# Performance-based Seismic Assessment and Retrofit of Flexible Diaphragms for School Buildings in British Columbia, Canada

A. Bebamzadeh & C.E. Ventura The University of British Columbia, Canada

**G. Taylor** TBG Seismic Consultant Ltd, Canada



#### SUMMARY

A state-of-the-art performance-based methodology is currently being implemented in a major seismic mitigation program by the Ministry of Education of British Columbia (BC), on the West Coast of Canada. Life safety is the primary performance objective considered in this methodology. The roof and floor diaphragms comprise one of the principal building elements that have a significant influence on the seismic performance of a structure. This paper presents an innovative approach for the assessment and retrofit design of flexible diaphragms. In this approach, the cyclic force-deformation behavior of different wood and steel deck diaphragms is modeled based on reverse cyclic static test data. These models are then subjected to suites of ground motions of varying intensity in performing incremental non-linear dynamic analysis. This analysis is repeated for a range of different resistances and span lengths for each diaphragm model. Diaphragm performance is measured by the probability of shear strain exceedance and the probability of lateral displacement exceedance. These probabilities must conform to specific criteria to prevent excessive damage in the building. This paper focuses on the assessment and retrofit design of flexible diaphragms in one-storey buildings.

Keywords: Flexible Diaphragm, Assessment and Retrofit, Incremental Non-linear Dynamic Analysis

### **1. INTRODUCTION**

The seismic risk assessment and retrofit design of flexible diaphragms has been a persistent problematic topic in current seismic engineering practice in British Columbia. The current building code (National Building Code of Canada 2005 edition) requires that diaphragms and their connections be designed not to yield. The next edition of the building code (to be released in 2015) permits inelastic behaviour of appropriately design flexible diaphragms. This paper presents analytical procedures that highlight the value of utilizing inelastic behaviour in the design of flexible diaphragms and their connections.

### 2. DIAPHRAGM PERFORMANCE-BASED DESIGN METHODOLOGY

The paper presents a performance-based methodology for the assessment and retrofit of flexible diaphragms. The cornerstone of this approach is achieving life safety by reducing the probability of structural collapse to acceptable levels. This methodology is currently being expanded to incorporate performance objectives other than life safety. The principal elements of this non-linear performance-based methodology are discussed in the following sections.

### 2.1. Reliance on Inelastic Deformations of Diaphragm

Traditionally, diaphragms have been designed elastically. This requirement ensures the integrity of diaphragms and the proper distribution of the horizontal forces to the lateral deformation resisting systems such as walls, braced frames, etc. (NBCC 2005). New changes in the upcoming building code

(NBCC 2010) permit the inelastic behavior of appropriately designed and detailed wood and steel deck diaphragms. However, these building codes use a force-based design approach. The required forces are determined on the basis of earthquake intensities at specific hazard levels (2% in a period of 50 years, NBCC 2010).

The methodology described in this paper is based on the inelastic deformation analysis of diaphragms. This approach is significantly different from force-based design. Non-linear dynamic response deformation limits are used to achieve appropriate levels of life safety probability.

## 2.2. Seismic Hazards and Ground Motions

British Columbia has significant hazard contributions from crustal, subcrustal and subduction earthquakes. The design response spectrum in the current National Building Code of Canada (NBCC 2005) is based on a Uniform Hazard Spectrum (UHS) that envelops the spectral acceleration values from all three earthquake types. In this methodology, the total seismic hazard data is de-aggregated by considering each type of source separately in order to reduce the conservatism of the NBCC approach. These results are verified by comparison with open source data provided by the Geological Survey of Canada, GSC (Adams and Halchuk, 2003). To be consistent with the GSC approach, the crustal and subcrustal data are treated probabilistically and the subduction data is treated deterministically. The attenuation relationships given by Young et al. (1997) are used for subcrustal and subduction earthquakes and relationships given by Boore et al. (1993) as crustal earthquakes were used for the analysis. 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. The Cascadia subduction fault source model is used for the deterministic data.

To select the earthquake records, British Columbia is divided into three seismicity regions (low, medium and high seismicity). A set of ten earthquake records has been selected for each hazard type and each medium or high seismicity region. In total, six suites of ten motions are used for the calculation of the total seismic risk in this methodology. Each record is scaled to its corresponding Uniform Hazard Spectra (crustal, subcrustal and subduction) (Pina et al. 2010a).

## 2.3. Non-linear Characteristics of Diaphragm

This paper addresses existing flexible wood and steel diaphragms. The flexible wood diaphragms are divided into three categories: (1) blocked OSB / plywood diaphragms; (2) unblocked OSB / plywood diaphragms; and (3) horizontal board wood diaphragms. The flexible steel deck diaphragms are divided into three categories: (1) ductile diaphragms (mechanical fasteners for both deck-to-frame and side lap connections); (2) moderately ductile diaphragms (welded deck-to-frame fasteners with washers); and (3) low ductility diaphragms (welded deck-to-frame connections and button-punched side lap connections).

The diaphragm is modelled with multiple diaphragm elements that span between the two end walls (Lateral Deformation Resisting Systems – LDRSs). Each diaphragm element is modeled by non-linear shear springs. The cyclic force-deformation behavior of the wood diaphragms is based on the experimental results from the quasi-static in-plane tests at UBC (SRG1 2011). The cyclic force-deformation behavior of the steel deck diaphragms is based on the experimental results from the in-plane monotonic and cyclic testing of steel roof deck diaphragms with nailed and welded connections at UBC (Motamedi and Ventura 2011) and several other sources (Essa et al. 2003, Tremblay et al. 2004, Engleder and Gould 2010). In the analysis model, it is assumed that 100% of the p-delta forces are effective. Figs. 2.1 and 2.2 show a blocked OSB/plywood and ductile steel deck diaphragms and their force-deformation curves of diaphragm shear element for the respectively.



Figure 2.1. (a) Flexible blocked OSB / plywood diaphragm and (b) Force-displacement curve



Figure 2.2. (a) Ductile Steel Deck Diaphragm (b) Force-displacement curve

### 2.4. Incremental Non-linear Dynamic Analysis

Incremental Non-linear Dynamic Analysis (IDA – Vamvatsikos and Cornell, 2001) is the analytical basis of this performance-based methodology. The purpose of the non-linear dynamic analysis of diaphragms is to determine the appropriate capacity that will provide a high probability of an acceptable level of damage based on the non-linear characteristics of the diaphragm. Furthermore, the non-linear dynamic analysis is preformed incrementally to study the sensitivity of dynamic response of diaphragm to increasing ground motions. This approach reduces the probability of premature failure at higher levels of shaking.

In implementing the IDA approach, the diaphragm model is analyzed for each suite of ground motions (Section 2.3) and for each level of shaking in 10% level of shaking increments from the 10% level of shaking to the 250% level of shaking. The 100% level of shaking corresponds to ground motion with a 2% probability of exceedance in 50 years. IDA results are then used to determine the conditional probability and annual rate of damage exceedance in diaphragms as described in Section 3.1. IDA is carried out for a wide range of resistances (varying from 2% to 100% of tributary weight of diaphragm) to define the relationship between resistance and performance of the diaphragms. This process is performed for all wood and steel deck diaphragms, as discussed in Section 2.3 with span length varying from 10 m to 50 m in 10 m increments.

## 3. DIAPHRAGM PERFORMANCE OBJECTIVES

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 life safety performance objective for all principal building elements has a maximum probability of unacceptable damage of 2% over a 50-year period. The life safety design of flexible diaphragms has the following three performance requirements:

- (a) Probability of Design Shear Strain Exceedance (PSE)  $\leq 2\%$  in a period of 50 years. This requirement ensures that the maximum inelastic strain does not exceed the appropriate Design Strain Limit (DSL) within the acceptable level of risk.
- (b) Probability of Lateral Displacement Exceedance (PLDE) ≤ 2% in a period of 50 years. The maximum lateral displacement is determined such that the vertical load bearing systems do not exceed their design drift limits.
- (c) Conditional Probability of Strain Exceedance (CPSE)  $\leq 25\%$  for near-failure conditions for the 100% level of shaking.

The sections 3.1 and 3.2 describe the calculation of DSL, CPSE, PSE, and PLDE in flexible diaphragms.

### 3.1. Probability of Design Shear Strain Exceedance (PSE)

Probability of Shear Strain Exceedance (PSE) for a diaphragm is the percent probability that the diaphragm shear strain 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 as discussed in Section 2.4. The Conditional Probability of Shear Strain Exceedance (CPSE) is calculated using a log-normal fit of the non-linear dynamic analysis results. The annual rate of shear strain exceedance is calculated by multiplying the individual CPSE 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):

$$\lambda(strain > St) = \int CPSE(strain > St|_{Sa}) d\lambda_{Sa}$$
(3.1)

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). CPSE is the conditional probability of shear strain exceedance at given intensity  $S_a$ . The total annual rate of strain exceedance is then calculated by summing up the rates over all sources of hazards (e.g., crustal, subcrustal, and subduction). The PSE is estimated using the temporal Poisson probability model at given time interval T as follows (Pina et al. 2010 b):

$$PSE(strain > St) = 1 - \exp\left(-T\sum_{i=1}^{n}\lambda_i\right)$$
(3.2)

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 PSE. The subduction data is treated deterministically by checking the CPSE for the 100% level of shaking for the subduction ground motion at high levels of shear strain (close to failure).



Figure 3.1. PSE versus R<sub>m</sub> for flexible blocked OSB / plywood diaphragm with 20m span length in Vancouver and firm ground



Figure 3.2. PSE versus R<sub>m</sub> for ductile steel deck diaphragm with 20m span length in Vancouver and firm ground

The above procedure is done incrementally for a wide range of resistances so that the relationship between PSE and resistance can be determined. Figs. 3.1. and 3.2. show the probability of shear strain exceedance (PSE) at different strain levels for a blocked OSB / plywood diaphragm and a ductile steel deck diaphragm, respectively. The span length of diaphragm is 20m and it is located in Vancouver with firm ground (very dense sand or soft rock). The diaphragm resistance  $R_m$  is given as a percentage of the tributary weight ( $W_d$ ) for the diaphragm including the diaphragm's self-weight, 25% snow load on the roof and the weight of a portion of the walls and parapets laterally supported by the diaphragm. For a given hazard level, the lower the strain limit, the higher the resistance required. Fig. 3.1. illustrates that the probability of exceedance of 3% and 4% inelastic strain merge together at most of resistance and hazard levels. This suggests that strains higher than 4% are close to failure of the diaphragm. Fig. 3.2. shows that the PSE of 2% and 2.5% are almost identical. Therefore, 2% shear strain is the assumed failure strain for ductile steel deck at all level of hazards and resistances.

#### 3.2. Design Shear Strain Limit (DSL)

The Design Shear Strain Limit (DSL) is the maximum shear strain that can be used in retrofit design or seismic risk assessment of flexible diaphragms. Qualitatively, this strain limit represents a damage level in a diaphragm that is "life-safe". The DSL was also checked to ensure it met the CPSE requirement ( $\leq 25\%$  at high strain rates).



**Figure 3.3.** Required resistance (R<sub>m</sub>) versus strain limit for flexible blocked OSB / plywood Diaphragm and ductile steel deck diaphragm, PSE = 2%, span =20m, Vancouver and firm ground

Fig. 3.3. shows the minimum required resistance ( $R_m$ ) for the life safety retrofit design for a maximum risk of 2% in a period of 50 years. The design resistances are presented for two types of flexible diaphragms (blocked OSB / plywood diaphragm and ductile steel deck diaphragm located in Vancouver on firm ground). The flat part of the curves denotes the failure strains for a hazard level of 2% in 50 years. The design strain of DSL=3% with resistance of  $R_m = 6.7\%$  ( $W_d$ ) is selected for the blocked OSB / plywood diaphragm. At this resistance level, the conditional probability of exceeding (CPSE) collapse strains (strain greater than 4%) is 24%. The DSL value is reduced to 1.5% for the ductile steel deck with  $R_m = 23.4\%$  ( $W_d$ ) and CPSE of 23%. Table 3.1. summarizes the DSL and the required  $R_m$  resistance values for different types of flexible diaphragms for a 20 m span length, Vancouver seismicity and firm ground.

Diaphragm Type	Design Shear Strain Limit (DSL)	Required Resistance (R <sub>m</sub> )
Blocked OSB / plywood	3.0%	6.7 %W <sub>d</sub>
Unblocked OSB / plywood	2.75%	8.6 %W <sub>d</sub>
Horizontal boards	3.5%	7.1 %W <sub>d</sub>
Steel deck (ductile)	1.5%	23.4 %W <sub>d</sub>
Steel deck (moderately ductile)	1.0%	35.0 %W <sub>d</sub>
Steel deck (low ductility)	0.5%	50.0 %W <sub>d</sub>

Table 3.1. DSL and  $R_m$  values strain limit for PSE = 2%, span = 20m, Vancouver and firm ground

#### 4. DIAPHRAGM PERFORMANCE SENSITIVITIES

This section presents a sensitivity analysis for wood and steel deck diaphragms to illustrate their seismic performance characteristics. The span length, mid-span lateral displacement limit, soil type and regional hazard variations are studied. All analysis results given in this section are the life safety retrofit design of 2% in period of 50 years.

#### 4.1. Span Length

Figs. 4.1.(a) and 4.1.(b) illustrate the required resistances for the life safety design of different types of wood and steel diaphragms for a range in span lengths. Fig. 4.1.(a) shows that the resistance requirements for wood diaphragms reduce with increasing span length. However, the resistance requirements for steel deck diaphragms as shown in Fig. 4.1.(b) do not change appreciably with increasing with increasing span length.



(a) Blocked OSB / plywood Diaphragm

(b) Ductile Steel Deck Diaphragm



### 4.2. Lateral Displacement Limit

Lateral displacement at diaphragm mid-span influences the retrofit design in addition to the design strain limit (DSL). The probability of Lateral Displacement Exceedance (PLDE) for a flexible diaphragm is the percent probability that the lateral movement at the mid-span of the diaphragm relative to the lateral movement of the end wall supports (LDRSs) will exceed the specified out-of-plane displacement. PLDE is calculated in a similar manner to PSE for a range of out-of-plane displacements. The required resistance value,  $R_m$ , for the retrofit design of a diaphragm is the greater of (a) required resistance for 2% PSE corresponding to selected DSL, and (b) required resistance for 2% PLDE corresponding to the desired lateral displacement limit (LDL).

Figs. 4.2.(a) and 4.2.(b) give the minimum resistance requirements for 100 mm and 200 mm out-ofplane displacement limits for a PLDE value = 2% for different span lengths of blocked OSB / plywood and ductile steel deck diaphragm, respectively. Fig. 4.2.(a) shows that the resistance requirements for out-of-plane displacement limits only govern for small displacement limits (100 mm) for the blocked OSB / plywood diaphragm. Fig. 4.2.(b) illustrates that the out-of-plane displacement limit has even less influence on the minimum resistance requirement for a ductile steel deck diaphragm.



(a) Blocked OSB / plywood Diaphragm

Figure 4.2. Out-of-Plane displacement limit sensitivity for (a) flexible blocked OSB / plywood diaphragm and (b) ductile steel deck diaphragm, Vancouver and firm ground

### 4.3. Regional Hazards and Soil Type

Figs. 4.3. and 4.4. show the variation of required resistances for the shear strain limit for a blocked OSB / plywood diaphragm with 20 m span length in Southern BC and two types of soil. As anticipated, communities in high seismicity regions such as Duncan, Gold River, Ganges, Sooke, and Victoria require higher resistances compared to low seismicity regions such as Ashcroft, Kitimat, Merritt, and Princeton. Fig. 4.3. also illustrates that the resistances required for design of the diaphragm in stiff and soft soil (Site Class D /E /F) is approximately 1.3 to 1.9 times higher than in firm ground (Site Class C). This difference is more pronounced in high seismicity regions.



Longitude (Degree)

Figure 4.3. Life safety required resistance (%W) map in Southern BC for flexible blocked OSB / plywood diaphragm and span = 20 m and Site Class C



Longitude (Degree)

**Figure 4.5.** Life safety required resistance (%W) map in Southern BC for flexible blocked OSB / plywood diaphragm and span = 20 m and Site Class D/E/F

#### **5. CONCLUSION**

This paper presents a rational approach where analytical procedures model inelastic behaviour of flexible diaphragms in both the risk assessment and retrofit design procedures. The analytical results presented in this paper highlight the value of adopting a performance-based approach that incorporates inelastic behaviour to generate safe and cost-effective solutions.

The findings presented in this paper are best highlighted by an example of a retrofit design for an existing concrete masonry school gymnasium with a 30 m span tongue-and-groove wood roof. The gymnasium is located in Vancouver and is founded on Site Class C soils. The diaphragm retrofit comprises the installation of new blocked plywood at the underside of the existing wood roof.

It is not practical to upgrade the roof diaphragm so that both the blocked plywood and its connections behave elastically (roof diaphragms substantially stronger than reinforced masonry end walls). Figure 5(a) illustrates a minimum required factored resistance of approximately 6%Ws for the new blocked plywood diaphragm (allowing 200 mm maximum mid-span movement). This contrasts with the concrete masonry end walls that need to be upgraded to a minimum factored resistance of 18%Ws. Utilizing inelastic behaviour in the new wood diaphragm yields a safe, practical, cost-effective solution. Utilizing inelastic behaviour in the diaphragms also reduces the demand on the masonry end walls (maximum inertia forces from diaphragm limited by diaphragm overstrength).

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