COMPARATIVE STUDY OF MASONRY SEISMIC RESPONSE TO THE WEST COAST SOIL SPECTRUM AND EAST COAST ROCK SPECTRUM IN NUCLEAR POWER PLANTS

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ABSTRACT

Seismic qualification of existing masonry walls in Nuclear Power Plants (NPPs) has been challenging due to perceived masonry vulnerability to the lateral forces caused by a seismic event. Due to some of the recent earthquakes exceeding the design basis, beyond design basis evaluation of NPPs is required by the regulators all across the world including Canada. The existing masonry construction is required to be evaluated for its seismic interaction with the seismically qualified systems and components. In some of the probabilistic risk assessment studies, the unreinforced masonry (URM) has been found to be one of the dominant contributors to the reactor core damage frequency.

The Design Basis Earthquake (DBE) spectrum recommended by CSA N289.3 is similar to the NBK spectrum (by Newmark, Blume and Kapoor) based on various California earthquake records and hence is referred to as the west coast spectrum. Recent research indicates that the hazard at rock sites on the east coast of North America is better represented by a spectrum having lower spectral accelerations than the west coast spectrum at the low frequency range and higher spectral accelerations than the west coast spectrum over the high frequency range up to 100 Hz. Such a spectrum is known as the East North American (ENA) spectrum. Masonry walls in NPPs in Eastern Canada that are designed for the west coast spectrum are required to be evaluated for the ENA spectrum. This paper focuses on the beyond design basis evaluation of typical masonry wall configurations that can be found in Canadian NPPs and attempts to find their seismic margin over and above the design or evaluation basis.

KEYWORDS: concrete block masonry, nuclear power plants, seismic response, beyond design basis seismic evaluation

INTRODUCTION

NPPs in Canada can be divided into two categories depending on their seismic design. The NPPs built in the early days of nuclear age had limited seismic design basis, whereas the relatively newer NPPs were seismically qualified for a properly defined DBE [1]. The older plants with no DBE, have been assessed in the recent past on the basis of a less probable and stronger seismic event known as the Review Level Earthquake (RLE) by utilizing the Seismic Margin Assessment (SMA) methodology [2]. Since SMA is based on a less probable earthquake, the allowable stresses for masonry are higher than those for DBE based design. RLE is the basis for seismic evaluation, while the DBE is the basis for seismic design. SMA (in comparison to probabilistic risk assessment) is easier to be understood and incorporated by engineers used to pass/fail criteria. It recommends the RLE to be the 84^{th} percentile Uniform Hazard Spectrum with $1x10^{-4}$ probability of exceedence per annum, to be treated as a deterministic hazard for the Conservative Deterministic Failure Margin method based on pass/fail criteria with higher allowable stresses than what is recommended by the design codes. SMA captures the features of the probabilistic risk assessment methods (high confidence and low probability of failure) while maintaining the simplicity of the design methods. Complete details about the SMA methodology can be found elsewhere [3]. Since the SMA of the NPPs without DBEs was performed in the last decade, the chosen RLEs for such plants were based on the latest research having high frequency content similar to the ENA spectrum [4]. For the purpose of evaluation or capacity assessment of an existing NPP, the RLE's probability of occurrence has to be lower than the DBE's in order to identify possible vulnerabilities in existing NPPs. Hence the RLE (being stronger than the DBE due to lower probability) generally corresponds to higher peak ground acceleration (PGA) or zero period acceleration (ZPA) than the DBE and hence is interpreted as a stronger earthquake than the DBE.

Apart from the design requirement to be based on the DBE, CSA N289.3 [5] also requires evaluation of structures, systems and components for a seismic event known as checking level earthquake having lesser probability of occurrence than the DBE whereas the 1981 version of the same standard requires the design to be based on the DBE with no further evaluation. Hence the plants (constructed in the late 1970's and early 1980's) based on the DBE having low frequency content are required to be evaluated for a high frequency checking level earthquake based on the latest research. This calls for evaluation of masonry for a response spectrum having high frequency content whose design is based on the DBE with low frequency content. This paper highlights the inherent challenges in this process and attempts to evaluate the margin between the capacity and design/evaluation basis by comparing the dynamic characteristics of masonry for the two types of seismic events whose frequency contents are entirely different.

BEYOND DESIGN BASIS SEISMIC EVENTS AND ACCEPTIBILITY CRITERIA

According to the United States Nuclear Regulatory Commission (USNRC) document on the 2007 Niigata-Chuetsu-Oki Earthquake" in Japan [6], the NPP at Kashiwazagi Kariwa experienced the ground motion with peak ground accelerations exceeding more than two times the design basis. With regard to the 2011 North Anna (Virginia) earthquake, the USNRC summary sheet [7] states that, "at several frequencies, the spectral and peak ground accelerations as a result of the August 23, 2011 earthquake were greater than those used for the Operating Basis and Design Basis Earthquakes." In such situations, post event walk-downs are necessary in order to know the status of the NPP. In beyond design basis events, the URM may crack at various locations. Once cracked, the URM cannot be considered as qualified for the design basis earthquake if its original qualification was based on the gross section. Such walls would require evaluation for their revised capacities. Good seismic qualification book keeping is very helpful in identifying the vulnerable locations and making the post event walk-downs more focused on the components at risk.

The standard CSA N291 [8] mandates the elastic response for the safety related structures for design. The seismic interaction of masonry (mainly due to collapse) with the surrounding equipment is the governing criterion for evaluation (for the RLE) in accordance with the SMA methodology [3]. Hence the design is based on the elastic behaviour whereas the evaluation is based on non-linear behaviour aiming at prevention of collapse as the acceptability criterion. However, if masonry supports piping or some other system, then its acceptability criteria would depend on that particular system. In certain situations, small safety related components are installed on masonry where it is essential to know the acceleration imparted by masonry to that component. Because of the difference in frequency content of the DBE and the RLE, a rigid RM may attract only the ZPA on the DBE spectrum but would be susceptible to much higher acceleration on the RLE spectrum and hence would tend to soften after cracking without yielding, exhibiting a decrease in its apparent frequency between cracking and yielding.

MASONRY APPLICATIONS IN CANADIAN NPPs

In NPPs, masonry is found almost everywhere ranging from the conventional area (where the turbines are located) to areas prone to nuclear radiation such as the reactor building. Masonry has been found to be a predominant factor leading to the reactor core damage frequency in some of the probabilistic risk assessment studies [9]. Some of the typical applications [10] are summarized below:

Radiation Protection Walls – This is one of the most common applications of masonry in the areas prone to radiation. These are generally 8 to 12 in thick and 8 to 10 ft high fully-grouted reinforced hollow concrete block walls cantilevered from the floor designed for out of plane bending.

Other applications – URM is an excellent fire barrier but it may be closer to a stairwell which may be a part of an essential seismically qualified escape route for certain personnel. URM is also used in box structures surrounding seismically qualified equipment or used as an office enclosure inside the NPP.

Components installed on masonry- In NPPs, sometimes small seismically qualified electronic components may be installed on masonry walls. Such components are often rigid and respond to high frequency excitations. The acceleration imparted to these components would depend on the masonry response. When a wall cracks, its effective period increases which, depending on what portion of the response spectrum the un-cracked wall frequency is in, results in amplified or reduced acceleration. For the purpose of evaluation of such components, it is essential to know the peak acceleration response of the masonry between the cracked and un-cracked frequencies because of the large difference between the two. The cost of inelastic analysis by expensive and time-consuming software is prohibitive for simple masonry wall applications.

ENA AND CSA SPECTRA

The standard CSA N289.3 [5] states that the generic response spectrum is based on the California records and recommends the formulation of a site-specific response spectrum addressing the issue of the east coast rocks. Nevertheless, the generic response spectrum for the east coast sites is absent from the standard CSA N289.3 [5] despite the fact that all the NPPs in Canada are situated on the east coast. Atkinson and Elgohary [4] have attempted to fill this gap by introducing a generic response spectrum for the east coast sites known as the ENA spectrum. A comparison of the ENA spectrum and the CSA spectrum anchored at 0.05g ZPA for 5% damping is given in Figure 1 where the difference between the frequency contents of the two spectra is clear. The damping is considered as 5% (compatible with masonry). The ENA spectrum being rich in high frequency content continues up to 100 Hz and beyond but for the purpose of evaluation, it is considered to have achieved the ZPA at 100 Hz. The CSA spectrum is similar to the NBK spectrum developed by Newmark, Blume and Kapoor [11 & 12]. Some of the older NPPs (without a DBE) have been evaluated for the RLE (similar to the ENA spectrum) [2] and the relatively newer NPPs (seismically qualified for the DBE) are to be evaluated for the RLE (likely to be similar to the ENA spectrum).

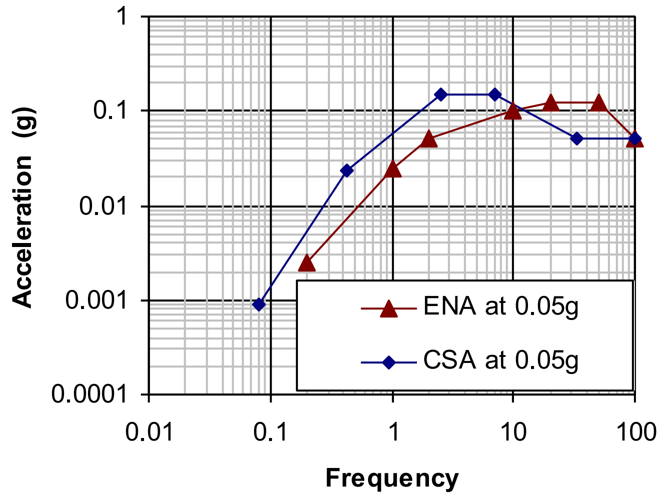

Figure 1: CSA N289.3 generic response spectrum and ENA response spectrum at 5% damping (Both anchored to 0.05g ZPA for frequency comparison)

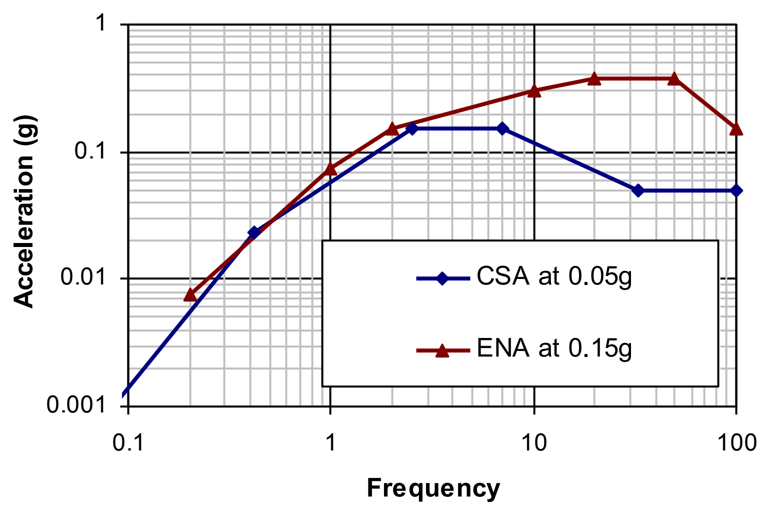

Figure 2: CSA N289.3 generic response spectrum and ENA response spectrum anchored at 0.05g and 0.15g ZPAs respectively with 5% damping. (Both similar to the events incorporated in some of the SMA studies)

In order to evaluate the direct approximate impact of the change in the frequency contents, it is more prudent to examine the difference between the two response spectra directly despite the difference in the probability levels and in the allowable stresses of various structures, systems and components. Figure 2 compares the generic response spectrum recommended by CSA N289.3 [5] with the ENA response spectrum recommended by Atkinson and Elgohary [4]; both at 5% damping. For convenience of comparison and to establish the similarity to the chosen RLEs of the existing plants [2], the CSA spectrum is anchored at 0.05g ZPA and the ENA spectrum is anchored at 0.15g ZPA.

Looking at the SMA studies of the existing NPPs in Ontario, Canada [2& 13] and similarities of their DBEs with the CSA spectrum and of their RLEs with the ENA spectrum, the ZPAs of 0.05g and 0.15g are found to be appropriate for the purpose of the comparative study in this paper. The probability levels of these two spectra and allowable stresses are entirely different but are irrelevant for the purpose of discussion here because the focus is on the comparison of the frequency content and their acceleration levels for seismic screening rather than the pass/fail analysis. The difference in the acceleration levels is much higher than the allowable stresses given in the SMA methodology and hence the emphasis on frequency and acceleration comparison is justified to arrive at the results with a reasonable accuracy. For example in Figure 2, the acceleration of the ENA spectrum at 25 Hz is approximately seven times the acceleration of the CSA spectrum. This gap cannot be bridged by any amount of increase in the allowable stresses by inspection especially in the case of URM. At lower frequencies, the CSA spectrum is over conservative and hence the masonry designed for the CSA or similar spectrum would be acceptable if evaluated for the ENA spectrum.

URM AND RM FREQUENCIES

The thickness of the URM or RM depending on the radiation protection requirements can vary from 8 to 16 in. For the purpose of discussions in this paper, three typical thicknesses, 8, 10 and 12 in are considered. Table 1 and 2 give details of these walls and Table 3 contains the fundamental frequencies considering un-cracked and cracked sections. Due to cracking and the non-linear stress strain properties of masonry it is “impossible” to calculate a “fundamentally correct value of moment of inertia” [14]. However, the range of frequencies would depend upon the gross section and cracked section.

Clauses 7.7.6.4 and 10.7.4.4 of the Canadian standard S304.1 [15] recommend the following relationships for the effective flexural stiffness represented by $(EI)_{eff}$ for post cracking behaviour.

For URM $$(EI)_{eff} = 0.4E_m I_o \tag{1}$$

For RM $$(EI)_{eff} = E_m\left[0.25I_o - (0.25I_o - I_{cr})\frac{e-e_k}{2e_k}\right] \quad but\ E_m I_{cr} \le (EI)_{eff} \le 0.25E_m I_o \tag{2}$$

Where E_m is the modulus of elasticity as defined by the standard S304.1 [15], I_o is the gross moment of inertia, I_{cr} is the cracked moment of inertia, e_k is the Kern eccentricity and e is the eccentricity of the axial load. The post cracking behavior of URM would induce rocking which is not covered by the standard S304.1 [15] and is beyond the scope of this paper.

Due to the wall heights not being too large and bending being out of plane, the effect of the restoring moment due to the self weight has been ignored. For the purpose of evaluation of RM, the focus here is on the upper bound and lower bound frequencies and also on what happens in between the two limits. Before cracking the RM would assume the un-cracked frequency based on E_mI_o (and not on $(EI)_{eff}$) and after cracking the maximum value of $(EI)_{eff}$ would lead to the maximum frequency. This frequency would most likely be on the left of the peak of the response spectrum curve attracting the maximum post cracking acceleration. Hence the maximum value of $(EI)_{eff}$ (which is $0.25E_mI_o$) would attract the highest post cracking acceleration. However masonry may soften further and the entire range of effective flexural stiffness is required to be investigated. Table 3 provides two of such frequencies for three cases of RM. For simplicity and for the purpose of understanding the effect of frequency variation on seismic acceleration, it is assumed that the wall does not support any load other than the self weight.

Table 1: Fully-grouted Masonry Cases

Case I	190 mm 20 MPa block with No 20 rebar at 600mm c/c
Case II	240 mm 20 MPa block with No 20 rebar at 600mm c/c
Case III	290 mm 20 MPa block with No 20 rebar at 600mm c/c

Table 2: Common Properties

f'_m (MPa)	f'_t (MPa)	g (mm/sec^2)	$E_m = 850\ f'_m$ (MPa)
10	0.4	9810	8500

Table 3: Un-cracked and Cracked Natural Frequencies

CANTILEVER BLOCK WALL FREQUENCIES								
Width = 1m								
Case	Wall Height l	Self Weight w	Gross Moment of Inertia I_o	E_mI_o	*Natural frequency based on EI_o	$EI_{eff} = 0.25E_mI_o$	*Natural frequency based on $(EI)_{eff}$	Cracking acceleration a_{cr}
	(mm)	(N/mm)	(mm^4)	($N.mm^2$)	(Hz)	($N.mm^2$)	(Hz)	(g)
I	2200	4.12	5.72E+08	4.862E+12	12.42	1.22E+12	6.23	0.24
II	2000	5.22	1.15E+09	9.792E+12	18.95	2.45E+12	9.50	0.37
III	1500	6.32	2.03E+09	1.7272E+13	40.67	4.32E+12	20.39	0.79
*Natural frequency of a cantilever with horizontal uniformly distributed load = $(3.52/2\pi)\sqrt{(EIg/wl^4)}$ [16]. Axial load due to self weight is ignored.								

SEISMIC SCREENING OF BLOCK WALLS

For evaluation of a number of block walls in a NPP in the high frequency requirement, screening facilitates minimizing the cost of analysis which would be limited to only those walls which cannot be screened out. For the purpose of discussion here, the effect of the resistance factors recommended by S304.1 is not considered. Out of the three cases in Table 1, Case I is taken as an example. The following process can be followed for screening:

1. For Case I in table 3, the un-cracked frequency is 12.42 Hz with the corresponding cracking acceleration of 0.24g. Looking at Figure 2, the corresponding spectral acceleration of CSA spectrum for 12.42 Hz frequency is less than the cracking acceleration. Hence this wall is screened out for the CSA spectrum.
2. For the ENA spectrum, the spectral acceleration at 12.42 Hz (being more than its un-cracked capacity acceleration) would cause the wall to crack altering its frequency. The wall is required to be evaluated for frequencies between 6.23Hz and fully cracked frequency. Hence it cannot be screened out without further evaluation in detail.

The wall in Case III can be screened out for both the spectra without further evaluation since its cracking acceleration is more than the maximum peak spectral accelerations for both spectra whereas Case II may require further investigation.

DETERMINATION OF BEYOND DESIGN BASIS SEISMIC MARGIN

The ultimate capacity is considered as acceptable for the beyond design basis events because the aim is to prevent collapse on the surrounding systems. However, the existing masonry's ultimate capacity may correspond to very low frequency which may never be experienced by it for a given response spectrum as demonstrated in the example below. Hence it is necessary to locate a given masonry at one point on the given response spectrum. The following procedure may be adopted to arrive at the combination of frequency and acceleration in order to establish the seismic margin of a cantilevered RM over and above the design or evaluation basis with no imposed load except the self-weight. This procedure is also useful in determining the amplified acceleration imparted by masonry to a component installed on its surface. The effect of self-weight is ignored in order to be conservative. CSA N287.3 [17] recommends flexural stiffness to be based on the average of gross and cracked moment of inertia whereas ACI-318 (2011) [18] recommends 35 to 50 percent of gross moment of inertia. For this procedure, the Canadian masonry code [1511] is followed. Figure 3 demonstrates this procedure assuming the cracked frequency to be on the left of the peak of evaluation spectrum.

1. Calculate cracking acceleration a_{cr} and un-cracked frequency f_{uncr}. (Figure 3)
2. Compare cracking acceleration with evaluation spectrum acceleration. (Point A).
3. Since the spectral acceleration is higher than the cracking acceleration, the wall will crack. Calculate maximum moment for the spectral acceleration (at point A) at the fixed end.
4. Calculate frequency with $(EI)_{eff} = EI_{cr}$. (The acceleration and frequency correspond to point B)

   $$f = \frac{3.52}{2\pi}\sqrt{\frac{(EI)_{eff}\, g}{wl^4}} \tag{3}$$

   where g is the gravitational acceleration [16] and the other symbols are given in Table 3.

5. But at this point the masonry would attract much lower acceleration. (Point C).
6. The solution lies in between A and C but to be conservative, consider highest value of $(EI)_{eff}$ allowed by the code to get the maximum acceleration.
7. Choose maximum value of $(EI)_{eff} = 0.25EI_o$ (Per masonry code)
8. Find new frequency for this value of $(EI)_{eff}$. Its intersection with the response spectrum is point D.
9. Calculate maximum moment (Ma) for the acceleration at D and calculate I_{ef} by the masonry code [15] clause 11.4.3.2 given below.

$$I_{ef} = \left(\frac{M_{cr}}{M_a}\right)^3 I_o + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \tag{4}$$

where the moment M_{cr} corresponds to a_{cr}.

10. Repeat the cycle starting with new $(EI)_{eff} = E_m I_{ef}$. It may happen that the solution lies between A and D. In that case we stop at D to be conservative.
11. Assuming it converges at point E, calculate moment M_E for the acceleration at point E.
12. Calculate ultimate moment capacity M_u per masonry code.
13. If $M_u > M_E$, seismic margin beyond evaluation earthquake per masonry code = M_u/M_E.
14. The acceleration imparted to the component installed on masonry is the acceleration at point E.
15. If the un-cracked frequency is at the right of the peak of the response spectrum, cracking may lead to the peak acceleration and masonry has to be evaluated for it.
16. The method described above assumes the first mode behavior (with 70% mass excitation) and the higher modes for uniform cantilevered beams are expected to be at much higher frequencies than the first mode. Their effect can be investigated with Square Root of Sum of Squares (SRSS) method if more accuracy is desired.

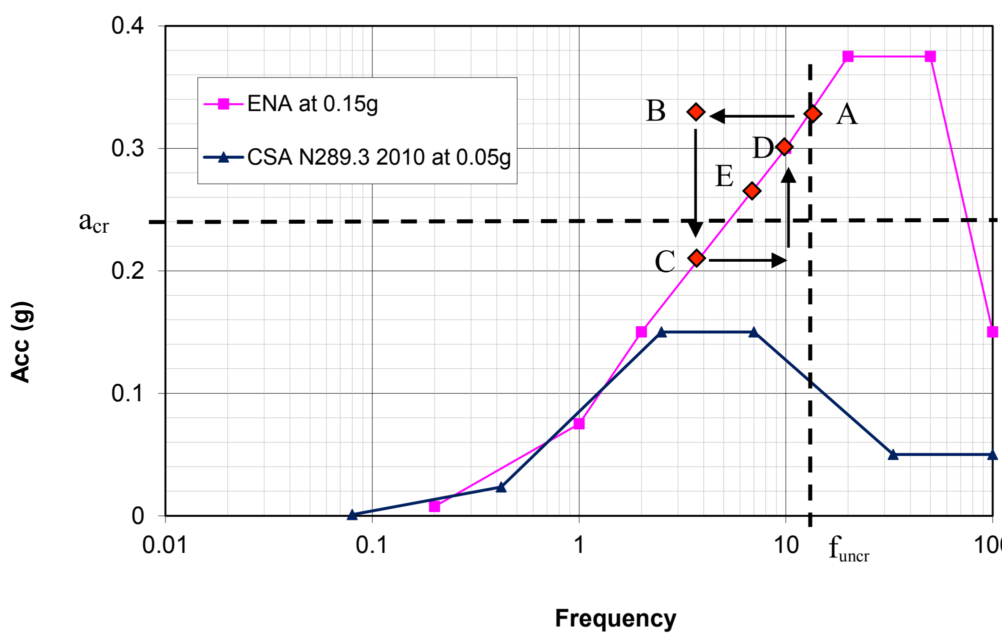

Figure 3: Iterative process to arrive at the frequency-acceleration combination on the evaluation spectrum for a given masonry wall

BOX STRUCTURES

The box structures inside the plant generally contain four hollow concrete block walls with the steel deck on top. Such structures are generally seismically rugged because two walls out of four would behave as shear walls which receive the lateral load transferred by the steel deck due to diaphragm action. However such structures are rigid and they require special attention for the high frequency content of the response spectrum. The shear walls are generally squat walls susceptible to the damage by shear rather than in-plane bending. The walls are also required to be seismically qualified for out of plane bending.

CURVED WALLS

In high radiation areas inside containment structures such as the reactor building, sometimes curved masonry is required to be qualified for the high frequency ENA spectrum. One such example is shown in Figure 4 where a 8 in thick fully grouted concrete block masonry is shown. This wall is divided into approximately 400 mm x400 mm elements pin supported at the base as shown. Modal frequencies are given in Table 4 along with the mass participation factors. For the purpose of discussion and the prevailing trend in the industry, the number of modes is restricted up to 90% cumulative mass participation, which is obtained in all three directions at the 23rd mode. The frequency range of various modes from the first to the 23rd mode is between 34 and 289 Hz. The CSA spectrum in Figure 2 achieves its PGA 0.05g at 33 Hz. Hence for the CSA spectrum this masonry would be subject to 0.05g acceleration for all the modes. However for the ENA spectrum, the same masonry construction would be subject to much higher accelerations in this frequency range. The seismic analysis for this masonry becomes somewhat complicated because of the modal combination techniques pertaining to closely spaced modes. SRSS or any other acceptable modal combination rule may be adopted. This refined analysis is essential to avoid un-necessary modifications leading to the unwanted project costs.

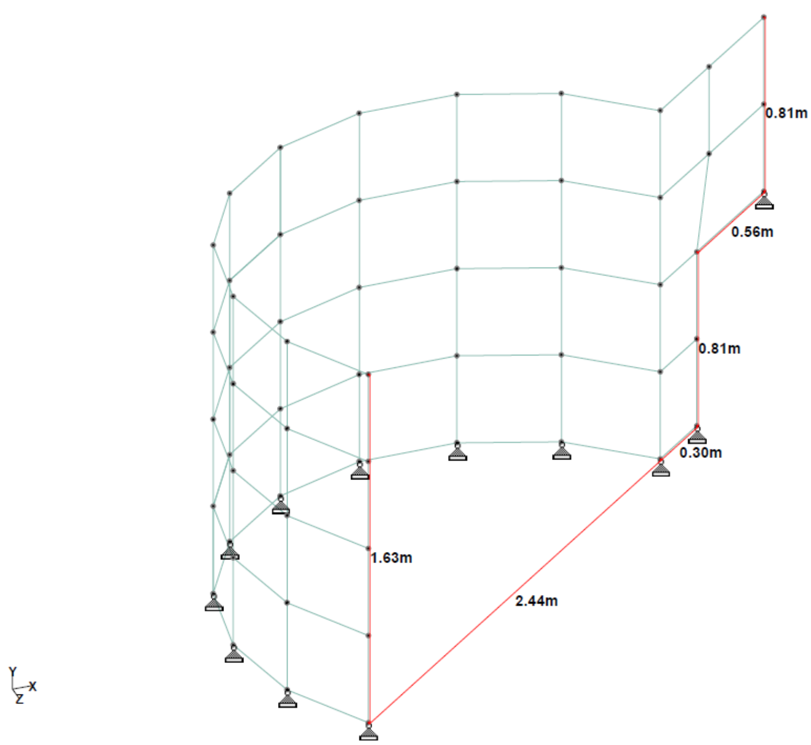

Figure 4: 8 in Thick Curved Masonry Shielding Wall

Table 4: Curved Masonry Un-cracked Modal Frequencies and Cumulative Participation

Mode	Frequency Hz	Cumulative Participation X %	Cumulative Participation Y %	Cumulative Participation Z %
1	34	16	0	4
2	56	16	0	27
3	68	39	0	42
4	82	40	0	60
5	100	45	0	63
6	118	49	0	64
7	132	58	0	70
8	149	85	2	75
9	157	85	2	81
10	179	86	3	83
11	180	91	3	86
12	188	91	4	90
13	209	91	4	90
14	222	92	4	90
15	228	92	15	90
16	238	94	33	91
17	240	94	35	91
18	242	94	75	91
19	247	94	84	91
20	254	94	85	92
21	264	94	89	94
22	273	94	89	94
23	289	94	90	94

CONCLUSIONS

The Canadian NPPs designed for the DBE based on California earthquake records from the west coast (close to the Pacific Ocean) are required to be evaluated for the ENA spectrum (based on the east coast records) which has considerably higher demand than the DBE over the high frequency range. The frequency contents of the two spectra are significantly different from each other. Many masonry applications in a NPP are found to be susceptible to high frequency excitation. Such structures do not attract more than the ZPA for the DBE whereas in the case of high frequency ENA spectrum, they are likely to respond to much higher acceleration levels. Establishing seismic margin of existing masonry for out of plane bending over a given response spectrum requires determination of acceleration experienced by masonry for that particular spectrum. Cracking of masonry alters its frequency making it practically impossible to determine the correct moment of inertia. Various standards suggest different values for the cracked moment of inertia. A practical iterative approach is described for predicting the acceleration-frequency combination of reinforced masonry to assess its seismic margin over a given response spectrum.

The iterative procedure can also be used to assess the seismic acceleration experienced by rigid small safety components installed on masonry surface.

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