DEVELOPMENT OF FLOOR DESIGN SPECTRA FOR OPERATIONAL AND FUNCTIONAL COMPONENTS OF CONCRETE BUILDINGS IN CANADA

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ABSTRACT :

Previous earthquakes have clearly demonstrated that the damage to operational and functional components of buildings usually cause more injuries, fatalities, and property and financial loss than those inflicted by structural damage. Operational and functional components of a building include architectural components, mechanical and electrical equipment, and building contents. An important step towards ensuring the safe design of operational and functional components of buildings is to develop floor design response spectra. This constitutes the scope of the current research program described in this paper. A total of 6 buildings, consisting of 5, 10 and 15-story frame buildings were designed and analyzed to establish floor response spectra. The buildings were designed for Ottawa and Vancouver representing Eastern and Western Canadian seismicity. Fifteen artificially generated earthquake records (accelerograms) were used for each structure, compatible with the Uniform Hazard Response Spectra specified in the 2005 edition of the National Building Code of Canada. It was observed that the response amplifications relative to ground excitations varied from floor to floor, and were frequency dependent. Generally, the higher floors showed higher amplification with differences in spectra between the floors being more pronounced in low-rise buildings, as compared to medium and high-rise buildings. The acceleration floor response spectra for individual floors are presented in the paper with details of the analyses that led to their developments. Recommendations are made for design floor response spectra for reinforced concrete frame buildings in Canada.

KEYWORDS: Earthquakes, floor response spectra, frames, nonlinear analysis, non-structural components, operational and functional components, response spectra.

1. INTRODUCTION

Past earthquakes have shown that the damage to operational and functional components of buildings usually cause more injuries, fatalities, property, and financial loss than those inflicted by structural damage. There have been many incidences that a building, which sustained only minor structural damage, was deemed unsafe and unusable as a result of extensive damage to its operational and functional components. Failure of such components also poses serious problems for search and rescue operations after the earthquake, resulting in additional and unnecessary increases in casualties. Equipment failures and the debris caused by falling objects could critically affect the performance of vital facilities such as emergency command centers, fire and police stations, hospitals, power stations and water supply plants. During the 1994 Northridge earthquake in California, several major hospitals had to be evacuated, not because of structural damage, but because of the failure of emergency power systems, air control units, falling ceilings and light fixtures. In Canada, the 1988 Saguenay earthquake, the strongest event in eastern North America recorded within the last 50 years, caused very little structural damage. It has been well documented that a great majority of the injuries, property damage, and economic loss was caused by the failure of operational and functional components (OFC) in buildings.

Operational and functional components of buildings include:

- **Architectural components** – parapets, claddings, partitions, stairways, lighting systems, suspended ceilings, etc.
- **Mechanical and electrical equipment** – pipes and ducts, escalators, central control panels, transformers, emergency power systems, fire protection systems, machinery, etc.
- **Building contents** – books and shelves, furniture, file cabinets, storage racks, etc.

A rational approach to designing operational and functional components of buildings includes the use of floor design response spectra. Such design spectra, conforming to the 2005 National Building Code of Canada (NBCC 2005) are currently not available in Canada. The NBCC-2005 reflects the most recent seismic hazard data available, based on Uniform Hazard Spectra (UHS). The current research program is an effort towards filling this gap and meeting the needs of the design profession in terms of providing floor design spectra compatible with the current seismic hazard levels incorporated in NBCC-2005. This was achieved through comprehensive dynamic analyses of selected reinforced concrete frame buildings. Accordingly, three buildings were designed for Vancouver, reflecting the western Canadian seismicity and an additional three buildings were designed for Ottawa, reflecting the eastern Canadian seismicity. The buildings had 5, 10 and 15 stories, covering a wide range of periods of vibrations for frame building. NBCC-2005 UHS compatible earthquake records were used to conduct dynamic inelastic response history analyses. Acceleration time histories were obtained from the analyses and response spectrum was constructed for each floor under each record. Mean, and mean plus standard deviation response spectra were generated to develop floor design response spectra. The details of the procedure followed, and the floor response spectra obtained are presented in the following sections.

2. REINFORCED CONCRETE BUILDINGS SELECTED

A total of six reinforced concrete buildings, designed on the basis of the seismic provisions of NBCC-2005 were selected. The buildings consisted of 5, 10 and 15 storey heights. Moment resisting frame buildings were considered in Vancouver and Ottawa for each of the three building heights. The frame buildings in Vancouver included cases where the buildings were designed without considerations for the current drift limitations, including cases that may also represent buildings built prior to the enactment of modern building codes. Figures 1 and 2 show the floor plan and elevation views of buildings. The floor plan was selected to be symmetrical to minimize the effects of torsion. The analyses were conducted in the short direction of plan. The lateral load resisting system was considered to consist of bare building frames, without any participation from non-structural elements, such as architectural and masonry enclosures and/or partition walls. Therefore, their periods may be longer than those with significant participation of non-structural components providing unintentional stiffening effects. The fundamental period computed for 5, 10 and 15-storey buildings were 1.75 sec, 3.47 sec and 5.11 sec, respectively.

The design force levels were computed on the basis of the seismic design provisions of NBCC-2005. UHS provided for Vancouver and Ottawa were used to reflect the appropriate seismic hazard levels. The buildings in Vancouver were designed and detailed as fully ductile buildings, using the ductility related force modification factor, $R_d = 4.0$. The buildings in Ottawa, on the hand, were designed to be moderately ductile, with $R_d = 2.5$. All buildings were assumed to be located on firm soil (Class “C” in NBCC-2005), without the amplification effects of soft soils. The force levels were based on building periods obtained by employing the empirically developed period expressions suggested by the code, incorporating field data and the possibility of additional stiffening by non-structural elements, with due allowances made by the code for relaxation up to 50%, provided the computed periods based on bare frame models showed higher values.

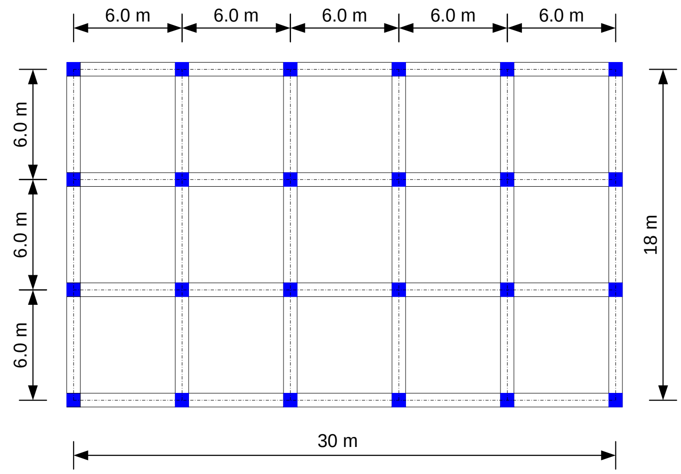


Figure 1 Typical floor plan for the frame buildings selected

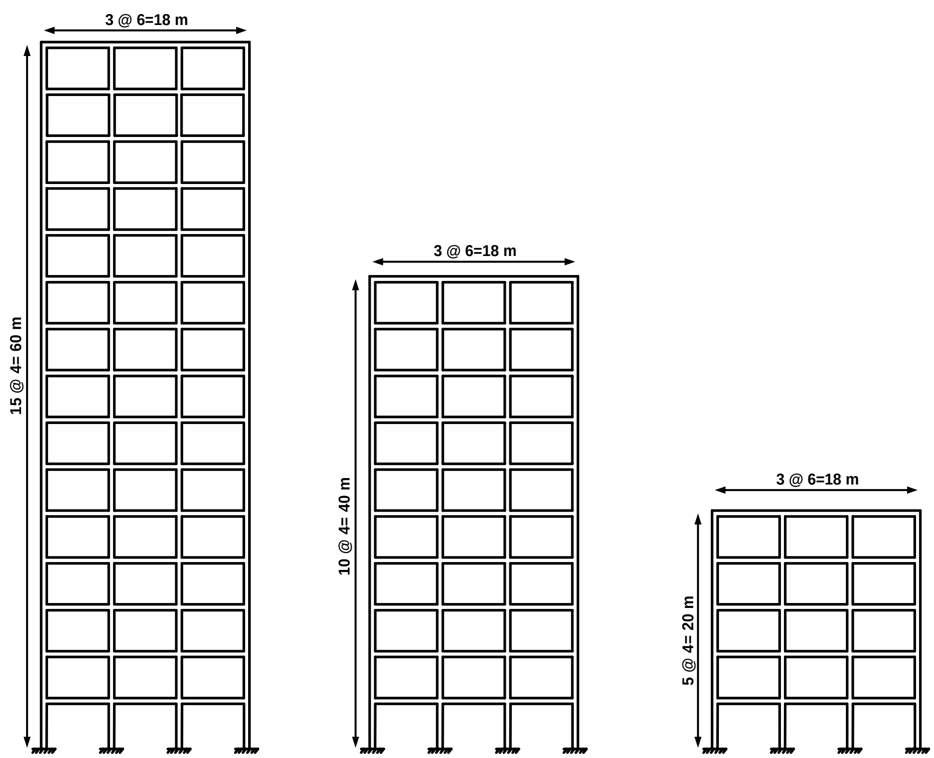


Figure 2 Elevation views of buildings selected

The buildings were analyzed using equivalent static loads and computer software SAP2000 to establish design forces. Two-dimensional static analyses were conducted using preliminary member dimensions to determine the critical values of bending moments, axial forces and shear forces at each joint. These values were then used to design the beams and columns, based on the CSA Standard A23.3 (2004). The analyses were repeated with revised member sizes when necessary.

3. GROUND MOTION RECORDS EMPLOYED

The current research project involves development of floor response spectra for reinforced concrete frame buildings in Canada. Therefore, the ground motion records required for dynamic analysis need to reflect the Canadian seismicity. Although a large database of California earthquakes is available, they may not be applicable to seismic analyses of buildings in Canada, because of differences in their frequency content. Therefore, the use of actual California earthquake records is not feasible. Instead, simulated records, compatible with target spectra are needed.

The seismic hazard maps of Canada have been updated by the Geological Survey of Canada (GSC) recently. The GSC proposed a new seismic hazard model that includes new knowledge from recent earthquakes, new strong motion relations and better understanding of site conditions. The newly proposed hazard computation was made for 2% in 50 year probability level, instead of the 10% in 50 years used earlier. The seismicty is presented in the form of uniform hazard spectra (UHS), incorporating the composite effects of earthquakes that contribute most strongly to the hazard at the specified probability level. These spectra were adopted by the National Building Code of Canada in 2005, in an idealized manner. Accordingly, the spectral accelerations are specified at 0.2, 0.5, 1.0 and 2.0 second periods. These four spectral quantities allow the construction of approximate uniform hazard spectrum for each location. For the objectives of the current investigation, it is important to select ground motion records that are compatible with the UHS given in the code. Atkinson and Beresnev (1998) developed artificially generated records with a view that they provide realistic representations of ground motions for selected earthquake magnitudes and distances that contribute to hazard at respective locations and for selected probability levels. In their approach, more than one record is used to match the target UHS. Four horizontal components are generated for a moderate nearby earthquake, and four horizontal components are generated for a larger distant earthquake with a total of eight records for every selected city. This was found necessary because moderate nearby earthquakes would not produce sufficiently long-period energy to match the long-period band whereas the larger distant earthquakes would have lost too much high-frequency energy to match the short-period band. The records suggested by Atkinson and Beresnev (1998) were selected for use in the current investigation. The records selected correspond to the probability of occurrence of 2% in 50 years, as they were compatible with the uniform hazard spectra specified in NBCC-2005. For Ottawa, 15 artificial records were selected from M (magnitude) 6.0 and M7.0. The distance for these records varied from 25 km to 50 km for M6.0 and from 50 km to 100 km for M7.0. Fifteen different records were selected for Vancouver. They included 7 records from M6.5 (short event) and 8 records from M7.2 (long event).

4. DYNAMIC INELASTIC RESPONSE HISTORY ANALYSES

Analysis of structures due to earthquake motions should be done in two orthogonal directions, separately and independently. Most computer software includes planar frame models. Three-dimensional structures can be modeled as a series of two-dimensional frames linked together to reflect the actual strength and stiffness in each direction. In a two-dimensional analysis, the structure is idealized as a planar assemblage of discrete elements. The buildings designed in the current investigation all have four interior and two exterior frames with three bays in the short direction. Due to the rigid slab diaphragm assumption, each frame was assumed to have the same horizontal deflection at each floor level. Hence, the frames were modeled by connecting them in series with rigid links. The links were modeled by specifying rigid beam elements having infinitely high axial rigidities and infinitely small flexural rigidities. These elements transfer lateral forces without moment connections between the frames.

Each of building designed was modeled having six frames connected to each other by rigid links. Identical frames with the same member sizes, strengths and stiffness parameters were lumped into one frame to reduce the number of elements and the required processing time. Thus, the structures were modeled as two frames connected by rigid links, where one of the frames represented the strength and stiffness of four interior frames and the other represented the same for two exterior frames. This is shown in Figure 3 for the 10-storey frame building.

Computer software DRAIN-2DX (Prakash, V. et al. 1993) was used to carry out nonlinear time history analysis of buildings. DRAIN-2DX is a general purpose dynamic analysis program that utilizes the stiffness method of structural analysis. Dynamic equation of motion is solved numerically through the step-by-step linear acceleration method. Three degrees of freedom were used at each node, consisting of X and Y translations and R rotation about the Z-axis. Flexural yield strengths for all structural elements are defined as part of the parameters that defined the hysteretic model. The element stiffness was defined with due considerations given to concrete cracking. The flexural rigidities were taken as 70% and 35% of those computed based on gross sectional properties for vertical and columns and beams, respectively. Takeda's hysteretic model (Takeda et al. 1970) was employed to describe the element stiffness during loading, unloading and reloading under earthquake excitations. Figure 4 shows the hysteretic model used for flexure. Structural mass was assumed to be concentrated at each floor level and was specified at each node. Damping was specified as 5% of critical damping, consisting of stiffness and mass dependent components. Response time histories of floor accelerations were obtained as output.

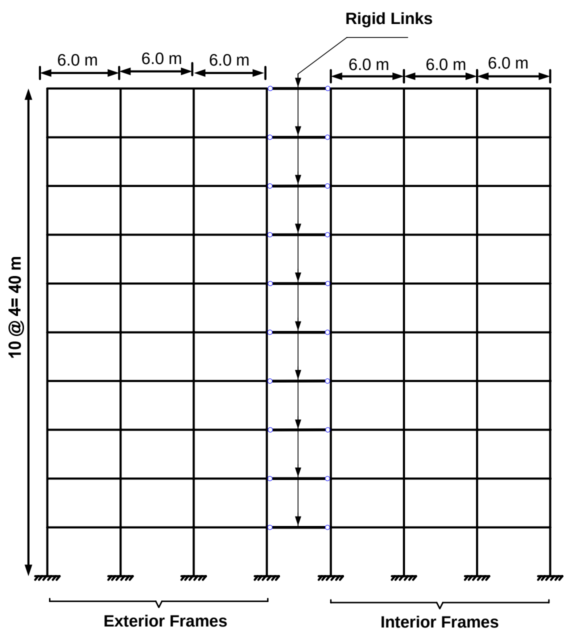


Figure 3 Frame model used in analysis

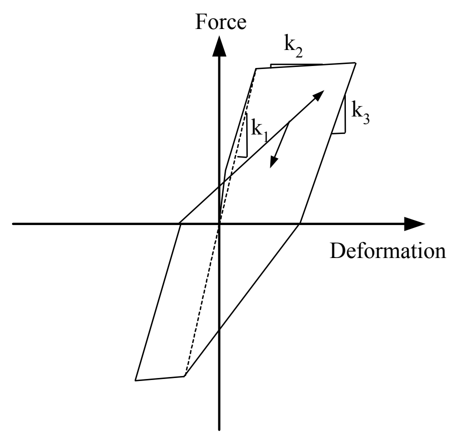
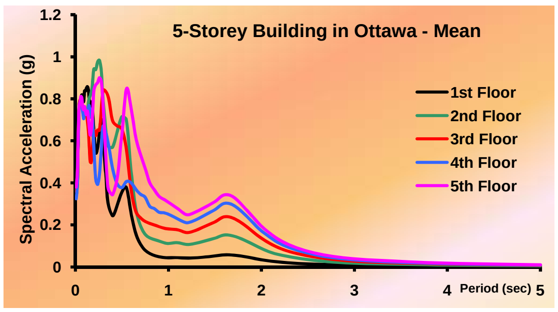


Figure 4 Takeda's hysteretic model

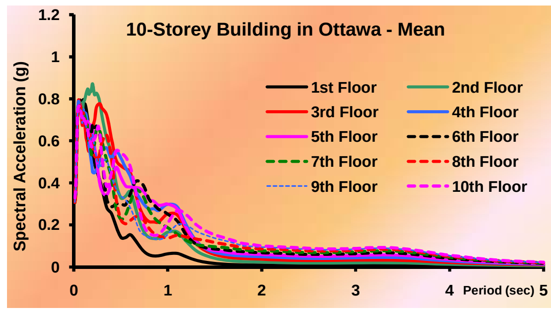
5. DESIGN FLOOR RESPONSE SPECTRA

Dynamic analyses of buildings under 30 earthquake records resulted in a total of 900 floor response spectra for buildings in Ottawa and Vancouver, one for each floor of each building. The mean spectral relationships were computed for each floor. Figure 5 shows the acceleration response spectra at all levels of buildings designed for Ottawa and Vancouver.

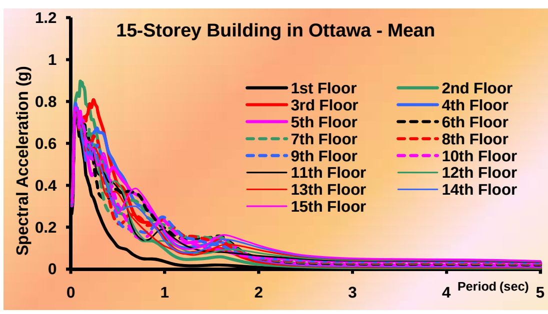
The examination of response spectra indicates that there is a progressive increase in response going from the first floor to upper floors. The rate of response amplification is higher in low-rise buildings as compared to companion medium and high-rise buildings. The 5-storey building showed an approximate amplification factor of 4 for roof response relative to the response of the first-storey. However this amplification factor was approximately 3 and 2 for 10-storey and 15-storey buildings. This may be explained by the ground motion response spectra for buildings with different fundamental periods. The amplification of response for each building was higher in the short period range relative to the medium and long period ranges, indicating the period dependence of response acceleration. The response spectra further indicated a higher amplification for buildings in Vancouver which were subjected to stronger ground motions, relative to those in Ottawa.



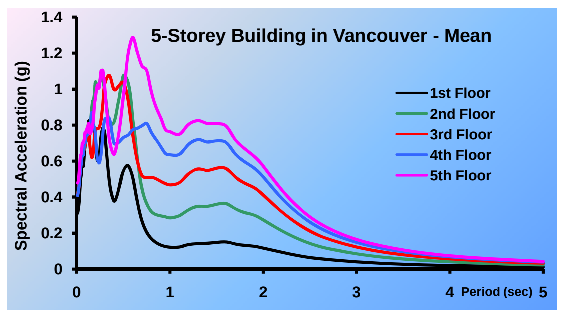
a) 5-Storey building in Ottawa



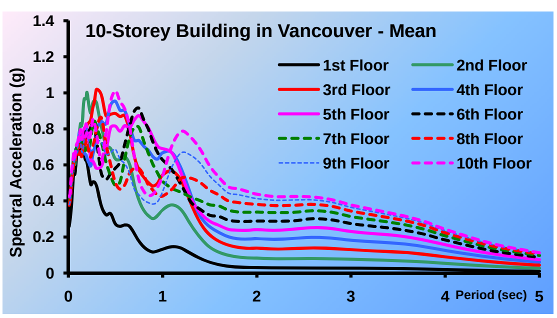
b) 10-Storey Building in Ottawa



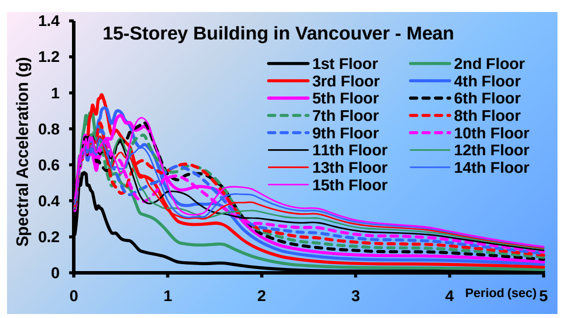
c) 15-Storey building in Ottawa



d) 5-Storey Building in Vancouver



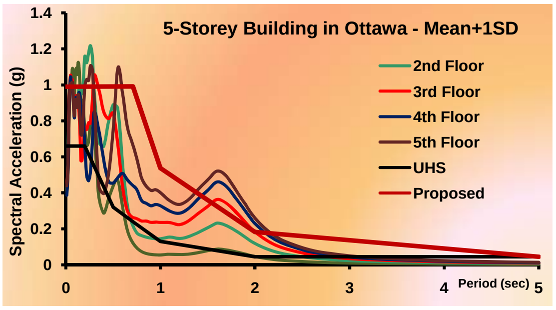
e) 10-Storey building in Vancouver



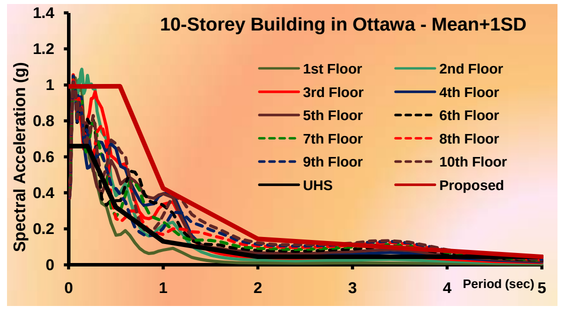
f) 15-Storey Building in Vancouver

Figure 5 Floor response spectra – Mean values

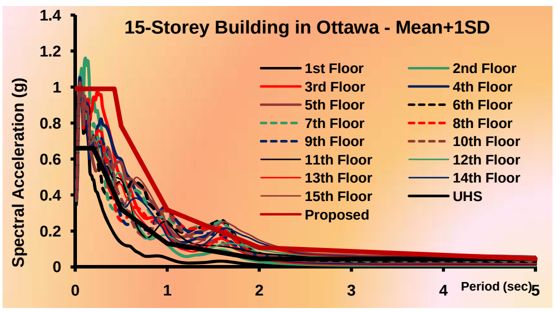
Floor response spectra were also constructed for mean plus one standard deviation values. These spectra were deemed appropriate for the development of floor design spectra. This is shown in Figure 6 for all buildings. The figure also includes the UHS for respective cities, depicting the amount of amplification observed at each floor level relative to ground. An upper bound design spectrum was derived through curve fitting, while incorporating the effects of period.



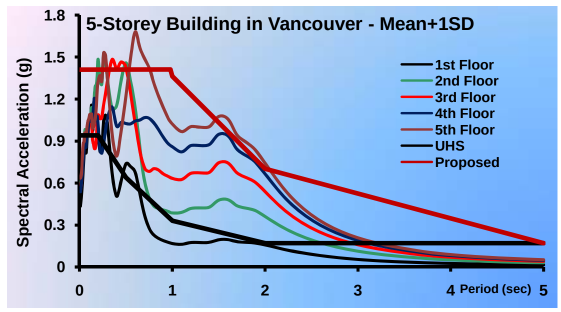
a) 5-Storey building in Ottawa



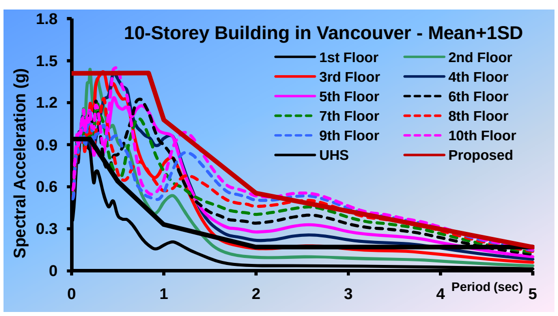
b) 10-Storey Building in Ottawa



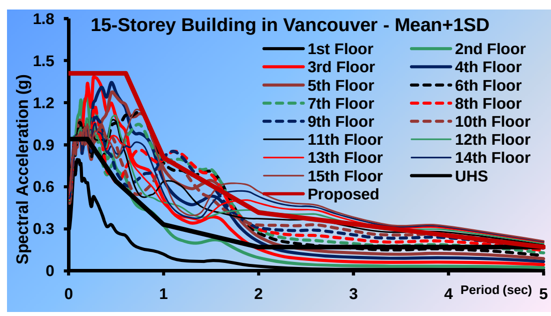
c) 15-Storey building in Ottawa



d) 5-Storey Building in Vancouver



e) 10-Storey building in Vancouver



f) 15-Storey Building in Vancouver

Figure 6 Floor response spectra – Mean plus one standard deviation values

The following expression may be used to compute the amplification factor for floor response spectra relative to UHS.

$$C = 5.0 - 0.5T_a \tag{1}$$

Where T_a is the fundamental period of building housing the OFC in seconds, computed by an accepted method of mechanics; and C is the response amplification factor relative to UHS.

The floor design spectral acceleration values for a building on firm ground, $S_f(T)$ can be computed from the expression given below:

$$S_f(T) = C\, S(T_a) \leq 1.5\, S(0.2) \qquad \text{for } T \leq 2.0 \text{ sec} \tag{2}$$

Where, C is the floor response amplification factor defined in Eq. 1; T is the vibration period of the OFC to be designed; T_a is the fundamental period of the building that houses the OFC, computed using accepted methods of mechanics; $S(T_a)$ is the UHS value specified in NBCC-2005 for firm ground (Class "C" soil). The floor design spectrum has a cutoff value at 1.5 S(0.2). Beyond T = 2.0 sec, the use of constant UHS, i.e., S(T > 2.0) = S(2.0) appears to overestimate the spectral values obtained from the use of artificially generated ground motions. Further amplifications of UHS to obtain floor response spectra overestimates the floor acceleration values obtained from dynamic analysis. Therefore, it is suggested that the amplification introduced beyond T = 2.0 sec is reduced linearly as the period increases from 2.0 sec to 5.0 sec to merge with the UHS vale at T = 5.0 sec. i.e., $S_f(5.0) = S(5.0)$, and follow the UHS thereafter. The floor design spectra obtained by Eq. 2 are plotted in Figure 6 as the proposed spectra for design, providing upper bound values to mean plus one standard deviation floor design spectral accelerations. Figure 6 indicates that the proposed floor design spectra represent spectral values for the roof, whereas the UHS can be used as the floor response spectra for the first floor. A linear interpolation can be made between the two sets of spectra for in between floors.

6. CONCLUSIONS

The following conclusions can be derived from the analytical research project presented in this paper:

- Floor response spectra show amplified spectral accelerations relative to the ground spectral accelerations as described by UHS.
- The amplification of floor spectral accelerations is the highest at the roof level. The amplification factors for roof is approximately 4.0 for 5-storey frame buildings, 3.0 for 10-storey frame buildings and 2.0 for 15 storey frame buildings. The amplification decreases gradually going from the roof level to the first storey level. The first storey floor response spectra approach to UHS for the ground.
- Floor to floor amplification of spectral accelerations are more pronounced in low-rise buildings. The spectra for different floors tend to approach each other as buildings become taller.
- Eqs. 1 and 2, developed as part of the current research project, can be used to generate floor design spectra from the UHS specified for a given location.

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