NONLINEAR PERFORMANCE BASED SEISMIC ASSESSMENT FOR LOW-RISE BUILDINGS

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INTRODUCTION

In May 2004 the Ministry of Education of British Columbia (Ministry) announced a $1.5 billion seismic mitigation program for public K-12 school buildings. The goal of this program is to achieve the seismic mitigation of the most hazardous school buildings within a reasonable length of time. The development of innovative, cost effective assessment and mitigation design is being prepared under contract with the Ministry through a cooperative agreement between the Association of Professional Engineers and Geoscientists of British Columbia (APEGBC) and the Department of Civil Engineering of the University of British Columbia (UBC). The unique approach to develop a new cost effective seismic mitigation program within a reasonable period of time is a real challenge. The basic concepts of the approach and preliminary data were generated and released to the design community in November 2006. Internally this document is referred to as the Bridging Guidelines. However, the official title of the copyrighted document is Performance-Based Seismic Assessments and Retrofit Design Concepts – Second Edition [1]. In March 2007 the Ministry announced the availability of $254 million in funding for capital construction. The Ministry required that the funds for this program be utilized for the assessment and structural seismic retrofit designs of the school buildings. Non-structural seismic mitigation was in a separate program. APEGBC and UBC are continuing the program development. The concepts and ideas given herein are a combination of those released in November 2006 and more recent developments. This paper is intended to be consistent with the new, significantly improved version of the program. However, as new concepts continue to be developed this paper cannot assure the reader that what is presented will be the final approved program.

The building configuration limitations imposed during this development are (1) 3-story or shorter school buildings, (2) the primary lateral deformation resisting systems (LDRS) provide acceptable life safety risk, (3) heavy partition walls have adequate restraint, (4) non-seismic elements have adequate drift compatibility, and (5) the retrofitted building must have an adequate load path from foundation to roof. This concept utilizes a criteria based on maximum displacement limits for each building LDRS to preclude the risk of collapse. It applies a nonlinear hysteretic single-degree-of-freedom analytical model subjected to ground motions selected to representative of those expected at each site. These analyses also provide the necessary information as to the desired level of seismic upgrade necessary to achieve noncollapse for each specific building being evaluated and provide a process for combining the LDRS properties of the original building and the retrofit LDRS properties.

MINIMUM PERFORMANCE REQUIREMENTS

There are three basic performance requirements. They are (1) the seismic risk assessment or the retrofit must meet or exceed the minimum requirements for (a) the LDRS, (b) vertical load-bearing supports, (c) walls subject to out-of-plane collapse, (d) diaphragms and (e) connections; (2) all buildings not meeting these minimums will require retrofitting; and (3) all building elements that pose an unacceptable risk are assigned a priority retrofit ranking.

Although this paper will focus on the development of the minimum required lateral resistance for the assessment of existing building LDRS and the evaluation of the retrofitted LDRS combinations, the other elements cannot be neglected in any assessment or retrofit design.

Next a brief summary of the seismic environment used for these assessments is provided. The results of the manifold nonlinear response history analyses are assembled into a database that the designers use to assist in their assessments and evaluations. A brief description of the required input data and expected output is provided.

LATERAL DEFORMATION RESISTING SYSTEMS (LDRS)

The key concept of this development is that building deformations are critical to estimating the risk of collapse. Although lateral strength plays a role in the nonlinear dynamic response of the building, it is the deformation that sets the decision limits. This is very different than the tradition code design approach where pseudo-static lateral forces are used to select member sizes.

The range of structural systems included in this methodology can be seen in Table 1. These include most of the structural systems existing in existing school buildings and those that could be utilized for seismic upgrading. The Bridging Guidelines have preliminary drift limits for many of these systems, but those values are not included in Table 1 because of their preliminary nature. The current development of the methodology provides significant improvement over the Bridging Guidelines, so it is considered inappropriate to provide those values in this paper. The intent is to establish these drift limits by December of 2009. For illustration of the concept and methodology, the probable woodframe drift limit values are included in Table 1.

WOOD FRAME PROTOTYPE W-2

To illustrate the process, only the unblocked OSB or plywood shear wall will be discussed. The hysteretic data were developed from the CUREE-Caltech Wood-frame project and the EQ-99 project at UBC [2]. In the CUREE project many numerical models were developed, but they are all based on the models proposed by Stewart [3] and Rahnama and Krawinkler [4]. The EQ-99 project worked on an adaptation of the Stewart's model that was already implemented on the CASHEW/SAWS computer program [5]. This model shown in Figure 1 essentially captured the three most characteristic effects during the cyclic loading of these wood systems: pinching behaviour, strength and stiffness degradation. The primary program used to generate the

WOOD FRAME	SHEATHING OR FINISH	DRIFT LIMIT
W-1	Blocked OSB or plywood shear wall	4%
W-2	Unblocked OSB, plywood, gypsum or horizontal boards	4%
STEEL FRAME		
S-1	Concentric braced – tension only	
S-2	Concentric braced – tension/compression	
S-3	Eccentrically braced	
S-4	Ductile steel moment resisting	
CONCRETE FRAME		
C-1	Moderately ductile squat shear wall <=2.5	
C-2	Moderately ductile shear wall	
C-3	Conventional squat shear wall <=2.5	
C-4	Conventional shear walls	
C-5	Ductile moment resisting	
C-6	Moderately ductile moment resisting	
C-7	Conventional moment resisting	
CONCRETE MASONRY		
M-1	Base of wall sliding	4%
M-2	Bed joint sliding	2%
M-3	Reinforced masonry in flexure	3%
CLAY BRICK MASONRY		
B-1	Bed joint sliding	
ROCKING ELEMENTS		
R-1	Low aspect ratio = maximum of 1 for cantilevers and 2 for piers	
R-2	Medium aspect ratio = maximum of 2.5 for cantilevers and 5 for piers	
R-3	High aspect ratio = maximum of 4 for cantilevers and 8 for piers	

TABLE 1 - LATERAL FACTORED RESISTANCE OF EXISTING AND RETROFIT MATERIALS

assessment data is CANNY [6]. Several other programs were used for the same earthquake records to validate the accuracy of the CANNY program. Among the programs used were Quakesoft, PERFORM-3D [7], and the data from FEMA-440 [8].

In the Bridging Guidelines, a slight modification of the final models proposed in the CUREE and EQ-99 projects was adopted for the dynamic analyses, based on engineering judgement and to simplify calculations. The analytical computer program, CANNY, allows for pinching effect and both strength and stiffness degradation. A three-parameter model that follows a four-branch backbone curve with the option of two negative slopes at the end is used. An illustration of the hysteretic rule is shown in Figure 2. The hysteretic model follows the backbone curve until it reaches the deformation Δ_1 at point D. The unloading curve has the same slope as the initial loading curve until it reaches zero force at point E. Point G has the negative values of point D and $\Delta 2$ equals one half of Δ_1. F_1 equals one half of the force at point G. The reloading segment A targets a force equal to F_1 until it intercepts the reloading segment B which is at the initial slope through point G. At point G the hysteretic model follows the backbone curve until a reversal of direction is achieved. Additional cycles follow this same pattern. In this model, the beta value (gap on strength) is set to zero and the pinching effect is captured by the model proposed by Stewart [3].

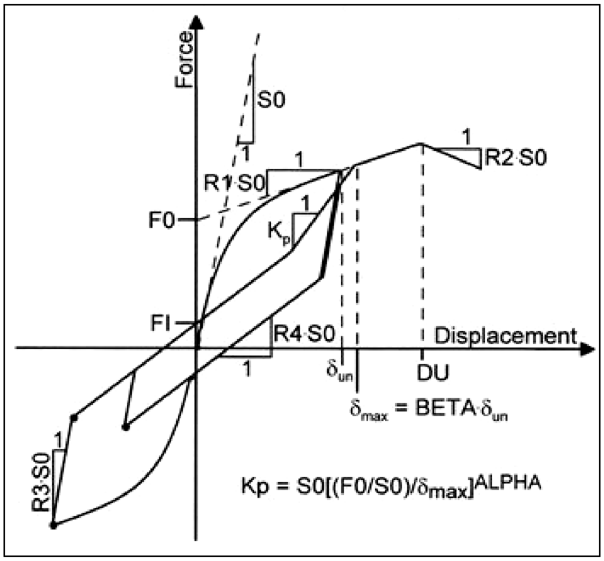


FIGURE 1 – CASHEWS/SAWS HYSTERESIS RULES FOR WOOD SYSTEMS (FOLZ AND FILIATRAULT, 2002)

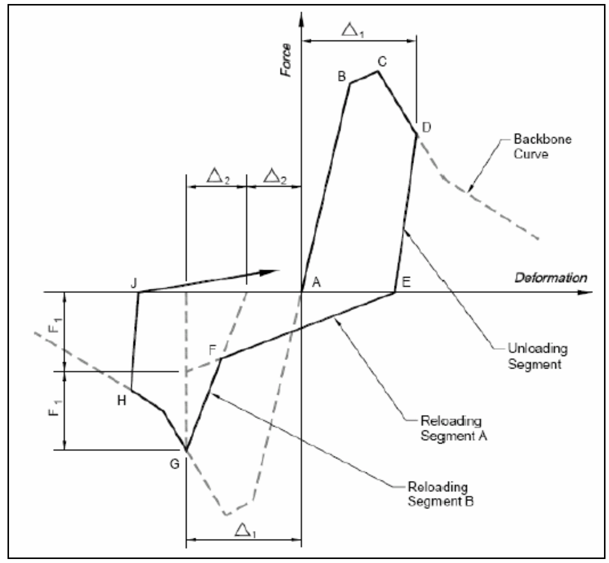


FIGURE 2 – HYSTERESIS RULE FOR W-2 PROTOTYPE

The variable deformation pattern shown in Figure 3 is used to illustrate the hysteretic behavior. The hysteretic response to the entire cyclic deformation pattern of Figure 3 is shown in Figure 4. In the first cycles, the reloading always points towards the first yielding point. After yielding, pinching effect will take effect in the reloading part of the cycle.

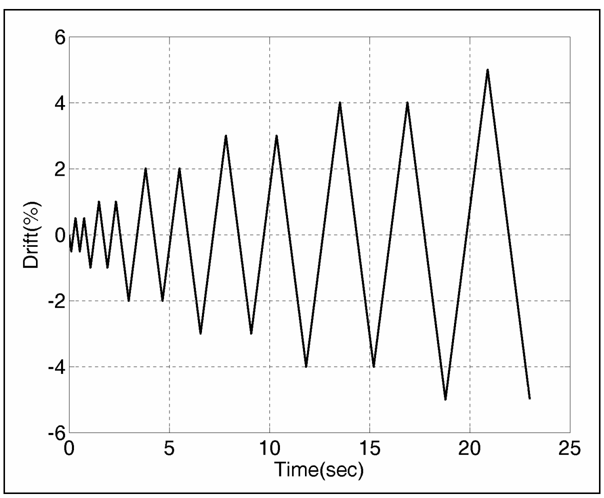


FIGURE 3 – IMPOSED SHEAR DRIFT FOR W-2

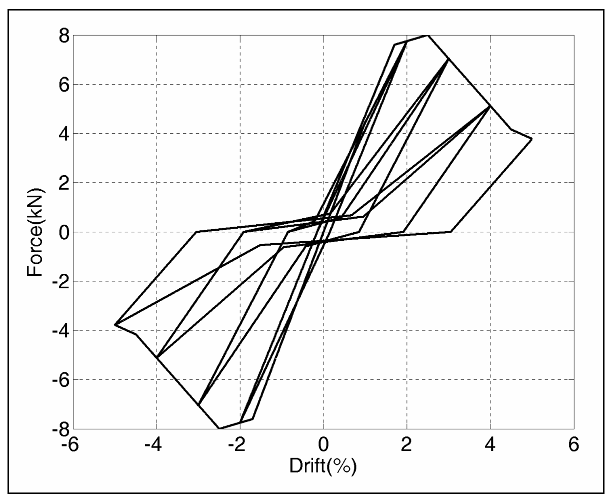
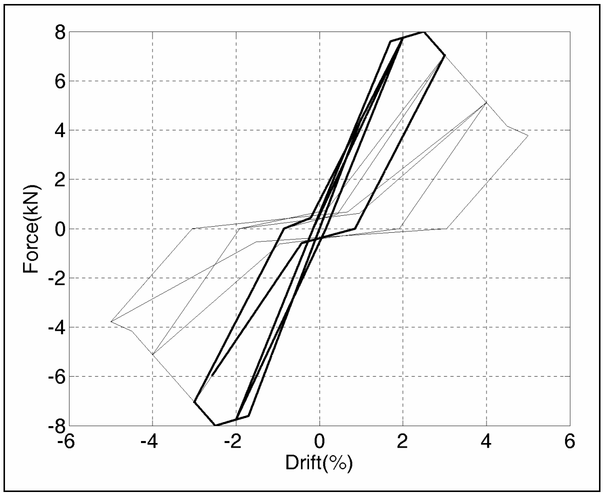


FIGURE 4 – CANNY HYSTERETIC RESPONSE FOR W-2

The important details of this hysteretic pattern is more clearly illustrated in Figures 5 and 6 where the hysteretic results are shown for drifts varying from zero to 3% and from zero to 5%, respectively. It should be noted that these hysteretic properties include both cyclic and in-cycle degradation. Where the cyclic degradation is defined by the decrease in strength capacity from one cycle to the next, and in-cycle degradation is defined as the decrease in strength capacity within a cycle. That is, the negative slope movement of the hysteretic response.



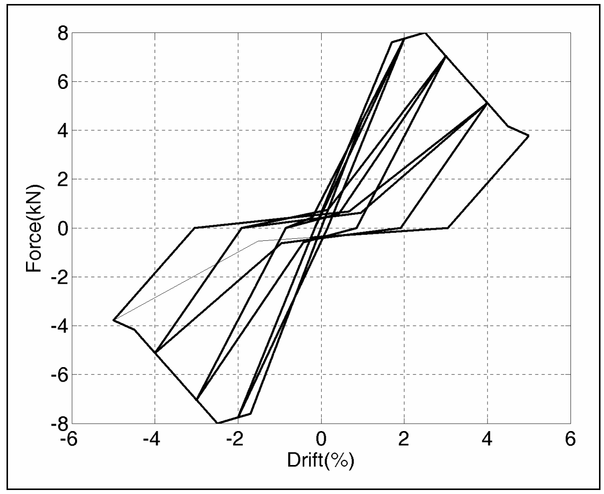


FIGURE 5 – CYCLIC RESPONSE FROM 0 TO 3% DRIFT

FIGURE 6 – CYCLIC RESPONSE FROM 0 TO 5% DRIFT

To further illustrate this point the hysteretic behavior of the W-2 model in response to three different types of ground motion are shown in Figure 7. It can be seen that for this example, the subcrustal ground motion results in a residual lateral displacement of the system. However, the subduction ground motion was large enough to create structural instability. That is, a drift much larger than the acceptable drift limit of 4% occurs. The numerical values for the basic W-2 model are given in Figure 8. These strength and hysteretic properties are limited to those controlled by standard practice. Irregular designs not within the listed specifications (e.g., very large or very close nail spacing) may not qualify to use these prototypes. These variations will influence the yield drift and length of the yield plateau. These specifications and properties are summarized in Table 2. The lateral capacity, P, is calculated from the materials and geometries of the existing school building, or the properties of the upgraded system as appropriate.

	UNBLOCKED OSB (11 mm)	**UNBLOCKED PLYWOOD (9.5 mm)**	**GYPSUM WALLBOARD (12.7 mm)**	**HORIZONTAL BOARDS (19 mm x 184 mm)**
Fastener	51 mm nails	51 mm nails	32 mm ring nails	64 mm nails
Spacing - edge	152 mm	152 mm	203 mm	2 nails / board
Spacing - interior	305 mm	305 mm	35 mm screws at 406 mm	
Factored resistance for assessment	3.5 kN/m	3.5 kN/m	1.1 kN/m/side	0.6 kN/m/side
Factored resistance for retrofit	3.5 kN/m	3.5 kN/m	1.1 kN/m/side	Not Permitted

TABLE 2 – SPECIFICATIONS AND PROPERTIES FOR PROTOTYPE W-2 UNBLOCKED OSB / PLYWOOD

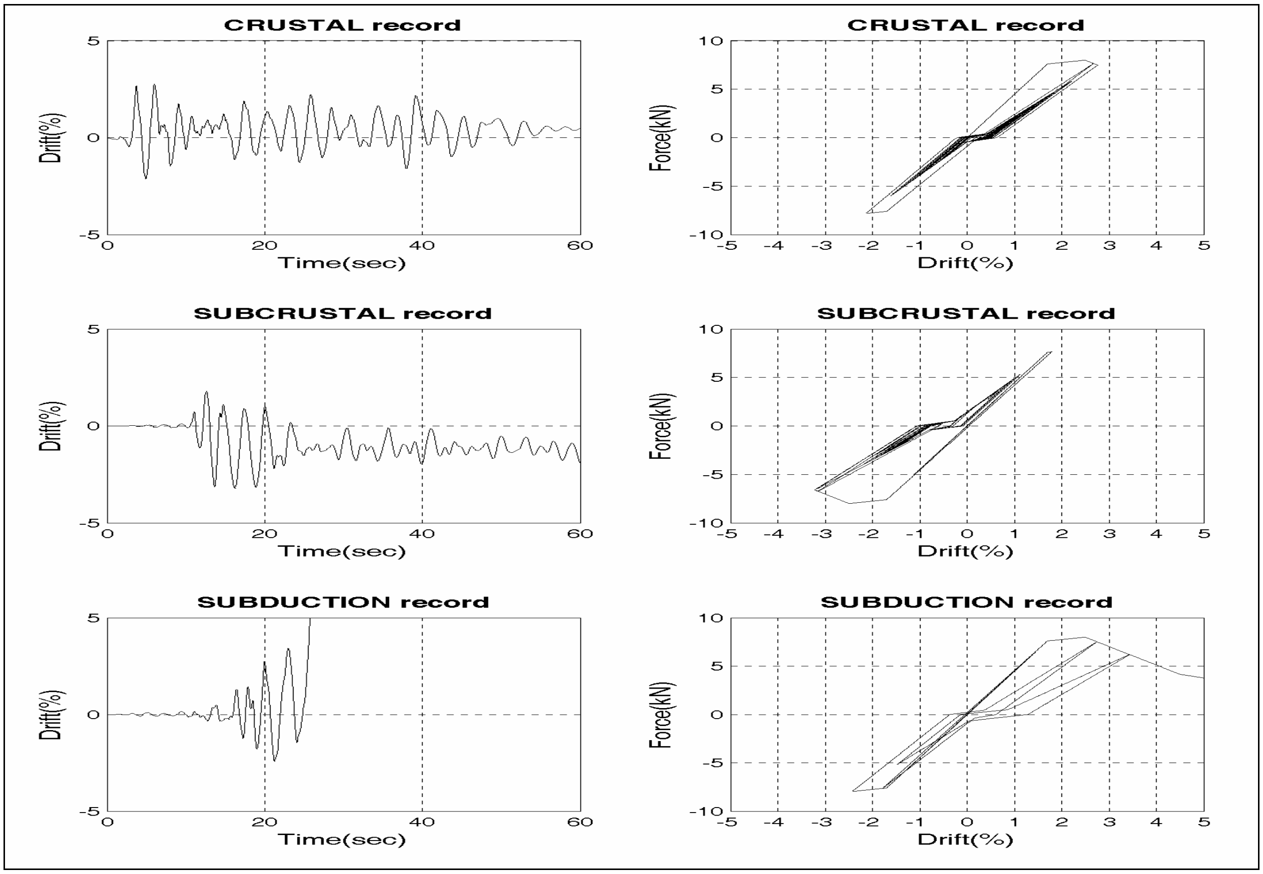


FIGURE 7 – FIRST FLOOR DISPLACEMENT RESPONSE AND HYSTERETIC CURVES FOR CRUSTAL, SUBCRUSTAL AND SUBDUCTION EARTHQUAKE RECORDS

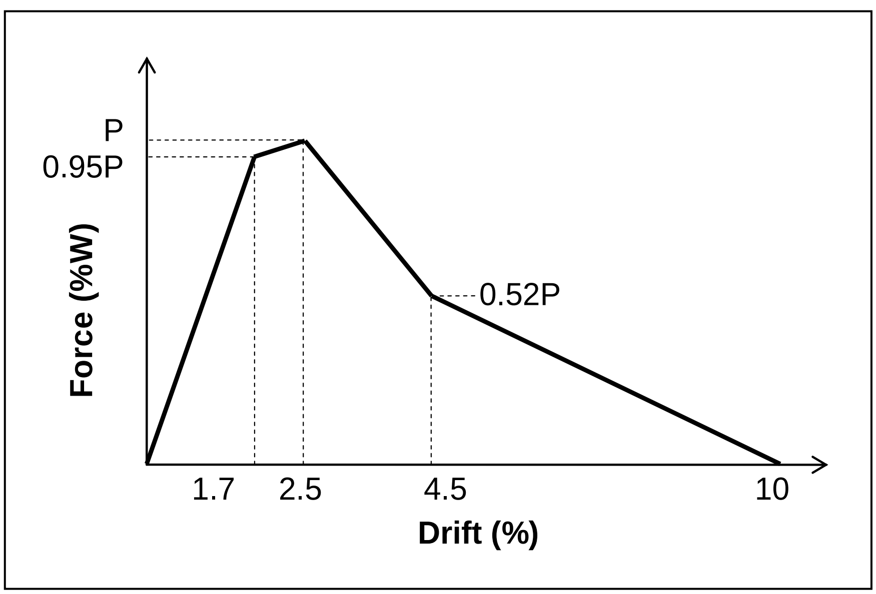


FIGURE 8 – BACKBONE CURVE FOR PROTOTYPE W-2

GROUND MOTION RECORDS

British Columbia is a large geographic area with multiple sources of expected ground motions. It was decided to incorporate crustal, subcrustal and subduction earthquake sources for all sites. The selection of earthquakes and records is based on ranges of magnitudes and hypocentral distances that contribute mostly to the hazard of Vancouver (crustal and subcrustal) and Victoria (subduction). In the case of Vancouver the Geological Survey of Canada provided the deaggregation of a 2% in 50-year earthquake level hazard at the same period values defined in the National Building Code of Canada [9] and for the two seismic sources (crustal and subcrustal). Two probabilistic models are used by NBCC. For crustal they used a regional model and for subcrustal they used a historical model. For the subduction records, the largest subduction events that have been recorded within an equivalent distance of Victoria to the subduction zone of Cascadia were selected.

Vancouver and Victoria are considered as moderate-to-high seismicity cities. Another set of records will be also selected using the same process for a city with low seismicity to minimize the impact of the methodology on places with low-to-none seismic activity. Thus, six suites of ten motions for low and high seismicity will be used for the calculation of the total seismic risk in this project.

The record scaling procedure was based on velocity spectral matching of the motion records to the target Uniform Hazard Spectra within the one to two second period range. Each record was scaled to its corresponding source Uniform Hazard Spectra (crustal, subcrustal and subduction). The selection criteria of the scaled records were: (1) select only records above 70% of the target for the scaling period range while giving preference to those that fall above 80% or 90% of the target, (2) the average of the final suite must be above the 90% of the target within the whole period matching range, (3) select motions recorded from different earthquakes rather records from the same earthquake, and (4) select one record per station.

ANALYTICAL DATABASE

A basic assumption of the methodology presumes that the controlling lateral resistance occurs in the first story of the school building. If this is not true, the results need to be adjusted to account for the appropriate weakest story. The analytical model assumes a two-story building while its results are utilized for one through three-story school buildings. For a given site, assumed to be a soil class D site, the selected input ground accelerograms are linearly scaled from 40% to 250% of its base spectral scaled values described above.

The lateral mass of the system is assumed to vary from 2% of its total gravity weight to 50% of its gravity weight. The corresponding calculated response drifts are then assembled and assessed through the Incremental Probabilistic Nonlinear Dynamic Analysis described in the next section. From these results it is determined what lateral capacity is needed to keep the probability of exceeding the selected drift limit to less than 2% in 50-years. This determines the acceptable minimum LDRSs lateral capacity.

The Guidelines incorporate this large database of analytical results for the full range of LDRSs subjected to a range of parameters including seismicity, soil type, drift limit and storey height. The engineer using the Guidelines does not need to undertake any non-linear dynamic response analysis. This analysis will have been completed and presented for use in the form of easily useable database software. All database analyses will have been independently validated and peer-reviewed. The software is called the "Seismic Performance Calculator."

INCREMENTAL PROBABILISTIC NONLINEAR DYNAMIC ANALYSIS

The details of this methodology are presented in a companion paper at this conference entitled. Suffice it to say here that the calculated building responses to ground motion intensities are separated into 22 ranges from greater than 30% to more than 250% in 10% increments. The probability of occurrence for each intensity range is calculated using the annual probability of exceedance. The variation of the annual probability of exceedance for a hazard spectrum at different intensities is represented by the variation in annual probability of exceedance for the one-second spectral values for the hazard spectrum at the different intensities.

COMBINING DIFFERENT LDRS SYSTEMS (TOOLBOX)

One of the features of the guidelines methodology is the ability to combine the contributions from different LDRSs in performing either a risk assessment of a building or refining the retrofit design. This method of combining different LDRSs is referred to as the Toolbox.

The principal technical consideration is the need for all included LDRSs to generate their lateral resistance in a drift-compatible manner. This requirement is implemented by the selection of the Governing Drift Limit. The Governing Drift Limit is the lowest drift limit of the participating LDRSs. Each LDRS can only generate resistance up to this Governing Drift Limit. Depending on the range of LDRSs drift limits being considered for combination, this requirement may result in the underutilization of certain LDRSs.   However, the combination may still result in a cost-effective solution by including all LDRS materials, existing and new, in the seismic performance of the building.

Conceptually, the Toolbox uses a simplified and modestly conservative approach to determine the contribution of each LDRS to the total lateral resistance. Each LDRS contribution is calculated as a percentage of the total inertia mass on the basis of its lateral resistance capacity at the Governing Drift Limit. These percentage contributions for the LDRSs are then totaled to determine the overall building response capacity. If the total capacity equals or exceeds 100% of the required capacity, the risk assessment or the retrofit design is deemed to be satisfactory. Conversely, if the total capacity is less than 100% of the required capacity, the building's risk assessment or retrofit design is deemed to be deficient.

The Toolbox permits a retrofit design to be fine-tuned. The composition of the participating LDRSs for a retrofit design can be strategically considered. An existing LDRS with a low drift limit and a small contribution to the overall building performance can, in many cases, be isolated from the overall lateral system. This isolation of the more vulnerable LDRS from the desired LDRSs will result in a higher Governing Drift Limit. The higher the Governing Drift Limit the greater the contributions from each of the remaining LDRSs. This iterative process can quickly determine the most effective combination of participating LDRSs for the mitigation design.

RETROFIT DESIGN

Retrofit design is a two-step process. The first step is to undertake a risk assessment to determine if the building needs to be upgraded. The second step is to quantify the retrofit requirements for new LDRSs to be added to a building that has a deficient risk assessment result.

The Guidelines database permits the engineer to quickly determine the minimum lateral resistance required for any LDRS to laterally support its apportioned inertia mass with the peak drift not to exceed a specified Governing Drift Limit. As noted above, the Toolbox permits the engineer to consider all potential LDRSs in planning the retrofit design of a seismically deficient building.

The Toolbox permits the engineer to optimize the retrofit design to create the most cost effective combination of existing LDRSs and new material LDRSs. As noted above, not all existing LDRSs need be considered, only those that make an effective contribution to overall seismic performance. The engineer can quickly determine the minimum contributions required for a range of new LDRS options to determine the most cost-effective solution. The Toolbox is a highly efficient design optimizer.

CONCLUSION

The concepts and methodology briefly discussed herein provide significant advances in the development of seismic hazard mitigation of existing buildings. These advances include:

- Utilization of lateral drift limits as the primary assessment criteria of acceptable performance,
- Utilization of three earthquake sources, crustal, subcrustal and subduction events, for computation of expected building responses at a site,
- Incremental probabilistic combination of the nonlinear analyses results for these three events,
- Realistic nonlinear hysteretic modeling of existing structural framing systems including cyclic and in-cycle degradation is the state-of-the-art,
- Realistic nonlinear hysteretic modeling of proposed seismic upgrade concepts that include cyclic and in-cycle degradation as appropriate,
- A methodology to combine any number of lateral structural systems, existing and new, in a logical conservative manner for a specific building, and
- Publication of a software program that accesses the manifold nonlinear dynamic results created by the techniques mentioned above into a designer friendly interaction solutions tool.

The goal is to have this tool available to the structural designers of British Columbia for their use in the assessment and seismic upgrade designs for BC school buildings by early 2010.

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