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SEISMIC REHABILITATION OF UNREINFORCED MASONRY BEARING WALL BUILDINGS: MODERNIZING CANADIAN PRACTICE

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ABSTRACT: Unreinforced masonry (URM) buildings have consistently been one of the most prominent contributors to loss of life and property damage in earthquakes. As such, several methodologies for seismic evaluation and rehabilitation have been created. The most current Canadian design reference on seismic evaluation of existing buildings (including URM bearing wall buildings) was published in 1992 and was developed to be compatible with the 1990 National Building Code of Canada: the Canadian "Guidelines for Seismic Evaluation of Existing Buildings" (CGSEEB) represented state-of-the-art design practices at the time of its release, but no updates have since been published. In contrast, the analogous American document (FEMA, 1992) has seen successors published in 1998 (FEMA 310), 2003 (ASCE 31), and 2013 (ASCE 41-13). This paper reviews the development of the URM provisions of the CGSEEB, compares the provisions to analogous American design references, and makes recommendations on modernizing the URM provisions of the CGSEEB.

1. Introduction

The most recent Canadian documents for seismic evaluation and strengthening of existing buildings were published in 1992 and 1995 (NRC, 1992; NRC, 1995). Specifically with regard to URM, the Canadian Guidelines for Seismic Evaluation of Existing Buildings (referred to herein as the "CGSEEB") includes Appendix A, which is specified for both evaluation and strengthening. Appendix A contains a Canadian version of what is commonly known as the "special procedure" for URM buildings with flexible diaphragms, which was developed based on research from the 1980's (ABK, 1984). The special procedure has been the most commonly applied methodology for seismic evaluation and strengthening of flexible diaphragm URM buildings in both Canada and the United States since its widespread adoption in codes and guidelines in the late 1980's (Paxton, Turner, Elwood, & Ingham, 2015). Appendix A of the CGSEEB represented state-of-the-art design practices at the time of its release (Bruneau, 1994), but no updates to the CGSEEB have since been published. In contrast, the analogous American document (FEMA, 1992) has been succeeded by several direct updates and closely related documents (ICBO, 1997; FEMA, 1998; ICBO, 2001; ICC, 2003; ASCE, 2003; ASCE, 2013).The lack of updates to the Canadian documents is significant for two basic reasons:

1) The body of knowledge of earthquake engineering has grown considerably with the many significant earthquakes that have occurred since the Canadian publications were issued
2) The Canadian documents were developed to be compatible with the 1990 National Building Code of Canada (NBCC). The NBCC has been updated several times since the publications were issued, including fundamental changes to assessment of seismic hazard and design forces in 2005 (DeVall, 2003), and thus the CGSEEB and the NBCC are now incompatible

Although all of the Canadian publications on seismic evaluation and strengthening need updating, the focus herein is restricted to Appendix A of the CGSEEB because this is the most pertinent item with regard to Canadian design practice for seismic rehabilitation of unreinforced masonry bearing wall buildings.

This paper reviews the development of the URM provisions of the CGSEEB, compares the provisions to analogous American design references, and makes recommendations on modernizing the URM provisions of the CGSEEB. As a preliminary matter, designers should note that the CGSEEB contains a clause (within Section 1.3.1) that states that the guidelines ***"...do not prevent an engineer from making a properly substantiated evaluation using other procedures."*** Thus, designers need not feel constrained by the provisions of Appendix A of the CGSEEB.

2. Review of Available Design Techniques

"Design techniques" refers not only to national codes and standards but also to guideline documents and other methodologies used in practice. Although it is recognized innumerable techniques exist worldwide, the focus here is restricted to those that have been or are currently commonly employed in Canada, the USA, and New Zealand, because the URM bearing wall buildings found in these regions are relatively similar although there are some key differences (Paxton, 2014).

2.1. ABK Special Procedure for URM

The "special procedure" for URM was developed in the 1980's through a joint effort of three prominent structural engineering firms in California (ABK, 1984). The joint effort was funded by the National Science Foundation and involved categorizing URM buildings, defining seismic demands, and static and dynamic testing of URM building components (eg. walls, diaphragms). The ultimate product was a methodology for seismic evaluation/retrofitting of URM buildings. The methodology was subsequently adopted, often with some modifications, into several city building codes and model seismic standards in the United States, including the following examples:

- 1981: Los Angeles Building Code, Division 88 (City of Los Angeles, 1987)
- 1988, 1991, 1997: Uniform Code for Building Conservation (UCBC) (ICBO, 1997)
- 1992: San Francisco Building Code, Chapter 16C (City of San Francisco, 2013)
- 1992: FEMA 178 (1992), NEHRP Handbook for The Seismic Evaluation of Existing Buildings
- 1998: FEMA 310, Prestandard for Seismic Evaluation of Existing Buildings (FEMA, 1998)
- 2001: Guidelines for Seismic Retrofit of Existing Buildings (ICBO, 2001)
- 2003: ASCE 31, Standard for Seismic Evaluation of Existing Buildings (ASCE, 2003)
- 2003, 2006, 2009, 2012: International Existing Building Code (IEBC) (ICC, 2012a)
- 2013: ASCE 41 (2013), Standard for Seismic Evaluation and Retrofit of Existing Buildings

The assumed dynamic behaviour underlying the methodology is significantly different from those underlying typical modern building codes. The special procedure recognizes that the walls of URM buildings are relatively rigid and the flexible (typically wood) diaphragms can dominate the dynamic response of the structure. Accordingly, the special procedure assumes that the end walls (see Figure 1) transmit the ground motions to the diaphragms without any amplification; the diaphragms in turn interact with the head walls (also known as out-of-plane walls), which are dynamically excited. There are often extensive interior walls (typically wood framed walls finished with lath and plaster) in such buildings and some versions of the special procedure provide relaxations for buildings with such "crosswalls" (see Figure 1) in recognition of the fact that these elements may act to provide hysteric damping.

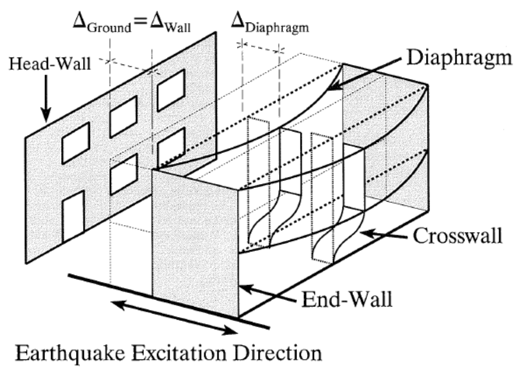

Figure 1 – URM Building Seismic Behaviour (from Bruneau, 1994)

Some versions of the special procedure have included rudimentary provisions to account for varying diaphragm stiffness, while others have not. The result is typically a lower design base shear than would be derived from otherwise-equivalent codes for new construction (Bruneau, 1994). Additionally, the acceptance criteria for resistances are often more liberal than those contained in typical building codes for new construction: for example URM walls are allowed to rock out-of-plane and URM piers are permitted to rock in-plane, provided they remain dynamically stable.

It is worth noting that the validity of the “special procedure” model has been questioned, as considerable amplification due to the side walls has been observed in at least one instrumented, retrofitted building (Tena-Colunga & Abrams, 1992) and a host of other items were identified during the development of the methodology (SEAOC, 1989). However, a detailed discussion on these issues is beyond the scope of the current paper.

2.2. ASCE 41-13

The American Society of Civil Engineers publishes a standard for the seismic evaluation and rehabilitation of existing buildings. The current standard, ASCE 41-13, addresses both evaluation and rehabilitation. Previously, separate standards were issued for evaluation (ASCE 31-03) and rehabilitation (ASCE 41-06). ASCE 41-06 was a developed based on minor modifications from FEMA 356 (2000) and FEMA 273 (1997).

Note that ASCE 41 contains provisions for all types of buildings/materials (not just a specific type of URM building). A key element of ASCE 41 is that it employs performance-based design principles. Under this approach, several performance objectives are possible, rather than just the “life-safety” objective of new building codes, for example. This is valuable for two main reasons:

1) it allows flexibility in setting performance criteria, and
2) requires an explicit conversation with the building owner (and possibly other stakeholders) about the expected seismic performance of the building.

The 2006 and 2013 editions contain provisions for masonry (and other materials), using linear and nonlinear procedures, as may be applicable to URM buildings in general (referred to henceforth as the “performance-based” provisions). The 2013 edition also contains a version of the special procedure (chapter 15) as an alternative method of rehabilitation, for limited performance objectives. Below we focus on the “performance-based” provisions of ASCE 41-13. It also contains methodologies for a checklist-type deficiency-only evaluation. Further details can be found in the standard.

Similar to the special procedure, the performance-based provisions (Chapter 7 - Analysis) contain a force distribution that ignores any dynamic amplification in the end walls, provided the diaphragms are flexible and the building is six storeys or less in height (ASCE, 2013). However, the provision is more rational in that Chapter 7 requires that the period of the diaphragm be calculated, which affects the forces transferred to end walls, as well as the deflection of the diaphragm. To comply with the aforementioned force distribution, diaphragm deflections are required to be less than 6 inches, yet the diaphragm must be classified as “flexible”: as per ASCE 41-13, a diaphragm is considered flexible if the diaphragm deflection is more than twice the average interstorey drift of the vertical LFRS of the storeys immediately above and below the diaphragm. With regard to design checks for in-plane walls, the performance-based provisions (Chapter 11 - Masonry) are more rigorous than the special procedure: the special procedure considers only two possible failure modes (pier rocking and diagonal tension), while the performance-based provisions consider two additional possible failure modes (toe crushing and bed joint sliding). Out-of-plane wall provisions in the performance-based procedure are similar to the special procedure: the same h/t values are specified, except that Chapter 11 does not allow for any relaxations based on the presence of crosswalls.

2.3. New Zealand Guidelines

The most relevant seismic guideline in New Zealand is the NZSEE (New Zealand Society for Earthquake Engineering) publication, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* (NZSEE, 2006). Consistent with national legislation for seismic risk mitigation, the assessment framework is intended to provide results in terms of %NBS (New Building Standard). In April 2015, the URM provisions were substantially revised with the publication of corrigendum No. 4. The provisions include guidance on determining design actions using the national building code, NZS 1170.5 (NZS, 2004). Guidance is also provided on strength and stiffness values for existing materials, such as URM walls and timber diaphragms. Although the focus of the document is on determining the %NBS, which is specific to New Zealand buildings, the material properties and component acceptance criteria (i.e. resistances) could possibly be implemented in Canadian practice due to the relatively similar URM construction practices in New Zealand and Canada (Paxton, 2014).

3. Opinion on “%Code” Achieved by CGSEEB Appendix A

Building authorities and owners often like to express seismic evaluation/retrofit requirements as a fraction current code forces (eg. “retrofit to 70% code”). Unfortunately, translating the design provisions of CGSEEB Appendix A into a “%code” is not a straightforward task. Determining the design forces for the building under the special procedure is quite simple, but the designer is then faced with the question: *are the “new building code” design forces to which the results of Appendix A should be compared?* To provoke debate and illustrate the challenge of addressing this question, the following are examples of possible conclusions, many of which are obviously flawed or incorrect:

1) CGSEEB Appendix A is the most “current” Canadian provision for URM, thus compliance with this document constitutes 100% code.
2) Because the building is (likely) only a few stories, the base shear from Appendix A should be compared to the short period cut-off values from the current NBCC.
3) Because the flexible (long period) diaphragm drives the period of the building, the NRC-specified forces should be compared to the corresponding forces from the current NBCC (note: NBCC 2010 does not provide guidance on establishing the period of flexible diaphragm buildings).
4) Because the assumed dynamic behaviour under CGSEEB Appendix A and the NBCC are fundamentally different, it is inappropriate to compare design forces from the two documents; therefore one can only make a comparison between the seismic hazard values.

Note that even with equivalent base shears, the distributions of forces over the height of the building would not be consistent due to the aforementioned “rigid wall” assumption underlying the special procedure. Finally, the issue is further complicated by the fact that the resistances specified in the special procedure (eg. in plane rocking resistance of URM piers) have no NBCC equivalent and other acceptance criteria is based solely on empiricism, to an unknown degree of structural reliability. Even if the “correct” 100% code forces could be determined and the building were retrofitted to these requirements, such a building could not be considered equivalent to a new building because new designs depend upon prescriptive requirements in material standards to ensure the structure has sufficient resilience for earthquakes beyond the intensity considered in design. In fact, the NBCC does not allow URM building construction in most seismic zones, making true equivalency impossible.

In light of the aforementioned issues, the authors feel that one can – at best – provide an opinion on the “%code” achieved by CGSEEB Appendix A under a given set of assumptions and restrictions. In the authors' opinion, the most appropriate and reasonable conclusion is #4 from above. Thus the ground motion parameters of CGSEEB Appendix A and NBCC 2010 were compared as described below.

CGSEEB Appendix A specifies the following "effective zonal velocity" (v'), which is a modified version of the NBCC 1990 zonal velocity ratio (v), for use throughout the document. For those unfamiliar with NBCC 1990 seismic hazard parameters, it is similar to the velocity-related acceleration coefficient (A_v) as defined in FEMA 178 (1992b).

$$v' = \frac{v*I*F}{1.3} \quad [1]$$

Where:
$v \equiv$ Zonal Velocity Ratio (NBCC 1990)
$I \equiv$ Importance Factor (NBCC 1990)
$F \equiv$ Foundation Factor (NBCC 1990)

With most international codes now focusing on spectral acceleration ordinates to define the seismic hazard, Chapter A1 of the IEBC (ICC, 2012b) has replaced the previously used 'A_a' and 'A_v' parameters with spectral acceleration parameters. A value of $0.8*S_{D1}$ [as defined in (ICC, 2012a)] is used therein to approximate the previously used effective peak acceleration (EPA) parameters, such as 'A_v' (FEMA, 1992) and 'Z' (ICBO, 1997). Making the same approximation, the effective zonal velocity based on NBCC 2010 parameters (v*) is:

$$v^* = 0.8 * S_a(1) * I_E * F_v \quad [2]$$

Where:
$S_a(1) \equiv$ Spectral Acceleration at T=1sec (NBCC 2015)
$I_E \equiv$ Importance Factor for Earthquake Loads (NBCC 2010)
$F_v \equiv$ Foundation Factor (NBCC 2010)

Note that NBCC 2015 is listed above for the parameter $S_a(1)$. The values for $S_a(1)$ from NBCC 2015 were used as these are based on the most recent hazard knowledge in Canada. For the BC west coast, the hazard contribution from a Cascadia Subduction Zone mega-thrust earthquake is now included probabilistically in the NBCC 2015 hazard values, which represents a significant increase for longer period parameters such as $S_a(1)$. In deriving the %code values, buildings on site classes A through E were considered. Note that the foundation factors between Equations 1 and 2 are not consistent. Heidebrecht (2003) compares the foundation factors in question and equates them as shown in Table 1.

Table 1 – Comparison of Foundation Factors

Soil Type (NBCC 1990)	'F' (NBCC 1990)	Site Class (NBCC 2010)	'F_v' (NBCC 2010)[1]
1	1.0	A	0.6
		B	0.8
		C	1.0
2	1.3	Between C and D	--
3	1.5	D	1.1
4	2.0	E	1.7

[1]F_v is a function of intensity and the values shown here correspond to $S_a(1)$=0.6g, which was used in calculating the %code for Victoria. F_v values corresponding to $S_a(1)$=0.14g were used in calculating %code values for Montreal.

With the foundation factors equated, the "%code" values implied by a design based on CGSEEB Appendix A can now be derived. Results are provided for Victoria (Table 2) and for Montreal (Table 3). Note that the F_v values were different for Montreal because $S_a(1)$=0.14g for this location, versus $S_a(1)$=0.67g for Victoria.

Table 2 – %Code Values for Victoria

Site Class	F	v'	F_v	v*	%code
A	1.0	0.23	0.6	0.32	72%
B	1.0	0.23	0.8	0.43	53%
C	1.0	0.23	1.0	0.54	43%
D	1.5	0.35	1.1	0.59	59%
E	2.0	0.46	1.7	0.91	51%
Results based on: v=0.3 (NRC, 1990), $S_a(1)$ = 0.67g(GSC, 2014)					

Table 3 – %Code Values for Montreal

Site Class	F	v'	F_v	v*	%code
A	1.0	0.077	0.50	0.056	138%
B	1.0	0.077	0.65	0.073	105%
C	1.0	0.077	1.0	0.112	69%
D	1.5	0.115	1.35	0.15	76%
E	2.0	0.154	2.05	0.23	67%
Results based on: v = 0.1(NRC, 1990), $S_a(1)$ = 0.14g(NRC, 2010)					

The results differ greatly depending upon soil conditions and location. In passing it is noted that, although many of the values fall below 70% (a common benchmark in seismic evaluation/rehabilitation), the Canadian seismic hazard values for long periods are higher than American values for similar locations. For a building in Victoria on Site Class C, for example, the IBC (ICC, 2012a) specifies S_{D1}=0.43g (as retrieved from: http://earthquake.usgs.gov/designmaps/us/application.php), which is two-thirds of the 2475-year value of 0.64g. If a 2/3 reduction factor was applied to the $S_a(1)$ values, the lowest "%code" result would be 64% (for Site Class 'C' in Victoria). Some may argue that this is irrelevant as the Canadian seismic hazard values "are what they are." However, retrofitting URM buildings to 50% greater seismic demands than would be required for the same buildings in the United States (for example the same building in Victoria, BC and Seattle, WA) would have significant cost implications – in the authors' opinion, this is an important consideration that deserves further attention.

4. Review of CGSEEB Appendix A and Recommended Actions

Although the validity of the special procedure has been questioned and there are other candidate design techniques available, CGSEEB Appendix A should be updated to be consistent with the current NBCC and other newer versions of the special procedure published elsewhere as a minimum action. This section briefly reviews and discusses various provisions of Appendix A and presents proposed changes intended to achieve the aforementioned minimum action. For brevity's sake, it is assumed that the reader is generally familiar with Appendix A and the various aforementioned American versions of the special procedure and no discussion is provided on aspects for which no significant actions are recommended.

4.1. Purpose/Performance Objective

4.1.1. Background

Most versions of the special procedure (eg. IEBC 2012 Chapter A1, ASCE 41-13 Chapter 15) have contained a section explicitly defining the purpose and/or performance objective of the provisions therein. Section 2.4.8 of FEMA 178 (which is the document most closely related to the CGSEEB) states the following:

> *"…the performance objective of [the special procedure] is significant hazard reduction, which is a lower objective than assumed for other sections of this handbook…"*

Despite the use of the word "hazard" where "risk" is clearly more appropriate, the statement is important in terms of clarifying the expected performance of the building assessed or retrofitted using the procedure. Curiously, there is no such statement in the CGSEEB. Additionally, the 2012 IEBC (Section A102.2) explicitly states that *"the provisions of this chapter shall not apply to the strengthening of buildings in Risk Categories [high importance] or [post disaster]"* buildings and recommends that such buildings be strengthened to current code requirements. The CGSEEB (Section 1.2) does state that its provisions *"will* *not necessarily assure continued post-earthquake performance,"* but does not specifically prohibit applications to evaluations/retrofits of high importance or post disaster buildings.

4.1.2. Recommended Action

It is recommended that a section be added to CGSEEB Appendix A clearly stating the performance objective and also clearly stating that the methodology should not be applied to high-importance or post-disaster buildings.

4.2. Limitations on Building Form

4.2.1. Background

The assumed dynamic behaviour underlying the special procedure is substantially different from that assumed in most modern building codes. All versions of the special procedure contain limitations on the building form intended to ensure the assumptions are not violated. Some versions of the special procedure contain more limitations than others, but one limitation recommended by the original methodology appears not to have been included in any of the subsequent documents: ABK (1984) noted that buildings founded on moderately soft soils and with in-plane wall aspect ratios greater than 1.5:1 would likely not fulfill the "rigid wall" assumption. Aspect ratios much greater are potentially permitted by CGSEEB Appendix A, which simply limits buildings to a maximum of 6 storeys. The significance of the aforementioned limitation has not been well studied in the literature.

4.2.2. Recommended Action

It is recommended that the significance of the aforementioned limitation be investigated, including nonlinear analysis capturing potentially mitigating factors such as foundation flexibility, pier rocking, and diaphragm yielding.

4.3. Seismic Hazard Parameters

4.3.1. Background

The CGSEEB (including Appendix A) was developed to be compatible with the 1990 NBCC. As such it is based on the seismic hazard parameters from that code. Appendix A is based largely on the zonal velocity ratio (v) as given in Eq 1.

The origin of the 1.3 factor is not explained in Appendix A, but it appears to account for the fact that the effective peak velocities and spectral velocities that were the basis of the ABK methodology corresponded to *"deep cohesionless or stiff clay soils"* (ABK, 1981), which equates most closely to NBCC 1990 Soil Category 2 (*"Compact coarse-grained soils; firm and stiff fine-grained soils with a depth greater than 15m"*) with a foundation factor, F, equal to 1.3.

The aforementioned zonal velocities/accelerations were calculated at a 10% probability of exceedance in 50 years and are no longer used in the NBCC, which now specifies seismic hazard parameters in the form of spectral acceleration ordinates with a probability of exceedance of 2% in 50 years. Chapter A1 of the 2012 IEBC now uses 80% of the design spectral acceleration at a period of one second (S_{D1}) as an approximation to the previously employed velocity-related acceleration coefficient (A_v), which is similar to the zonal velocity ratio (v).

4.3.2. Recommended Action

It is recommended that CGSEEB Appendix A be revised to employ seismic hazard parameters consistent with the current NBCC. One option would be to replace Equation 1 with Equation 3 (below).

$$v' = 0.8 * S_a(1) * I_E * F_v \quad [3]$$

Where:
$S_a(1)$ ≡ Spectral acceleration at T=1.0 second (NBCC 2010)
I_E ≡ Importance Factor (NBCC 2010)
F_v ≡ Long Period Foundation Factor (NBCC 2010)

4.4. Pseudoseismic Forces and Distribution

4.4.1. Background

With the walls assumed to be rigid and the diaphragm period not explicitly considered, the pseudoseismic design force is assumed to be equal at each storey in each direction (given that the seismic weight is the same). Appendix A (Section A.8) specifies a storey force at each level as the lesser of:

$$F_{wx} = v' * \left(W_{wx} + \frac{W_d}{2}\right) \quad [4]$$

$$F_{wx} = v' * W_{wx} + v_u * D \quad [5]$$

Where:
F_{wx} ≡ Storey force at each end of a diaphragm [Force]
v' ≡ Effective Zonal Velocity (NBCC 1990)
W_{wx} ≡ Weight of the wall tributary to that storey [Force]
W_d ≡ Weight tributary for diaphragm at that storey [Force]
v_u ≡ Diaphragm yield strength (as defined in Appendix A) [Force/length]
D ≡ Depth of the diaphragm [Length]

The design storey shear is then defined as the sum of the storey forces at and above the level under consideration. Note that the force is not dependent on the period of the diaphragm; equation 4 represents a somewhat flexible diaphragm: given that the response acceleration is taken to be numerically equal to v', the period of the diaphragm is effectively assumed to be well into the descending portion of a typical design acceleration spectrum and the wall is assumed to be rigid (see Figure 2). Knox (2012) and Wilson (2012) show that even straight sheathed diaphragms with aspect ratios of about 1.3:1 may have periods of 0.5s to 0.7s which would represent a substantial increase is spectral acceleration over the assumed value. In situ testing by Giongo et. al. (2015) confirmed the validity of the model proposed by Wilson (2012). Diaphragms constructed from diagonal sheathing, double layers of sheathing, or with plywood overlays also may lead to pseudoseismic elastic force demands beyond those specified by Equation 4.

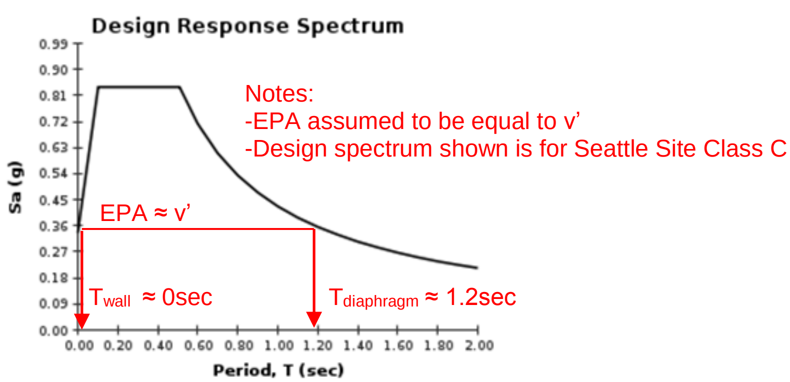

Figure 2 – Illustration of Assumed Period Underlying Equation 4

Finally, note that Appendix A allows a 25% reduction in the storey force specified by Equations 4 & 5 for buildings with qualifying crosswalls (interior partitions, as defined in the document). Based on the ABK work, these interior partitions act as yielding elements, providing hysteretic damping. More recent versions of the special procedure (ICC, 2012b; ASCE, 2013) have eliminated or significantly reduced this relaxation.

4.4.2. Recommended Action

It is recommended that the aforementioned issues be further investigated, but no specific changes are recommended herein.

4.5. Diaphragm Shear Transfer

4.5.1. Background

CGSEEB Appendix A specifies that diaphragms should be connected to shear walls for forces equating to the lesser of:

$$V_s = v' * \frac{W_d}{2} \qquad [6]$$

$$V_s = v_u * D \qquad [7]$$

Where:
$V_s \equiv$ *Diaphragm shear transfer [force]*
$v' \equiv$ *Effective zonal velocity*
$W_d \equiv$ *Weight tributary to diaphragm [force]*
$v_u \equiv$ *Diaphragm yield strength [force/length]*
$D \equiv$ *Diaphragm length [length]*

There are no capacity design provisions, as the diaphragm connections are designed for the same force as specified in Equations 4 and 5, which is the lesser of elastic demands or the specified yield capacity of the diaphragm; no increase to account for overstrength of the diaphragm is required. Other versions of the special procedure (FEMA, 1992; ICC, 2012b; ASCE, 2013) have typically specified an increase of 50-125% over specified storey forces through the use of 'C_p' factors, which are dependent upon the diaphragm construction (i.e. stiffness). However, all versions of the special procedure limit the design shear transfer to the specified yield capacity of the diaphragm.

4.5.2. Recommended Action

CGSEEB Appendix A is unconservative (relative to other versions of the special procedure) and it is recommended that Appendix A be revised to be consistent with the other versions of the special procedure. One option would be to revise Equation 6 as shown below:

$$V_s = 0.75 * v' * C_p * W_d \qquad [8]$$

Where:
$V_s \equiv$ *Diaphragm shear transfer [force]*
$v' \equiv$ *Effective zonal velocity*
$W_d \equiv$ *Weight tributary to diaphragm [force]*
$C_p \equiv$ *Component response factor (as defined below).*

The values for C_p could be adopted from Chapter A1 of the 2012 IEBC, as shown in the table below.

Table 4 – Proposed C_p Factors

Diaphragm Construction	C_p
Roofs with straight or diagonal sheathing and roofing applied directly to the sheathing, or floors with straight tongue-and-groove sheathing.	1.0
Diaphragms with double or multiple layers of boards with edges offset, and blocked plywood systems	1.5
Diaphragms of metal deck without topping and minimal welding or mechanical attachment	1.2
Diaphragms of metal deck without topping that are welded or mechanically attached for seismic resistance	1.35

Note: the Cp factors shown here have been doubled from those of IEBC 2012 Table A1-C because the coefficient in the equation has been halved, from 1.5 to 0.75.

4.6. Out-of-Plane Wall Bracing

4.6.1. Background

CGSEEB Appendix A specifies that URM walls exceeding certain height-to-thickness (h/t) ratios be braced (or otherwise retrofitted) for out-of-plane stability. The most common solution is to provide vertical bracing members (known as "strongbacks"), connected to diaphragms at floor/roof levels. Appendix A (Section A.6) specifies that bracing should be located such that:

1) Spacing between braces does not exceed 10 times the wall thickness, and
2) Distance from openings does not exceed 5 times the wall thickness

Chapter A1 of the 2012 IEBC specifies that the spacing between braces should not exceed the lesser of:

1) One-half of the unsupported wall height
2) 10 feet

The IEBC requirements are consistent with the original ABK (1984) requirements as well as those of FEMA 178 (1992) and they have the potential to be much more stringent than those of CGSEEB Appendix A: assuming a wall height of 12 feet and thickness of 13 inches, bracing would be required at 6 feet on center per the IEBC requirements versus approximately 10 feet on center per the CGSEEB requirements. CGSEEB Appendix A does not provide an explanation as to why the requirements were changed.

With regard to force demands, CGSEEB Appendix A specifies that URM walls should be anchored to the braces for $V_p=2.5*v'*W_n$ (where W_n is the wall weight tributary to the anchor) and that the deflection of such bracing should not exceed 10% of the wall thickness "at design loads". CGSEEB Appendix A is not clear as to the appropriate "design loads" for the bracing. The obvious solution is to design the bracing for the aforementioned anchorage force. However, ABK (1984) states that vertical braces should be designed for forces of just EPA*W_n and that deflection should not exceed 15% of the wall thickness. Various American documents (FEMA, 1998; ASCE, 2003; ASCE, 2013) are consistent with ABK (1984) by specifying brace design forces of $V_p=0.4*S_{DS}*W_n$, where $0.4*S_{DS}$ approximates the EPA; design forces for *anchorage* to floors and/or strongbacks are typically increased over this value. Of course, actual force demands on strongbacks depend on many factors, including the elastic properties and potential nonlinear response of the walls and diaphragms. Recent testing of unbraced wall panels (Penner, 2014) showed total peak anchorage force demands were up to 1.7 times greater than that predicted using the spectral acceleration (at the period of the diaphragm) times the tributary weight of the wall. The presence of strongbacks would substantially reduce or eliminate cracking/rocking and the resulting impact forces, leading to decreased force demands on the wall anchors. Finally, it is noted that Chapter A1 of the 2012 IEBC specifies design forces for braces (and anchorage to of walls to braces) as 75% of the demands per the parts and portions section of the "building code." Similar to other documents, it is specified that the deflection of such bracing be limited to 10% of the wall thickness.

Finally, it is noted that the requirements for "intermediate braces" (diagonal kicker braces connected to the wall and ceiling structure, reducing the unsupported wall height) in the various versions of the special procedure are similar, though not identical. In passing, it is noted that the effectiveness of this style of bracing came into question after observed failures in the 1994 Northridge earthquake (Bruneau, 1995). See FEMA 547 (2006) for further information on the aforementioned bracing methods.

4.6.2. Recommended Actions

In the absence of any supporting evidence for the increased brace spacing in CGSEEB (which is likely the case given that no buildings strengthened to the CGSEEB requirements have been subjected to significant shaking), it is recommended that the spacing requirements be revised to be consistent with the IEBC. Additionally, it is recommended that the "design load" for bracing be clarified; in the authors' opinion, the provisions of ASCE 41 (2013) are the most robust and suitable. Note that using the "parts and portions" provisions from the NBCC is perhaps inappropriate as these provisions are based on modern structural systems with rigid diaphragms (eg. concrete or steel buildings).

4.7. Vertical Elements of the SFRS

4.7.1. Background

Provisions for vertical elements of the SFRS are reasonably similar in CGSEEB Appendix A and IEBC 2012 Chapter A1. However, there are two notable differences:

1) IEBC Section A111.6.4 requires that moment frames added as part of a retrofit that are not in a line with other SFRS elements be designed in accordance with the "building code" and that the interstorey drift ratio be limited to 1.5% whereas CGSEEB Appendix A does not provide any guidance on this item. Note that both documents allow moment frames in line with URM walls provided the moments frames are designed for 100% of the tributary seismic forces, all piers in the line are rocking-controlled, and the drift ratio is limited to 0.75%.

2) CGSEEB Appendix A specifies that, where all piers in a line of resistance are rocking-controlled, they may be designed for 60% of the storey force demand; IEBC Chapter A1 has reduced the relaxation slightly, allowing such piers to be designed for 70% of the storey shear demand

4.7.2. Recommended Actions

It is recommended that the CGSEEB be revised to be consistent with the IEBC for both of the above-noted items.

4.8. Nonstructural Components

4.8.1. Background

CGSEEB Appendix A contains provisions for parapets, but not for veneers or non-bearing URM partitions. Other versions of the special procedure have typically included provisions for veneers and non-bearing URM partitions.

4.8.2. Recommended Actions

It is recommended that the provisions for veneers and non-bearing URM partitions of the 2012 IEBC Chapter A1 be adopted.

5. Conclusions

Although the CGSEEB represented state-of-the-art practices at the time of its publication, no updates have been published. The document was created to be compatible with the NBCC 1990 and the NBCC has subsequently been updated in 1995, 2005, 2010, and 2015. Similarly, several updates have been published to various American versions of the special procedure. Indeed, the entire CGSEEB requires updating, but the focus herein has been on Appendix A for URM buildings. Issues were highlighted herein along with potential updates aimed at rendering CGSEEB Appendix A compatible with current code seismicity and also more consistent with recently published American counterparts, such as Chapter A1 of the 2012 IEBC. It is noted that the observations and recommendations presented herein are of a small group of individuals and that changes to design documents such as the CGSEEB require careful review and input by a large host of experts and stakeholders. The intent of the information presented herein is solely to draw attention to this important matter and prompt further discussion and investigation.

An interesting alternative to CGSEEB Appendix A is the use of the performance-based provisions of ASCE 41-13. Although some judgment would certainly be required for application in Canada, it is actively maintained and provides a platform to explicitly discuss the performance of proposed retrofits with building officials and owners.

6. References

ABK. (1981). ABK TR-02 Methodology For Mitigation of Seismic Hazards in Unreinforced Masonry Buildings: Seismic Input. El Segundo, CA, USA: ABK Joint Venture.

ABK. (1984). ABK TR-08 Methodology For Mitigation of Seismic Hazards in Unreinforced Masonry Buildings: The Methodology. El Segundo, CA, USA: ABK Joint Venture.

ASCE. (2003). ASCE 31-03 - Seismic Evaluation of Existing Buildings. Reston, VA, USA: American Society of Civil Engineers.

ASCE. (2006). Seismic Rehabilitation of Existing Buildings. Reston, VA, USA: American Society of Civil Engineers.

ASCE. (2013). Seismic Evaluation and Retrofit of Existing Buildings. Reston, VA, USA: American Society of Civil Engineers.

ATC. (1978). Tentative Provisions for the Development of Seismic Regulations for Buildings. Palo Alto, CA, USA: Applied Technology Council.

Bruneau, M. (1994). Seismic Evaluation of Unreinforced Masonry Buildings - A State-of-The-Art Report. Canadian Journal of Civil Engineering, vol. 21, 512-539.

City of Los Angeles. (1987). Rule of General Application #1-87 Establishing an Alternate Design Methodology for Unreinforced Masonry Buildings. Los Angeles, CA, USA.

City of San Francisco. (2013). San Francisco Building Code, Chapter 16C. San Francisco, CA, USA.

FEMA. (1992). NEHRP Handbook for Seismic Evaluation of Existing Buildings. Washington, DC, USA: Federal Emergency Management Agency.

FEMA. (1992b). FEMA 178 - NEHRP Handbook for Seismic Evaluation of Existing Buildings. Washington, DC, USA: Federal Emergency Management Agency.

FEMA. (1998). Handbook for Seismic Evaluation of Buildings. Washington, DC, USA: Federal Emergency Management Agency.

GSC. (2014). Open File 7576 Fifth Generation Seismic Hazard Model Input Files as Proposed to Produce Values for the 2015 National Building Code of Canada. Geological Survey of Canada.

Heidebrecht, A. (2003). Overview of Seismic Provisions of The Proposed 2005 Edition of The National Building Code of Canada. Canadian Journal of Civil Engineering, vol. 30, pp. 241-254.

ICBO. (1997). Uniform Code for Building Conservation. Whittier, CA, USA: International Conference of Building Officials.

ICBO. (2001). Guidelines for Seismic Retrofit of Existing Buildings. Whittier, CA, USA: International Conference of Building Officials.

ICC. (2003). International Existing Building Code. Country Club Hills, IL, USA: International Code Conference.

ICC. (2012a). 2012 International Building Code. Country Club Hills, IL, USA: International Code Conference.

ICC. (2012b). International Existing Building Code. Country Club Hills, IL, USA: International Code Conference.

Ingham, J., & Griffith, M. (2011). The Performance of Earthquake Strengthened URM Buildings in The Christchuch CBD in The 22 February 2011 Earthquake. New Zealand Royal Commission of Inquiry.

Knox, C. (2012). Assessment of Perforated Unreinforced Masonry Walls Responding In-Plane. Auckland, New Zealand: University of Auckland.

LATF. (1994). Findings and Recommendations of The Unreinforced Masonry Building Subcommittee of The City of Los Angeles Department of Building Safety and The Structural Engineers Association of Southern California Task Force. Los Angeles: Los Angeles Task Force.

NRC. (1990). 1990 National Building Code of Canada. Ottawa, ON, Canada: National Research Council of Canada.

NRC. (1992). Guidelines for Seismic Evaluation of Existing Buildings. Ottawa, ON: National Research Council of Canada.

NRC. (1995). Guideline for Seismic Upgrading of Building Structures. Ottawa, CA: National Research Council of Canada.

NRC. (2010). National Building Code of Canada. Ottawa, ON, Canada: National Research Council of Canada.

NZS. (2004). Structural Design Actions, Part 5 - Earthquake Actions. New Zealand Standards.

Paxton, B. (2014). Quantifying Unreinforced Masonry Seismic Risk and Mitigation Options in Victoria, BC. Vancouver, BC, Canada: University of British Columbia.

Paxton, B., Turner, F., Elwood, K., & Ingham, J. (2015). URM Bearing Wall Building Seismic Risk Mitigation on The West Coast of The United States: A Review of Policies and Practices. Bulletin of The New Zealand Society for Earthquake Engineering, vol. no. 48, issue no. 1.

Rutherford and Chekene. (1997). Development of Procedures to Enhance The Performance of Rehabilitated URM Buildings. Gaithersburg, MD, USA: National Institute of Standards and Technology.

SEAOC. (1989). Ten issues Raised by SEAOC Task Committee on Earthquake Hazard Reduction of URM Buildings. Structural Engineers Association of California.

Tena-Colunga, A., & Abrams, D. (1992). Response of an Unreinforced Masonry Building during the Loma Prieta Earhtquake. Tenth World Conference on Earthquake Engineering. Rotterdam, Netherlands.

University of Auckland. (2011). Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance. Auckland, New Zealand: University of Auckland.

Wilson, A. (2012). Seismic Assessment of Timber Floor Diaphragms in Unreinforced Masonry Buildings. Auckland, New Zealand: University of Auckland.