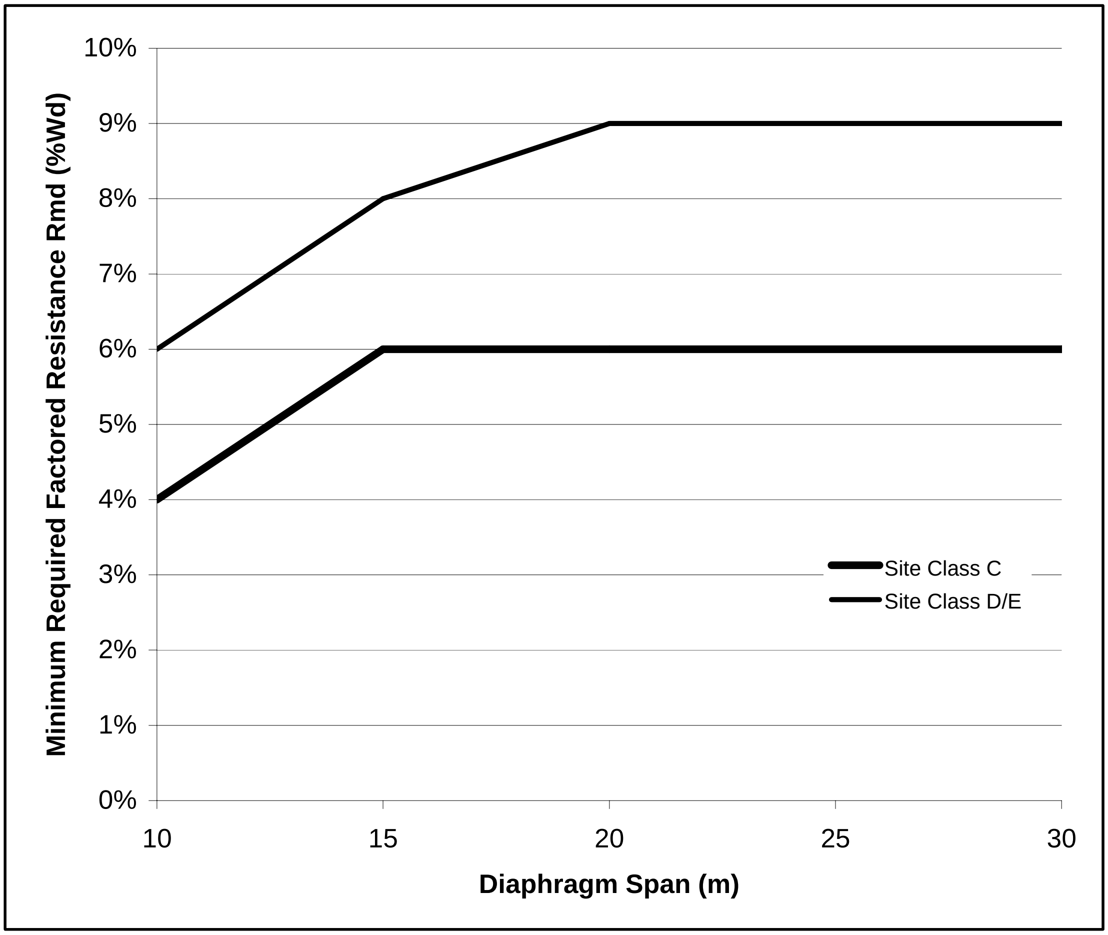



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 16% | 17% | 17% | 17% | NR | NR |
| *D* | 18% | 18% | 20% | 19% | NR | NR |
| *E* | 16% | 17% | 17% | 17% | NR | NR |

**Figure 10-5(d)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-5 Steel Bracing (Tension Only)*
*Seismic Zone 5 (Victoria)*

**D-6 Zone 2**

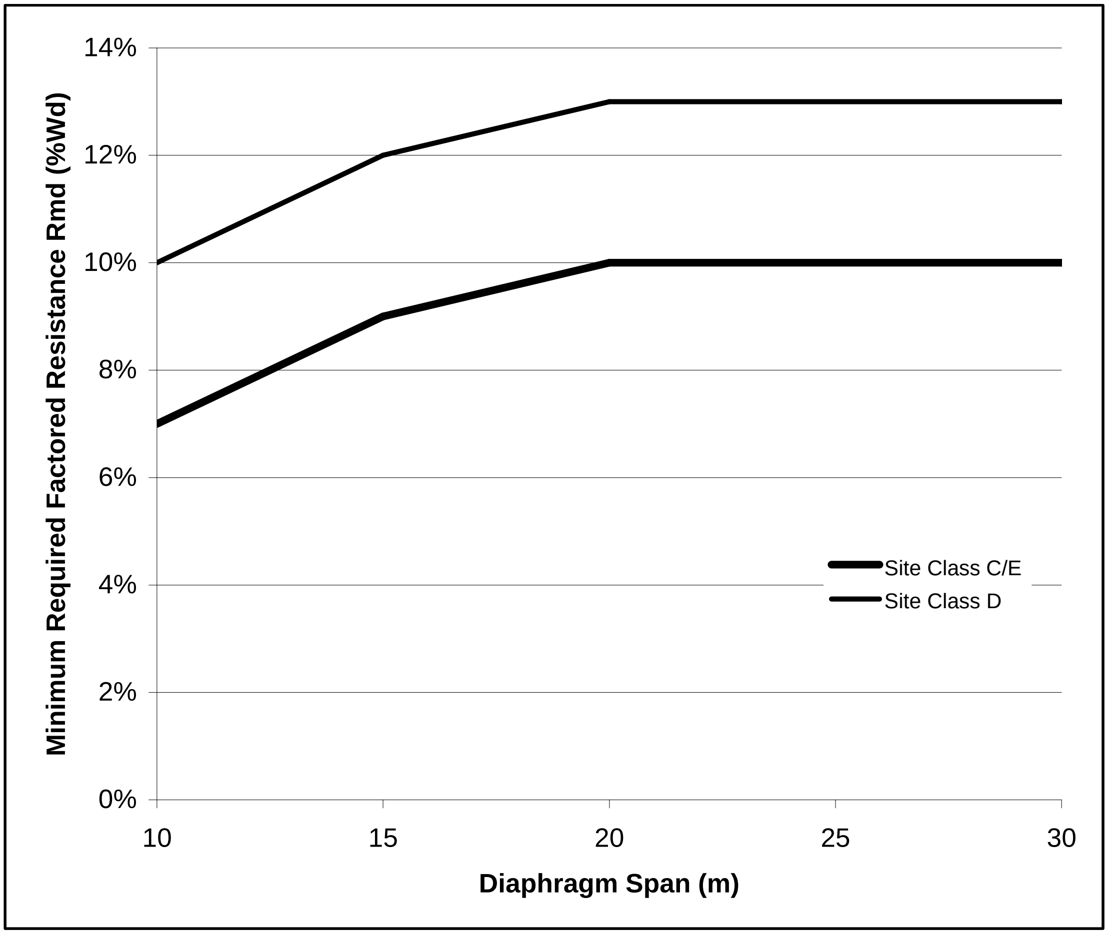



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 4% | 6% | 6% | 6% | 7% | 7% |
| *D* | 6% | 8% | 9% | 9% | 8% | 8% |
| *E* | 6% | 8% | 9% | 9% | 8% | 8% |

**Figure 10-6(a)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-6 Steel Bracing (Tens./Comp.)*
*Seismic Zone 2 (Princeton)*

**D-6 Zone 3**

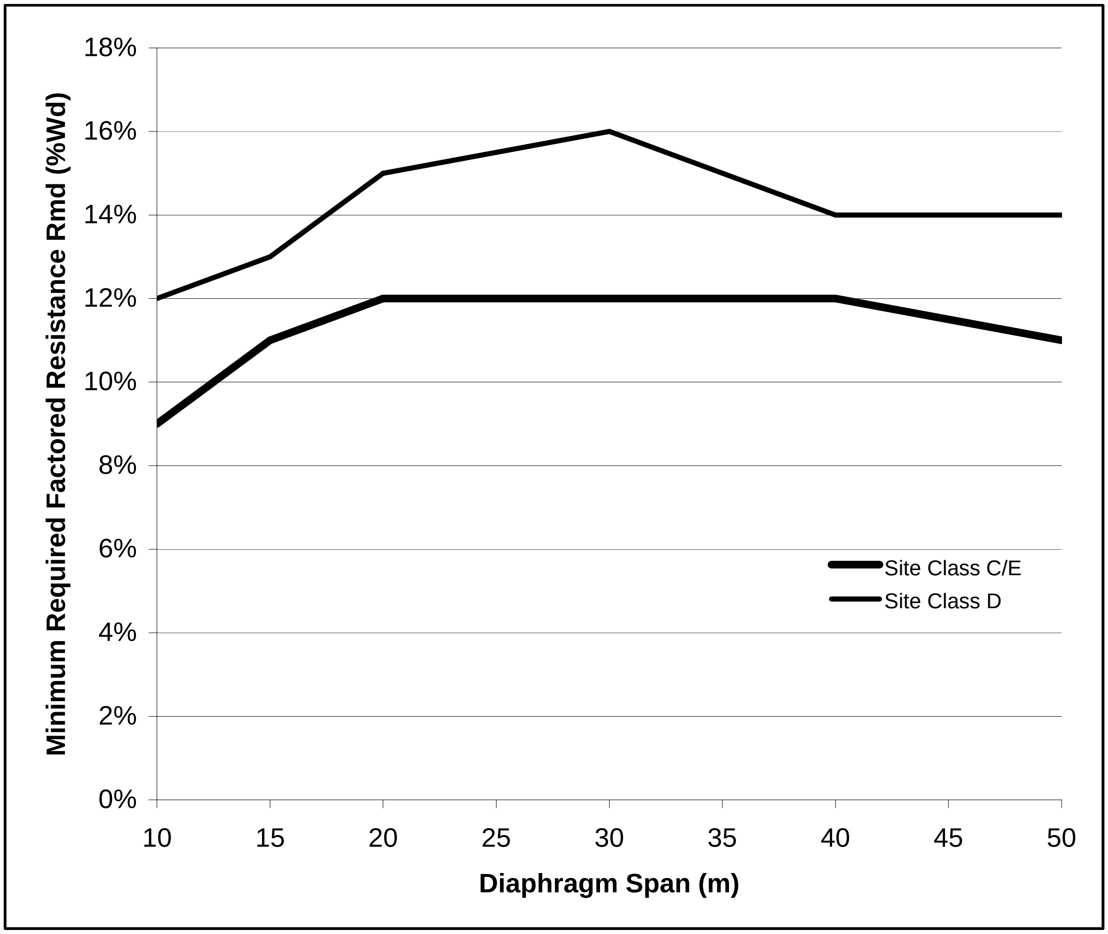



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 7% | 9% | 10% | 10% | 10% | 9% |
| *D* | 10% | 12% | 13% | 13% | 13% | 12% |
| *E* | 7% | 9% | 10% | 10% | 10% | 9% |

**Figure 10-6(b)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ ($\%W_d$)*
*Diaphragm Prototype D-6 Steel Bracing (Tens./Comp.)*
*Seismic Zone 3 (Chilliwack)*

**D-6 Zone 4**

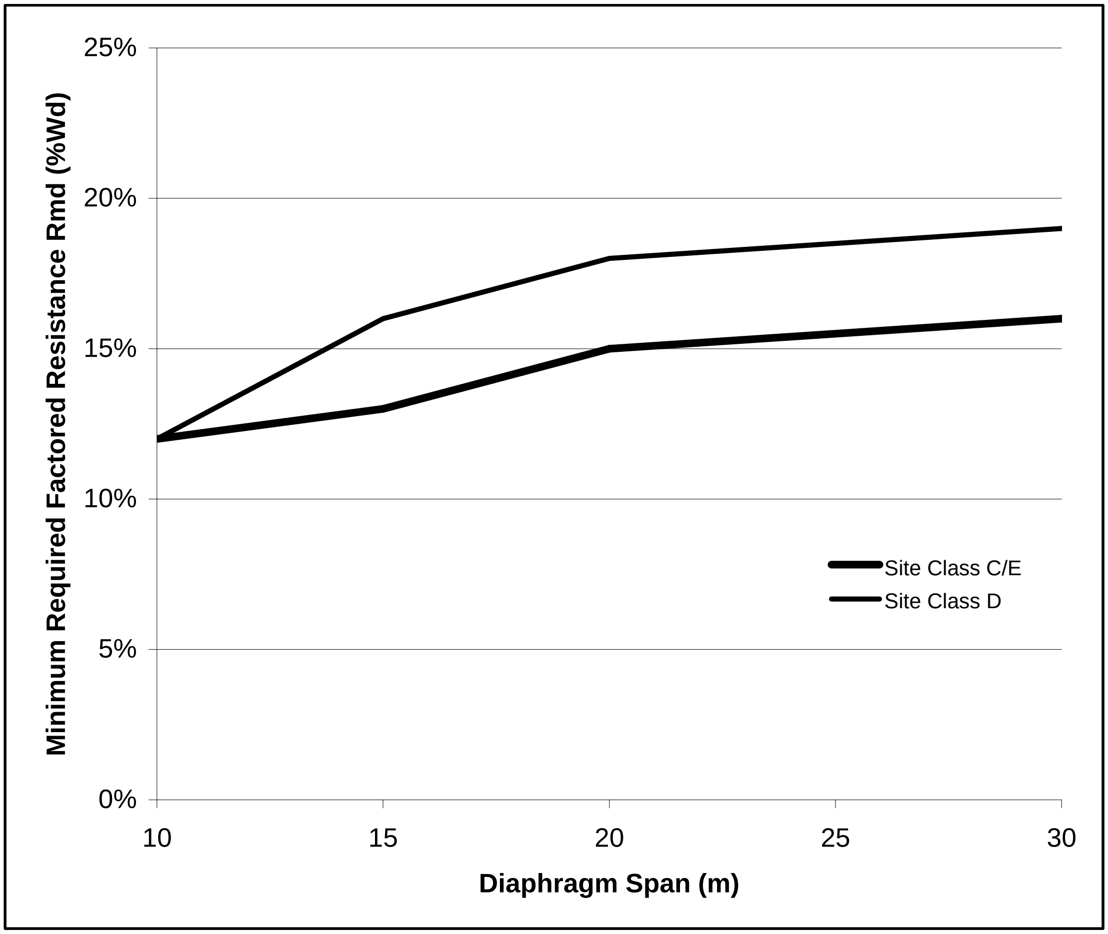



| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 9% | 11% | 12% | 12% | 12% | 11% |
| *D* | 12% | 13% | 15% | 16% | 14% | 14% |
| *E* | 9% | 11% | 12% | 12% | 12% | 11% |

**Figure 10-6(c)**

*Minimum Required Retrofit Factored Resistance $R_{md}$ ($\%W_d$)*
*Diaphragm Prototype D-6 Steel Bracing (Tens./Comp.)*
*Seismic Zone 4 (Vancouver)*

**D-6 Zone 5**




| Site Class | *10m* | *15m* | *20m* | *30m* | *40m* | *50m* |
|---|---|---|---|---|---|---|
| *C* | 12% | 13% | 15% | 16% | 14% | 14% |
| *D* | 12% | 16% | 18% | 19% | 19% | 20% |
| *E* | 12% | 13% | 15% | 16% | 14% | 14% |

**Figure 10-6(d)** *Minimum Required Retrofit Factored Resistance $R_{md}$ (%$W_d$)*
*Diaphragm Prototype D-6 Steel Bracing (Tens./Comp.)*
*Seismic Zone 5 (Victoria)*

## INTRODUCTION

This document contains questions and answers regarding the Bridging Guidelines. It also contains comments gathered during the Office Visits to engineering firms currently using the guidelines.

Each of the sections below corresponds to one of the sections of the guidelines.

## 1.0 GENERAL REQUIRMENTS

*Question:* What if the building has different story height? Say the first story is 4.0m high, the second and the third story is 3.6m high, should the average height be used for calculation?

*Answer:* Always use the first storey height. Variation in the storey height above the first storey is not an issue provided the variation in storey height does not exceed 30%. Uniform storey height is usual for school buildings.

*Question*: If using NBC 2005 I=1 results in retrofit within escalated 2004 budget, should that be used? To provide a higher resistance level for the school? Should all consultants compare costs of NBC 2005 I=1 upgrade vs. cost of guideline upgrade? That is, do parallel design for all upgrades?

*Answer:* Design to the Bridging Guidelines. They key is to save money, so do not design to a higher level. Only use 2005 NBCC I=1.0 if something is not possible with the Bridging Guidelines, or you suspect it would be more expensive.

*Comment*: The general methodology of using inelastic drift estimation (performance-based approach) is a good development in advancing seismic engineering.

*Comment*: In general, the Bridging Guidelines are easy to use.

*Comment*: Collaboration with design consultants is a good approach to improving the Bridging Guidelines (informal meetings with consultants, workshops) and to assisting consultants/school districts/Ministry with difficult or unusual technical issues.

*Comment*: Demonstration projects are a good resource for consultants, especially if all consultants have an opportunity to contribute.

*Comment*: Consultants would be most interested in any developments in adapting the Bridging Guidelines for non-school buildings (e.g. post-disaster buildings).

*Comment*: The feasibility guidelines are not very explicit on "soft" management issues such as minimum qualifications for a prime consultant (some structural engineering firms uncomfortable or unqualified to be prime). School districts would also appreciate some guidance. Phasing (scheduling, swing space) is a huge construction management issue where the consultant and the district would benefit from more explicit guidance.

*Comment*: Consultants could also use guidance regarding options that trade off cost vs. noise and disruption; e.g. reinforcing masonry by sawcutting face, installing rebar, grouting versus FRP application. Is the Ministry willing to accept a cost premium if noise and disruption is reduced that results in a better teaching environment during construction?

*Comment*: Suggest testing and added prototypes for such materials as Sureboard (on steel stud walls) and metal deck over masonry.

*Comment*: We are supportive of a collaborative process that provides all engineers with the opportunity to provide constructive criticism to improve the Bridging Guidelines, especially given the substantial change advocated in seismic engineering practice. We like the ability to be able to combine contributions from different materials in a deformation compatible manner. The second edition is a vastly improved document compared with the first edition. The first edition was more like a "black box". We are looking forward to using the second edition for assessing George Jay Elementary that has a large clay brick masonry building.

*Comment*: A suite of subduction ground motions needs to be included in the Bridging Guidelines to enable engineers to check Vancouver Island buildings for long duration shaking.

*Comment*: We are comfortable with the Bridging Guidelines methodology for their application to the retrofit of school buildings in the province. Those engineers initially reluctant to embrace the Bridging Guidelines are simply reacting to a major change in practice. We understand on-going professional development is a healthy aspect of our profession. The second edition is a more credible document compared with the first edition. Application of the guidelines to actual buildings makes the familiarization process easier.

*Comment*: Equation 1-2 for overturning moment has capacity of the walls (Re) in the upper levels of the building. I do not believe that the capacity of the upper floors have anything to do with the overturning moment of a wall. I suggest that Equation 1-1 be used instead.

*Comment*: Suggest dropping seismic zones and use *Sa* values given in the Code. Perhaps allow iteration between existing tables.

## 2.0 TOOLBOX METHOD

*Question*: How does one apply the Bridging Guidelines to a two-storey block with different LDRSs on each storey?

*Answer*: Use the Force Distribution Equation (1-1) to determine the force on each level, based on their own $R_m$. Retrofit for these values. This process may require iteration.

*Question*: What is the basis for introducing the minimum resistance threshold of 60% of the corresponding code value?

*Answer*: The 60% of code value was determined by consensus around the peer review table as the minimum level of resistance that should be permitted for upgrading school buildings.

*Question*: How are new and old systems combined together with a rigid diaphragm?

*Answer*: Regardless of the performance of a system, all LDRSs can be combined provided they have a common governing drift limit (GDL). Only a few combinations of systems are not possible (e.g. unreinforced clay brick masonry and steel moment frames).

*Comment*: Suggest adding prototypes for tall single storey buildings such as gymnasiums. This could make the retrofit of the e structures more efficient.

## 3.0 WOOD-FRAME BUILDINGS

*Question*: Where can I get values for older existing materials not listed in Table 3.1 or O86-05?

*Answer*: See the Errata Document. It contains an updated Table 3.1.

## 4.0 STEEL BUILDINGS

*Question*: Many existing braces do not meet capacity design requirements (i.e. connections do not meet AgFy). How can they be assessed by the Guidelines and how do I include them in a retrofit plan?

*Answer*: The Bridging Guidelines resistance tables are based in the inelastic response of LDRSs, which are adequately connected. "Bail-out" forces on connections are not included, as the calculations behind the resistance tables are based on the LDRSs yielding. It is not recommended to utilize braced systems that do not meet capacity design requirements. If this is not possible we suggest use 2005 NBCC with an I=1.0.

*Comment*: The Bridging Guidelines seem to penalize steel buildings, especially older steel buildings. Connections in older steel buildings are problematic. We understand the rationale for "AgFy". The options for upgrading older steel buildings are challenging.

*Comment*: The guidelines would benefit from an expanded number of building prototypes, especially for steel buildings.

## 5.0 CONCRETE BUILDINGS

*Question*: How can one wall prototype represent all concrete walls, which have a large range of stiffnesses?

*Answer*: The elastic stiffness of a concrete wall has a major influence on its ability to limit inelastic deformations. See Commentary C6.2.2. Prototypes C1 and C2 are meant to represent walls of a certain stiffness or lower. This is conservative. More flexible walls should use the moment-frame prototypes. We hope to add additional concrete wall prototypes to the next edition to capture the improved performance of stiffer concrete walls.

*Question*: How do we determine the corresponding base shear for the flexural resistance?

*Answer*: Suggest back calculating the force distribution in Equation (1-2). Equation (1-1) gives conservatively high moments, intended to boost the resistance for foundations and holdowns. If used for the flexural "base shear" it would underestimate it.

*Question*: How is $A_{st}$ calculated for the equation in Section A5.5?

*Answer*: $A_{st}$ is the same as $A_v$.

*Comment*: The procedure for dealing with concrete walls is confusing. We need an example to show how the governing mode of failure is determined (i.e. rocking, shear or flexure). It is confusing that both shear and flexural behaviour share the same resistance table. It is odd that a wall 30' long and a wall 10' long have the same strength to stiffness ratio. More tables should be provided to account for different strength to stiffness ratios for concrete walls.

*Comment*: Conventional construction concrete moment frames have an ISDL of 4%. This seems high and the resistance tables also do not seem to change much past 2% drift. I suggest dropping the values for 3% and 4%.

## 6.0 CONCRETE MASONRY BUILDINGS

Change in Out-of-Plane Requirements

*Question*: What was the reason why the out-of-plane requirements were changed to now allow 6" URM walls to be left alone?

*Answer*:

(a)The 1st edition did not intentionally exclude 6" walls. The intent was to exclude 4" walls like clay tile.

(b) 6' walls are thick enough to develop significant vertical restraint forces as the wall starts to rock out-of-plane about a mid-height hinge. For this reason, 6" walls are now included.

Column Gap

*Question:* For infill walls, the 1st edition guidelines specified the size of the gap that had to be cut beside the columns, but now there isn't specific guidance. What should he do?

*Answer*:

Some PRC members did not like imposing a retrofit solution. Sentence 6.6(2) requires the engineer to address the top corner block issue (high hazard locations). If these top corner hazards are mitigated, a 6" unreinforced confined in-fill wall is OK.

Vertical Rebar

*Question:* If they are going to reinforce the walls (to have them act as an LDRS) and only use vertical reinforcement, how do they calculate the strength? The guidelines say to go to S304 - but this requires horizontal reinforcement as well. What do they do for just vertical?

*Answer*:

We do indeed refer engineers to S304 for calculating the lateral resistance of reinforced masonry. The requirement for at least 1/3 horizontal rebar is problematic for most consultants. There is no easy resolution of this issue. Most consultants disregard this requirement for classroom walls (3.5 m or less). We did not have a clear PRC consensus. If only vertical rebar is to be installed, the vertical rebar should meet the minimum combined (vertical + horizontal) requirements given in the code.

Stack Bond

*Question*: Can prototype M-1 be used for unreinforced Stack Bond, and M-2 for reinforce Stack Bond?

*Answer*: No, refer to S304.1 for Stack Bond.

*Comment*: Some guidelines are needed for stack bond concrete masonry (some provisions were in first edition). Out-of-plane behaviour of stack bond is an important issue. A demonstration project with stack bond would be very helpful.

*Comment*: Fibre-reinforced polymer (FRP) seems to be used by other consultants. We need some guidelines on how and when FRP can be used.

*Comment*: Could use clarification on where sliding is assumed to occur (sketch?).

*Comment*: 1$^{st}$ Edition require that 75mm gaps be left on both sides, 2$^{nd}$ Edition only requires top corner blocks to be removed. Uncomfortable with 2$^{nd}$ Edition solution as compression struts can still form. Suggests this be verified experimentally.

*Comment*: Suggest providing a method to account for strength of URM infill and masonry walls with vertical reinforcement only. Currently the Guidelines send the engineer to the Masonry Code for the capacity of reinforced masonry. It does not allow for vertical bars only.

## 7.0 CLAY BRICK MASONRY BUILDINGS

*Comment*: There could be a cost savings in allowing for exterior clay tile to be protected on the interior and to have a protected fall zone on the exterior.

## 8.0 ROCKING

*Question*: How do you determine if rocking governs, and how do you account for the response of the soil?

*Answer*: For a rocking check of an LDRS wall governed by shear, calculate the maximum factored shear of the LDRS and compare this with the maximum factored shear that will result in overturning of the footing. Use the lesser of the two as the governing response. The underlying soil is assumed to be rigid. The resistance factor of 0.8 for rocking accounts for some soil deformation at the toe (loss of lever arm for rocking).

*Question*: When is $R_m$ used during the rocking check?

*Answer*: $R_m$ is not used as part of the rocking check. $R_m$ is strength to which an LDRS must be designed, after it has been determined which failure mode governs.

*Question*: Can we not just use basic engineering principles to allow for rocking? We don't find the rocking section easy to use.

*Answer*: We will expand the rocking section in Commentary Part A to help clarify the rocking issues. It is important to use the Bridging Guidelines approach to rocking. Rocking resistance is strongly influenced by the type of material, the aspect ratio and the Governing drift Limit. It is not simply the maximum lateral force resisting overturning.

*Comment*: Section 8 (rocking LDRSs) needs more clarification.

*Comment*: Some sort of “bail-out” force is required for foundation design where both the overturning capacity and shear capacity of a shearwall are much higher than the demands on the resistance tables.

## 9.0 HEAVY PARTITION WALLS

*Comment*: Suggest more guidance for 100mm URM walls, especially in areas very difficult and costly to upgrade, such as washrooms. Should low occupancy areas such as washrooms, storage areas, be deleted from upgrading? If not, can cost effective alternatives be provided.

## 10.0 DIAPHRAGMS

*Question*: Is the diaphragm demand, $R_{md}W_d$, divided by two to calculated the force on each side of the diaphragm?

Answer: No, $R_{md}W_d$ is the required minimum strength of the diaphragm, it is not a “force”.

<u>Diaphragms for Multi-Storey Buildings</u>

*Question:* New tables for diaphragms: Can they be used for multiple storey buildings? Should the seismic weight as an input for these tables represent re-distributed $F_x$ lateral loads?

*Answer*:

(a) Diaphragm tables are intended for all low-rise buildings (1-3 storeys). The diaphragm tables apply to both roof and floor diaphragms. The metal deck diaphragms are roof diaphragms only.

(b) Diaphragm weight $W_d$ should be calculated in accordance with its definition with no redistribution of inertia mass as given in Section 1.10 of the guidelines. Section 1.10 is for LDRSs, not diaphragms. There is no mechanism for dynamic transfer of diaphragm self weight from one diaphragm to another diaphragm above or below it. The wall contribution to $W_d$ is for out-of-plane wall behaviour. Walls acting out-of-plane have no reliable means of dynamically redistributing their self weight over storey height.

Diaphragm Shear Force Distribution

*Question*: Two different diaphragm shear diagrams were presented: a triangular shape for metal deck and rectangular for wood diaphragms. In my opinion, all these diaphragms represent a beam subjected to dynamic loads, therefore the resulting shear diagram is more of sinusoidal shape. I suggest to conservatively use a rectangular shape in all the above-mentioned cases. Especially that these two materials are similarly flexible diaphragms, so I do not think the response should be so much different.

*Answer*:

(a) The shear force distribution in the diaphragm comes directly from the non-linear dynamic analysis of the diaphragm over its length between supporting end walls.

(b) The metal deck diaphragms essentially behave elastically with the diaphragm lateral strain varying linearly (approximately) from a maximum at the supporting end walls to zero at mid-span.

(c) The diaphragm lateral strain in wood diaphragms is inelastic for almost the entire length of the diaphragm. This results in a uniform shear force from end wall to mid-span. This inelastic response distinguishes the behaviour of wood diaphragms from that of metal deck diaphragms.

*Question*: Clause 10.9 gives the inelastic strain limitation for diaphragms. But Table A.10-1 gives larger limitation values. Can we use the data on the table for design?

*Answer*: The title of Section 10.9 will be reworded slightly differently to avoid confusion. We will probably rename this section "Global Inelastic Strain Limitation". The strain limits in Section 10.9 have no direct relationship to those given in Table A.10-1. Better clarification of the strain limitations in Section 10.9 will be forthcoming.

*Question*: Where do the diaphragm chord force equations (10-1) and (10-2) come from?

*Answer*: Equations (10-1) and (10-2) are for the factored axial force in the chord along each edge of the diaphragm perpendicular to the direction of shaking. The equations are derived directly from the assumed shear force diaphragm illustrated in Figures A.10-3.

*Question*: How do we determine the forces in a rigid diaphragm?

*Answer*: Suggest using the force distribution in Equation (1-2), but use the overstrength of the walls (i.e. $R_e R_o$). Concrete School Demonstration Project will provide an example.

*Question*: Sentence 10.8(2) – Shiplap roofs – Does this limitation only apply to roofs or also suspended floors?

*Answer*: Only roofs have this limitation. Typically suspended floors have some sort of flooring material over the shiplap.

*Comment*: We would like number of diaphragm types expanded. We would like some guidance on a cost-effective way to upgrade an unblocked wood diaphragm to a higher strength blocked diaphragm. We need some resistance data for an upgraded Type B metal deck diaphragm (falls short of Type A but better than Type B). This may require some further research (Tremblay).

*Comment*: Clarify preferences for upgrading of metal decks:

- combine capacity of new pins and existing welds? Or only capacity of new pins?
- Laps: combine capacity of button punch and new screws? Or only capacity of new screws?
- Retrofit from underside OK?
- Retrofit laps from underside, then defer upgrade of connections from above until roofing scheduled for replacement
- Delete reference to welded washers?
- Offer guidance on Hilti tabulated values vs Hilti software; and what factor to use on allowable (vs ultimate) tabulated values

*Comment*: We noted the substantial difference in ductility between Type A and Type B steel deck diaphragms. We look forward to the review comments from the External Peer Reviewers (EPR) on diaphragms and steel deck diaphragms in particular (EPR of diaphragms in late 2007).

*Comment*: Rigid Diaphragms should be included. Some older concrete diaphragms have high stiffness but a low strength. A section on rigid diaphragms is needed to determine what force levels must be resisted by the rigid diaphragms, and what forces they need for connections.

*Comment*: Section 10 suggests that wood diaphragms that meet certain requirements can have their retrofit delayed until the school is scheduled for re-roofing. Is it possible to extend this to all types of diaphragms?

## 11.0 CONNECTIONS

*Question*: Is there a bail-out force for rocking foundations?

*Answer*: No, there are no bail-outs in the Bridging Guidelines. Design the foundation to $R_eR_o$ of your system. *Re* is based on the governing LDRS, which could be rocking.

*Comment*: Clarify required sliding resistance of foundations.

*Comment*: Some sort of bail-out force is needed for connections on very strong LDRSs. While it may not follow a capacity design philosophy, the code allows it. In some cases it might be more cost effective to address these types of systems with the code and not use the Bridging Guidelines. If the purpose of the Bridging Guidelines is to be more efficient than the code, it needs to incorporate some bail-outs fore connection forces.

**END OF CRITQUE DOCUMENT**