Design Drift Limits for Performance-based Seismic Assessment and Retrofit Design of School Buildings in British Columbia, Canada

A. Bebamzadeh, C.E.Ventura, B.H. Pandey, & W.D. Liam Finn
The University of British Columbia, Canada

G. Taylor
TBG Seismic Consultant Ltd, Canada

SUMMARY

The Province of British Columbia (BC), on the West Coast of Canada, is currently implementing a seismic mitigation program for over 750 provincial schools. A state-of-the-art performance-based methodology is being utilized in this program to provide a cost-effective tool for seismic assessment and retrofit design for the provincial school buildings. The life safety performance objective has been selected as the sole performance objective for seismic assessment and retrofit design of school buildings. The main goal in this methodology is to achieve optimum life safety by reducing the probability of structural damage to acceptable levels. This paper presents an innovative probabilistic approach that has been implemented to determine the crucial Design Drift Limit (DDL) for the life safety performance objective. The DDL is the maximum permissible inelastic drift that ensures the structural damage to the building components meets the minimum specified performance requirements. In this approach, the total seismic hazard data for the BC region is de-aggregated by considering three types of sources (i.e. crustal, subcrustal and subduction) separately. A suite of 20 ground motions representing high and moderate seismicity is used for each hazard type to perform incremental dynamic analysis and obtain the Probability of Drift Exceedance (PDE). The analysis covers a range of structural systems including wood framing, steel framing, concrete framing, concrete masonry, and clay brick masonry. This paper gives the Design Drift Limits (DDLs) for any seismicity, soil site class, and structural system in BC to achieve the life safety performance objective. The overall conclusion is that the DDL needs to be determined for the parametric combination of seismicity, storey height and soil type as opposed to the fixed DDL approach given in the building code. This approach can be adopted to any non-school low-rise buildings and to regions of different seismicity.

Keywords: Assessment and Retrofit, Incremental Non-linear Dynamic Analysis, Inelastic Deformation, Lateral Deformation Resisting Systems

1. INTRODUCTION

The province of BC has a wide variety of school building systems, over diverse seismicity regions. Thus, the need to develop a cost effective seismic mitigation program within a reasonable period of time is a real challenge requiring a unique approach. As a part of the Seismic Retrofit Guideline, First Edition (SRG1, 2011), this paper presents an innovative performance-based design methodology providing a cost-effective tool for seismic assessment and retrofit design for the provincial school buildings. Even though life safety criterion is considered as the sole performance objective in this study, other performance criteria can be developed through the same framework.

Probabilistic estimates of exceeding the selected maximum deformation levels are utilized for each building system for three potential earthquake sources: crustal, sub-crustal, and subduction, including the distance from the likely earthquakes and the local soil type. The Design Drift Limit (DDL) is then developed so that the probabilities of exceeding structural damages to the building components meet the life safety performance requirement. This paper focuses on the methodology and analysis to set the DDL values for the different Lateral Deformation Resisting Systems (LDRS). A sensitivity analysis is

also carried out to illustrate the impact of variation in DDL values for a range of seismicity regions, LDRS storey heights, and soil conditions.

2. RELIANCE ON INELASTIC DEFORMATIONS

The building code for new construction is based on force-based design (NBCC 2010). The required forces are determined based on the probability of occurrence of earthquakes with specific intensity levels. For example, the current Canadian Building Code (NBCC 2010) uses a two percent in 50 years probability of exceedance ground motions. This paper presents a performance-based philosophy which complements the Building Code for new construction by providing a rational method for life safe and cost effective retrofit of existing buildings. The key philosophy in this approach is to achieve life safety by reducing the probability of structural collapse to acceptable levels instead of concentrating on damage prevention. This study uses non-linear dynamic response deformation limits to achieve appropriate levels of life safety probability. The limits are established so that there is less than a two percent probability in 50 years that the selected drift for the selected building will be exceeded. This is essentially the same criterion used for new construction except it is based on inelastic deformations rather than forces obtained from linear analyses. The same methodology implemented in this paper may be further developed and utilized in the future to achieve goals other than life safety such as damage prevention and/or achieving immediate post-earthquake occupancy of buildings.

3. SEISMIC HAZARDS AND GROUND MOTIONS

British Columbia has significant hazard contributions from crustal, subcrustal, and subduction earthquakes. Uniform Hazard Spectrum (UHS) that envelops the spectral acceleration values from all three earthquake types is the basis of the design response spectrum of the National Building Code of Canada, NBCC (2010). In this study, in order to reduce the conservatism used in NBCC approach, the total seismic hazard data is de-aggregated by considering each type of the seismic sources separately. Verification of these results is done by conducting a comparison with the open source data provided by the Geological Survey of Canada, GSC (Adams and Halchuk, 2003). In order to be consistent with the GSC approach, the crustal and subcrustal data are treated probabilistically and the subduction data is treated deterministically. The analysis are done by employing the attenuation relationships, given by Young et al. (1997), for subcrustal and subduction earthquakes and relationships, developed by Boore et al. (1993), for crustal earthquakes. The probabilistic data are generated for both the H model (historical) and the R model (regional) based on the historical data and geological and tectonic considerations in Canada. While for the deterministic data, the Cascadia subduction fault source model is used.

In order to choose the earthquake records, British Columbia is divided into two regions: moderate-to-high and low seismicity. For each seismic sources and regions, a set of ten earthquake records is selected. In fact, six suites of ten motions are used for the calculation of the total seismic risk in this study. Eventually, each record is scaled to its corresponding source Uniform Hazard Spectra (crustal, subcrustal and subduction) (Pina et al. 2010a).

4. NON-LINEAR CHARACTERISTICS OF STRUCTURAL SYSTEMS

Incremental Non-linear Dynamic Analysis (IDA – Vamvatsikos and Cornell, 2001) is the analytical basis of the performance-based methodology in this paper. The purpose of the non-linear dynamic analyses is to determine the appropriate lateral strength of the structural system that will provide a high probability that the lateral deformation remains less than the specified maximum based on the non-linear characteristics of the structural system or component. The analytical model assumes a two-story building while its results are utilized for one through three-story school buildings with proper adjustments. All analyses include the additional deformations due to P-delta effect.
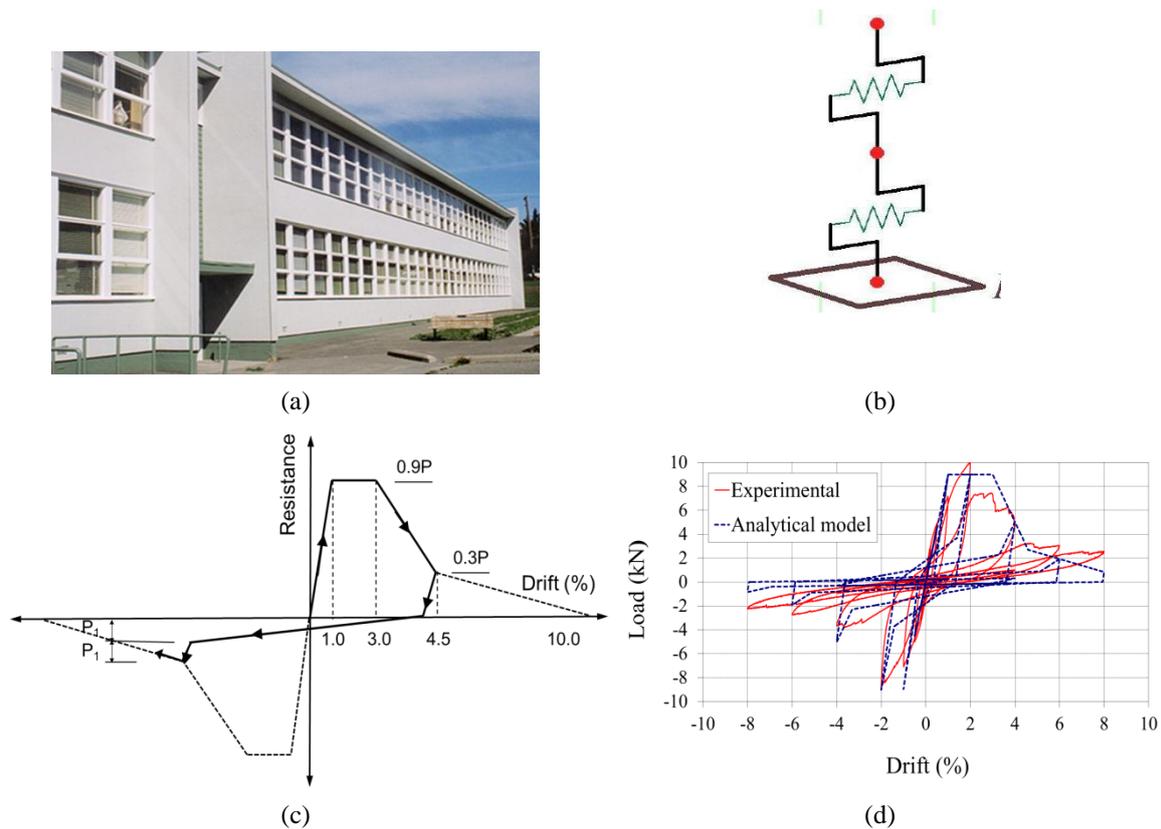
Figure 4.1. (a) Typical wood school building with blocked OSB / plywood lateral deformation resisting system (b) Analytical model (c) Hysteretic behaviour model (d) Comparison of experimental and analytical model

The primary Lateral Deformation Resisting Systems (LDRS) included in the dynamic response analyses are wood framing, steel framing, concrete framing, concrete masonry, and clay brick masonry in various construction styles. This study covers 27 types of LDRSs, as listed in Table 6.1, some of which are not allowed by the current code for new building construction, and new systems are designed in accordance with the new building construction codes. The cyclic force-deformation behavior of each existing and new structural system is modelled based on the experimental data gathered from many sources and by this project (SRG1 2011). Fig. 4.1. (a) shows a typical two-storey school building with blocked OSB / plywood LDRS system, which is modelled as a series of shear springs (Fig. 4.1. b). The cyclic force-deformation of blocked OSB / plywood LDRS (Fig.4.1 c) is based on the experimental results from the quasi-static in-plane tests at UBC (SRG1 2011). Fig. 4.1.d compares the hysteretic behaviour used in the analytical model with the experimental data.

5. PROBABILITY OF DESIGN DRIFT EXCEEDANCE (PDE)

Probability of Drift Exceedance (PDE) for a LDRS system is the percent probability that the governing drift limit will be exceeded over 50 years for all levels of shaking and for all types of earthquakes. For each combination of resistance, ground motion and earthquake hazard type, a suite of motions is used to perform IDA. The LDRS model is analysed for each of the suites of the ground motions and for each level of shaking in 10% increments, from 10% level of shaking to 250% of its base spectral scaled values. The Conditional Probability of Drift Exceedance (CPSE) is then calculated using a log-normal fit of the non-linear dynamic analysis results. The annual rate of drift exceedance is calculated by multiplying the individual CPDE for each level of shaking by its probability of occurrence (based on data from the Canadian Geological Survey) and then summing the contributions from all levels of shaking and each hazard type as follows (Pina et al. 2010b).:

$$\lambda(dr > Dr) = \int CPDE(dr > Dr\big|_{Sa}) d\lambda_{Sa} \tag{5.1}$$

where $\Delta\lambda_{Sa}$ is the rate of annual frequencies of ground motions with intensity S_a, which is directly calculated from the Probabilistic Seismic Hazard Analyses (PSHA – Kramer 1996). CPDE is the conditional probability of drift exceedance at a given intensity S_a. The total annual rate of drift exceedance is then calculated by summing up the rates over all sources of hazards (e.g., crustal, subcrustal, and subduction). The PDE is estimated using the temporal Poisson probability model at a given time interval, T, as follows:

$$PSE(dr > Dr) = 1 - \exp\left(-T\sum_{i=1}^{n}\lambda_i\right) \tag{5.2}$$

where n is the number of earthquake hazard sources. The crustal and subcrustal sources are treated probabilistically. These sources are included in the calculation of PDE. However, the subduction data is treated deterministically by checking the CPDE for the 100% level of shaking for the subduction ground motion at high levels of drift (close to failure).

This procedure is carried out incrementally for a wide range of resistances so that relationship between PDE and resistance can be determined. Fig 5.1. (a) through (d) show the probability of drift exceedance (PDE) for 4 types of LDRS systems at different drift levels. The LDRS storey height is 3m and is located in Vancouver at a Site Class C (very dense sand or soft rock NBCC 2010). The LDRS resistance R_m is given as percentage of seismic weight (W) at a given floor or roof location.
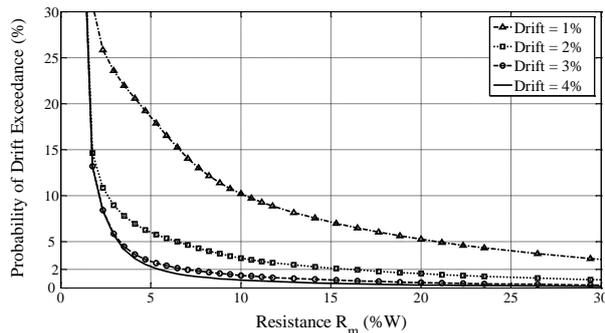
(a) Blocked OSB / plywood shear wall
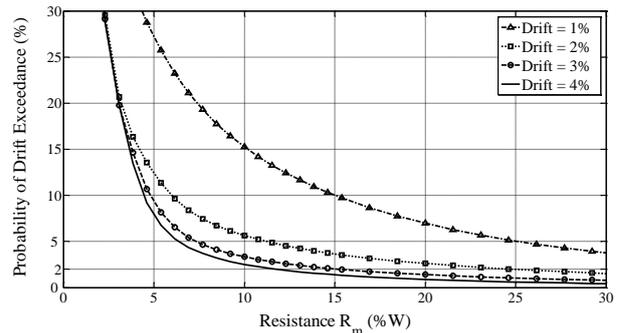
(b) Concentric steel braced frame (tension only – moderately ductile)
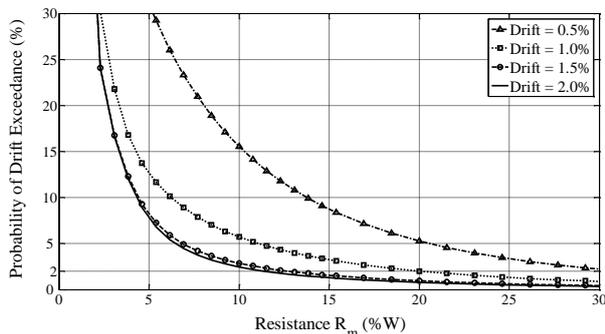
(c) Concrete Squat shearwall (shear)
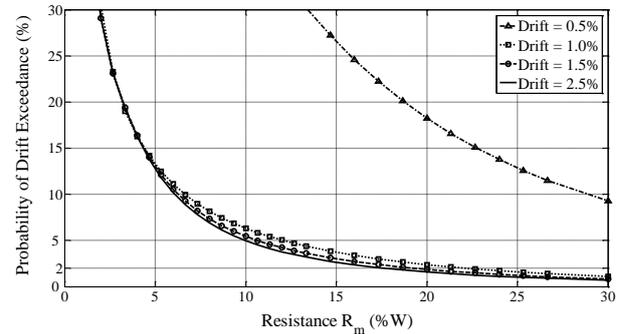
(d) Reinforced masonry wall

Figure 5.1. PDE versus R_m for different LDRS systems with 3m storey height in Vancouver and Site Class C

Fig. 5.1. (a) illustrates that the probability of exceedance of 3% and 4% drifts merge together at most of resistance and hazard levels. This suggests that drift levels greater than 4%, which are at the degradation part of the backbone curve (Fig. 5.1. c), are close to failure of the blocked OSB / plywood

shearwall. For moderately ductile concentric steel braced frame (tension only), the failure drift is 4% (Fig. 5.1. b). However, this drift is reduced to 1.5% and 2.5% for non-ductile squat shear wall (Fig. 5.1. c) and reinforced masonry walls (Fig. 5.1. d).

6. SELECTION OF DESIGN DRIFT LIMIT (DDL)

The life safety performance objective has been considered in this methodology as the primary performance objective for both the seismic risk assessment and retrofit design of school buildings. The DDL is the maximum drift that can be used in the retrofit design or seismic risk assessment. Qualitatively, this limit represents a damage level in a building that is considered "life-safe." The life safety performance objective is defined by the following two performance requirements:

(a) Probability of Design Drift Exceedance (PDE) $\leq$ 2% in a period of 50 years. This requirement ensures that the maximum inelastic drift does not exceed the appropriate Design Drift Limit (DDL) within the acceptable level of risk.

(b) Conditional Probability of Strain Exceedance (CPDE) $\leq$ 25% for near-failure conditions for the 100% level of shaking.

Specifically, the maximum DDL is set such that the required resistance for the 2% PDE does not result in a CPDE in excess of 25%. Fig. 6.2. (a) shows the minimum required resistance (R_m) for the life safety retrofit design for 2% PDE in a period of 50 years. The design resistances are presented for a 3m storey height blocked OSB / plywood LDRS located in Vancouver on Site Class C. The flat part of the curves denotes the failure drift for a hazard level of 2% in 50 years. The design drift of DDL=3% with resistance of Rm = 7.0% (W) is selected. At this resistance level, the conditional probability of exceeding (CPDE) collapse drift (drift greater than 4%) is 24% (Fig. 6.2. b). Table 6.1. summarizes the DDL and the required R_m resistance values for 27 types of LDRS.

(a)

(b)
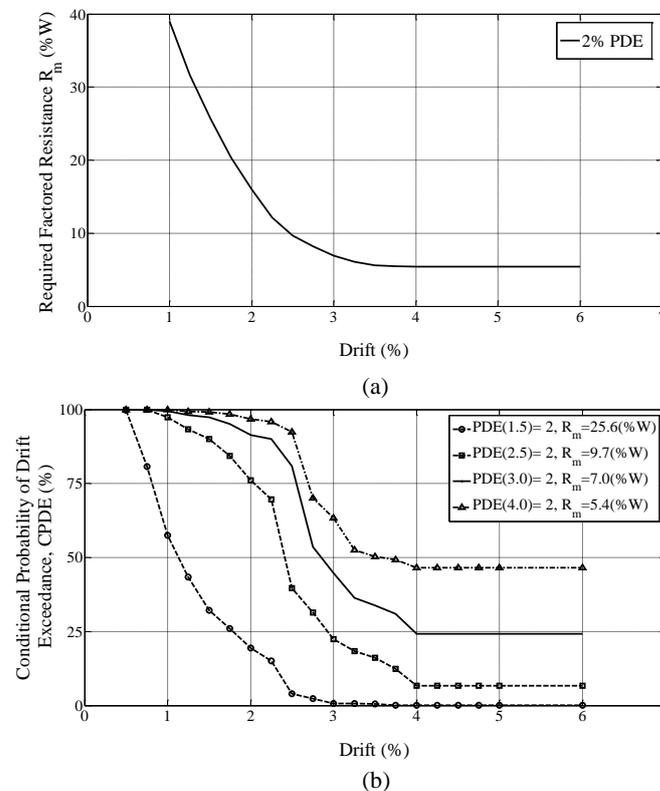
Figure 6.2. (a) R_m values for 2% PDE and (b) CPDE at different R_m values versus drift blocked OSB / plywood shear wall with 3m storey height in Vancouver and Site Class C

Table 6.1. DDL and R_m values for 3m story height located in Vancouver and Site Class C

Material	Prototype Description	Max DDL(%)	R_m (%W)
Wood	Blocked OSB / plywood shearwall	3.0%	7.0
	Unblocked OSB / plywood shearwall	2.75%	8.5
	Gypsum wallboard	2.5%	7.6
	Horizontal boards	3.0%	11.8
Steel	Concentric braced frame (tension only – moderately ductile)	2.5%	16.9
	Concentric braced frame (tension only – limited ductility)	1.5%	28.9
	Concentric braced frame (tension only – conventional construction)	1.0%	36.5
	Concentric braced frame (tension compression – moderately ductile)	2.5%	16.9
	Concentric braced frame (tension / compression – limited ductility)	1.5%	15.4
	Concentric braced frame (tension / compression – conventional construction)	1.0%	21.7
	Eccentric braced frame	2.25%	14.4
	Moment frame (moderately ductile)	2.5%	14.4
Concrete	Ductile Moment Frame	2.75%	6.2
	Partially Ductile Moment Frame	2.5%	9.1
	Non-ductile Moment Frame	1.25%	42.2
	Squat shearwall (shear)	1.0%	16.2
	Shearwall (shear)	1.0%	20.5
	Moderately ductile shearwall (flexure)	1.0%	19.1
	Shearwall (flexure) – conventional construction	0.75%	23.4
Concrete Masonry	Wall sliding at base	1.25%	10.7
	Unreinforced wall	2.5%	32.1
	Reinforced wall	2.25%	17.6
Clay Brick	Brick wall	1.0%	14.9
Rocking	Low aspect ratio (AR): AR ≤ 1.5 for cantilevers and 2 for pier	3.0%	13.2
	Medium aspect ratio (AR): 1.5 < AR ≤ 3 for cantilevers and 2 < AR ≤ 4 for piers.	3.0%	17.6
	High aspect ratio (AR): 3 < AR ≤ 5 for cantilevers and 4 < AR ≤ 7 for piers	3.0%	20.3
Foundation	Sliding foundation	3.0%	16.1

7. LIFE SAFETY RESISTANCE AND DESIGN DRIFT LIMIT SENSITIVITIES

This section examines the sensitivity of required resistance and design drift limits with respect to the storey height, regional hazard, soil type, and the performance objectives. All of the analysis results given in this section are for a blocked OSB / plywood shear wall as an example.

7.1. Storey Height

As the storey height of the LDRS increases, the seismic demand on the system decreases. Fig. 7.1. (a) shows the required resistance for 2% probability of drift exceedance for three different storey heights. The large variation of resistance with storey height is observed in Fig.7.1. (a). The required resistance (R_m) for 1m storey height is 3.5 larger than the 3m one as shown in Fig.7.1. (b). This sensitivity emphasizes the prudence required in the retrofit design of blocks with low storey heights (basements).
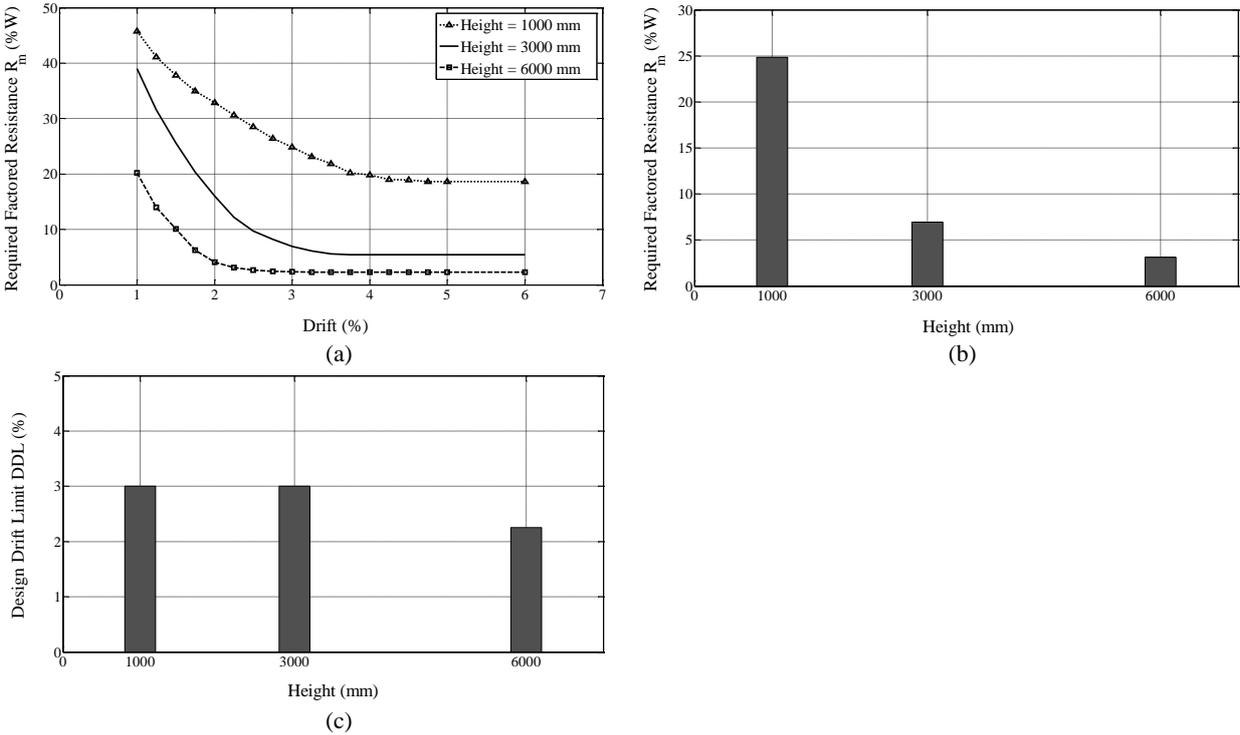
Figure 7.1. (a) R_m versus drift for 2% PDE and (b) design R_m (c) DDL values, Vancouver and Site Class C

However, as illustrated in Fig.7.1., the maximum DDL value for 6m storey height reduces to 2.25%. In fact, the required resistance for the 6m height is small relative to the 1m and 3m heights. Therefore, the LDRS does not have enough capacity to withstand large drift values. This also has been reflected in the flat portion of the curves in Fig. 7.1(a). The 6m height curve becomes flat at lower drift values, implying the lower design drift limit should be set to achieve the life safety objectives.

7.2. Regional Hazard

The life safety risk is set to be 2% in 50 years for all communities in BC; therefore, local seismicity has influence on the maximum DDL and required resistance (R_m). Fig. 7.2. (a) indicates that R_m values for low seismicity regions such as Prince Rupert are noticeably less than Vancouver and Gold River located in high seismicity regions. As observed in Fig. 7.2. (a), the R_m values of 2% PDE for Gold River is less than Vancouver; while, the required design resistance for Gold River is 1.3 times higher than Vancouver as indicated by Fig. 7.2. (b). The fact is that Gold River is located on the west coast of BC with high deterministic subdcution hazard, although the probabilistic crustal and subcrustal hazards in Gold River are less than Vancouver.
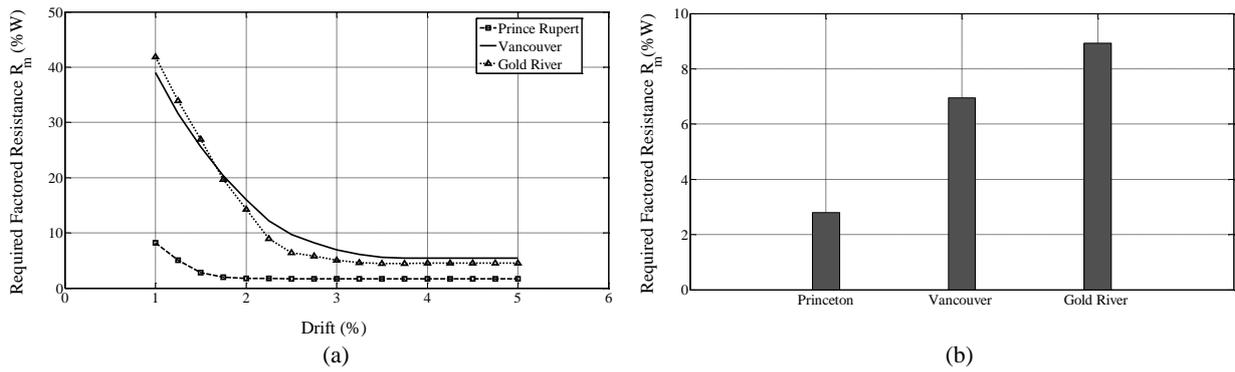
Figure 7.2. (a) R_m versus drift for 2% PDE and (b) design R_m for DDL, 3m storey height and Site Class C

7.3. Soil Conditions

The LDRS performance is also a function of the soil condition. Site Class C and Site Class D/E/F are the two types of soil classes which have been considered in this study. The national building code of Canada, NBCC 2010, uses site Class C (firm ground – very dense sand or soft rock) as the reference site classification and it is adopted as the reference site for the retrofit program. All of the soil classes softer than the firm ground, amplify/de-amplify the level of shaking underside of the building foundations, with reference to Site Class C.
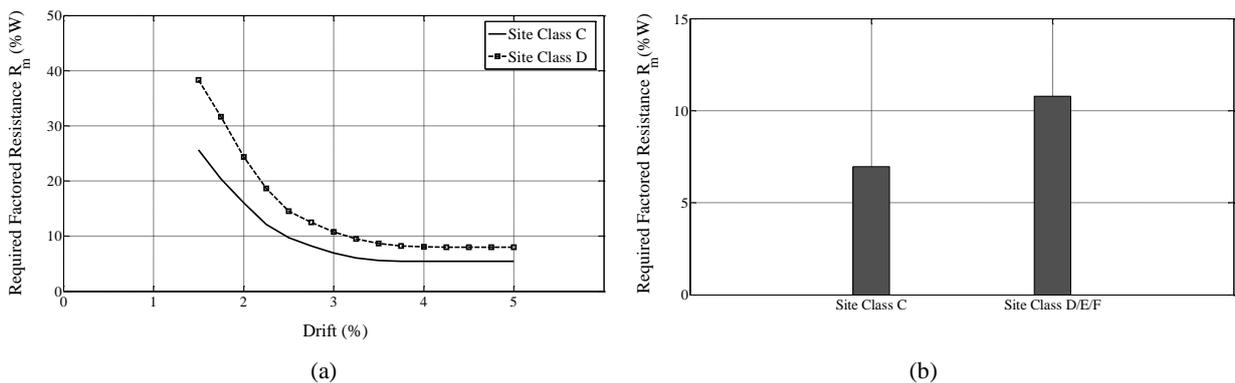
Figure 7.3. (a) R_m versus drift for 2% PDE and (b) design R_m for DDL, 3m storey height in Vancouver

The effects of these soils corresponding to Site Class D are introduced in the analysis through the use of an Equivalent Intensity Factor (EIF) (Pina et al. 2010b). Fig. 7.3(a) specifies that R_m values for Site Class D are substantially greater than Site Class C. The required design R_m for maximum DDL of site Class D/E/F is almost 1.5 times larger than Site Class C as shown in Fig. 7.3(b). Figs. 7.4. and 7.5. show the variation of required resistances for the DDL limit for a blocked OSB / plywood LDRS with 3 m storey height in Southern BC and two types of soil.

7.4. Performance Objective

Currently, life safety is the only performance objective for seismic assessment and retrofit design of school buildings. To achieve this goal, PDE has been set to 2% in 50 years for all structural components. However, in order to reach goals other than life safety, such as collapse prevention, damage mitigation, and immediate occupancy, different PDE values can be used.

The large variation of resistances with PDE values is observed in Fig.7.6. As the risk of drift exceedance increases, the required resistance is reduced. In addition, as the PDE increases, the flat portions of the resistance curves shift to the lower drift values. This implies that at the higher risk

performances, the structure has lower resistance to withstand greater inelastic drifts. Therefore, the maximum DDL should be limited to lower values for high risk performance objectives.
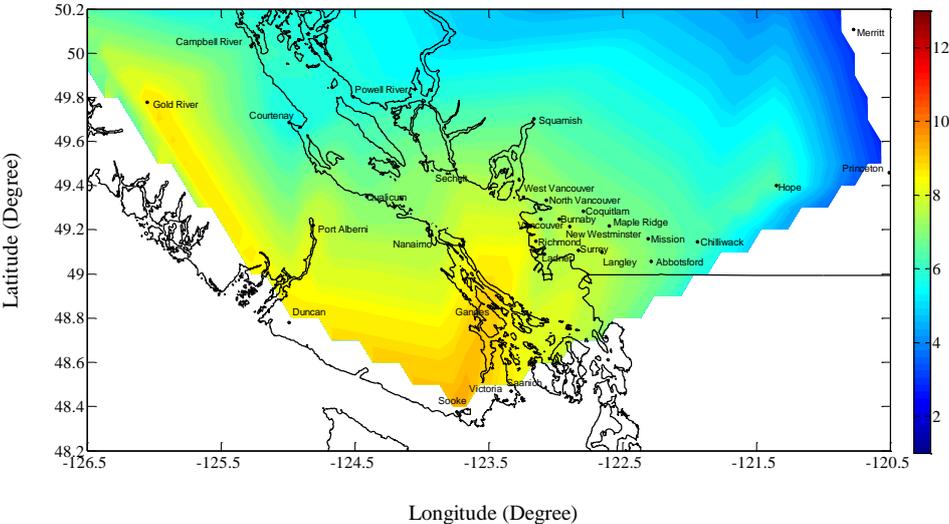
Figure 7.4. Life safety required resistance (%W) map in Southern BC, 3m storey height and Site Class C
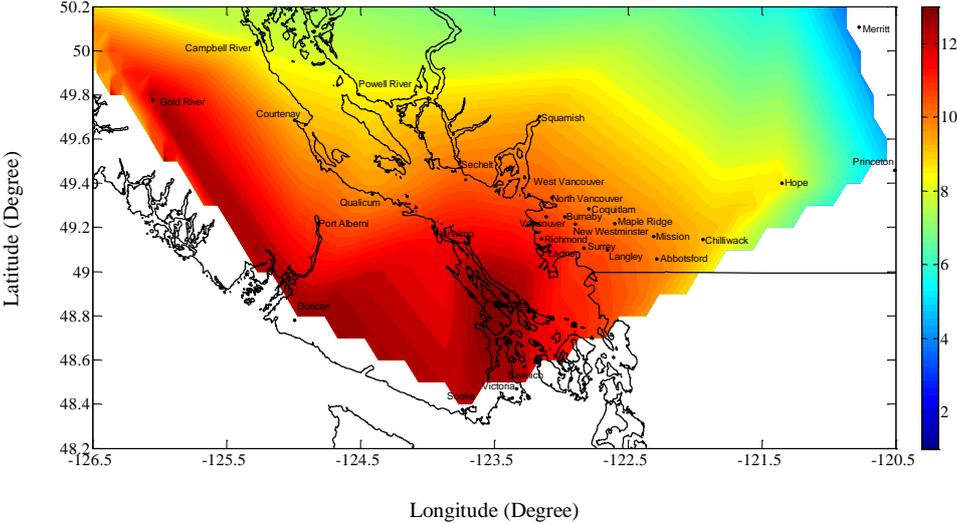
Figure 7.5. Life safety required resistance (%W) map in Southern BC, 3m storey height and Site Class D/E/F
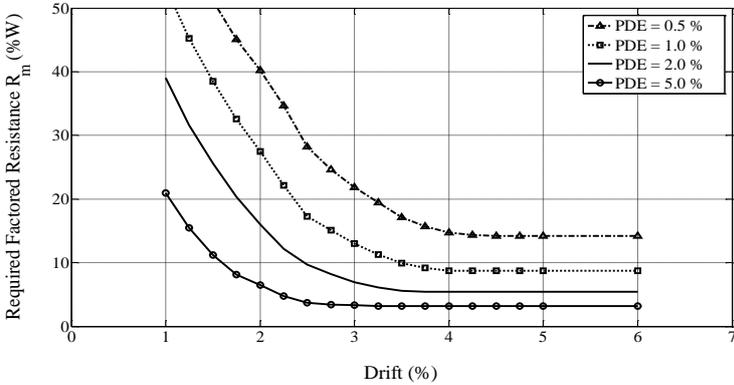
Figure 7.6. R_m versus drift for different PDE values, 3m storey height in Vancouver and Site Class C

8. CONCLUSION

The performance-based methodology described in this paper demonstrates that a particular building (given the type of construction, location and soil type) has a risk (PDE) that is a function of its resistance and its drift limit (DDL). The drift limit, in urn, is a function of storey height for the given building.

Use of a fixed drift limit as given in the building code (drift limit of 2% for all school buildings) can lead to potentially unsafe performance. Consider the case where a frame gymnasium in a Victoria school is seismically upgraded using blocked plywood construction for a maximum risk (PDE) of 2%. The drift limit for the gymnasium is 2% based on the procedure described in this paper. If the same level of resistance for the gymnasium were used to upgrade a wood frame classroom at the same school, the risk for the upgrade of a wood frame classroom building would be 7.3%. Such a retrofit design would be classified as high risk.

This paper presents a methodology to select the life safety design drift limits for a variety of Lateral Deformation Resisting Systems (LDRS). In this study, the drift limits have been calibrated to ensure that all of the prototypes have the same level of risk. Accordingly, a relationship between the design drifts and required resistances has been established for life safety objective. Additionally, different case studies are presented to show how the maximum drift limits vary amongst prototypes, seismic locations, and soil conditions.

REFERENCES

Adams, J., Halchuck, S. (2003). Fourth generation seismic hazard maps of Canada: Values for over 650 Canadian localities intended for the 2005 National Building Code of Canada, *Geological Survey of Canada Open File 4459*, Ottawa, Ontario.

Boore, D.M., Joyner, W.B. and Fumal T.E., (1993, 1994). Estimation of response spectra and peak accelerations from western North American earthquakes: An interim report. Part 1:U.S. Geological Survey Open-File Report 93-509. *Part 2. U.S. Geological Survey Open-File Report 94-127*, Menlo Park, California.

Kramer S. (1996). *Geotechnical Earthquake Engineering 1st edition*, Pearson Education, Singapore.

NBCC (2010). *National Building Code of Canada*, National Research Council of Canada, Ottawa, ON, Canada.

Pina, F., Ventura, C.E., Taylor, G., and Finn, W.D.L. (2010a). Selection of Ground Motions for the Seismic Risk Assessment of Low-Rise Buildings in South-Western British Columbia, Canada. *9th U.S. National and 10th Canadian Conference on Earthquake Engineering*, Toronto, ON, Canada.

Pina, F., Taylor, G., Ventura, C.E. and Finn, W.D.L. (2010b). Site Response Analysis for the Seismic Risk Assessment of Schools in British Columbia, Canada. *9th U.S. National and 10th Canadian Conference on Earthquake Engineering*, Toronto, ON, Canada.

SRG1, (2011). *Seismic Retrofit Guidelines, First edition*. Ministry of Education of British Columbia - Association of Professional Engineers and Geoscientists of British Columbia - The University of British Columbia, Canada.

Youngs, R.R., Chiou, S.-J., Silva, W.J. and Humphrey, J.R. (1997). Strong ground motion relationships for subduction zone earthquakes. *Seismological Research Letters*, **68**, 58-73.

Vamvatsikos, D. and Cornell, C.A. (2001). Incremental Dynamic Analysis. *Earthquake Engineering and Structural Dynamics*, **31:3**, 491-514.