Conventional steel constructions for the performance-based earthquake retrofit of low-rise school buildings

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ABSTRACT: An on-going $1.5 billion project for the retrofit of schools in the province of British Columbia, Canada, considers both the seismic risk assessment and retrofit of more than 700 low-rise buildings for the next ten years. Conventional steel constructions such as eccentrically brace frames and moderately ductile steel moment-resistance frames have been considered as feasible retrofit solutions. Guidelines have been published including a procedure for the determination of minimum required factored resistance values of a structural system. These resistance values are associated to an instability drift limit defined by a collapse performance objective under a 2500-year return period earthquake. Results presented in the guidelines are obtained from several nonlinear dynamic analyzes, including several sensitivity studies of parameters such as yield drift, strain hardening and the ratio tension to compression force of braced frames. This document summarizes some technical aspects of the retrofit guidelines and results obtained from the sensitivity study.

1 INTRODUCTION

The Ministry of Education of British Columbia (BC) has allocated CD$1.5 billion for the assessment and retrofit of the province's schools. Engineers and researchers have been working on implementing a set of guidelines (herein referred as the guidelines) for the practitioner engineers in order to use the Ministry's fund in a cost-effective manner. There are several schools in the project that require important seismic upgrading in the short term. The most common structural systems in the schools are plywood shear-walls, un-reinforced masonry walls and reinforced concrete squat walls. However, steel systems, such as eccentrically braced frames and moderately ductile steel moment-resistance frames, have also been studied and included in the guidelines as feasible retrofit options.

A performance-based approach has been used to define the assessment and retrofit targets in this project. This approach consists mainly on obtaining a maximum resistance demand for the systems for a given performance objective. The project group has adopted this objective as the failure of the system under a very-rear event (2500 year return period). The failure is associated to an instability drift ratio based on tests, recommendations and/or previous experience of these systems under high-intensity earthquakes.

Nonlinear dynamic analyses have been carried out to determine the maximum resistance under a certain drift demand. The results are then presented in the form of resistance tables that associate these two responses. Each structural system or prototype has been classified according to its core material, overall nonlinear behaviour and type of failure. The parameters to define the backbone curves and the hysteretic behaviour of prototypes were adopted from previous research and the literature (Saatcioglu and Humar 2003, ATC 1997). Some structural responses are very sensitive to these parameters, e.g (1) yield drift and (2) strain hardening of tension only braced and moment frames and (3) the ratio tension to compression force of tension/compression braced frames. An attempt has been made to define a proper value for these parameters in order to include as many configurations as possible for retrofit solutions.

The main objective of this paper is to briefly describe the current seismic assessment/retrofit procedure adopted in BC for the schools and some technical aspects, specially related to steel systems. The technical aspects correspond to a description of the nonlinear dynamic analyses, mathematical models and their corresponding sensitivity studies.

2 STEEL PROTOTYPES

The guidelines include the following forms of steel construction (prototypes) for low-rise school buildings:

(a) Prototype S-1 for concentrically braced steel frames with tension bracing only;
(b) Prototype S-2 for concentrically braced steel frames with tension/compression bracing;
(c) Prototype S-3 for eccentrically braced steel frames;
(d) Prototype S-4 for moderately ductile steel moment-resisting frames;

Unless a detailed analysis indicates otherwise, chevron braces are to be considered to have a negligible contribution in the assessment of risk.

Table 1 shows a summary of the steel prototypes with their respective instability drift limit, ISDL, and over-strength factor, R_o, according to the code (NBCC 2005). The ISDL was defined for a life safety objective and the Ro factor was used to modify the resistance values obtained from the analysis.

Table 1. Summary information of steel prototypes

Prototype No.	ISDL	R_o	Hysteretic Properties
S-1	4.0%	1.3	Slip
S-2	1-2.5%	1.3	Slip/Buckling
S-3	4.0%	1.5	Elastic-Plastic
S-4	4.0%	1.5	Elastic-Plastic

2.1 *Modeling*

Steel prototypes S-1, S-3, and S-4 were based on models commonly found in the literature (Saatcioglu and Humar 2003). The model for prototype S-2 (tension/compression CBF) was based on the Jain-Goel model shown in FEMA 274 (ATC 1997).

Backbone curves for the steel prototypes are shown in Figure 1. Hysteretic rules are shown in Figure 2. Yield drifts for the frame systems (0.3%) are based on a brace angle of 45 degrees, and no significant strain hardening is used in the models.

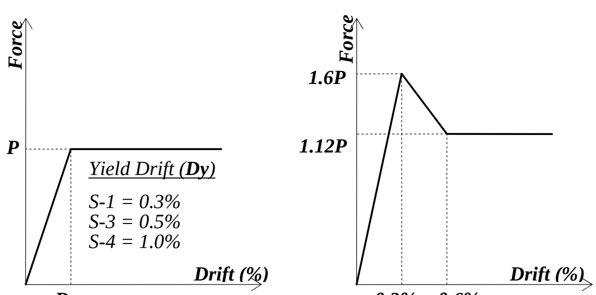


Figure 1. Backbone curves for steel prototypes S-1, S-3 and S-4 (left) and S-2 (right)

Prototype S-2 assumes that the compression strut has a maximum strength of 60% of the tension brace. The strength of the compression strut drops to 20% of its original capacity after buckling. In addition, the strength of prototype S-2 is based only on the strength of the tension strut.

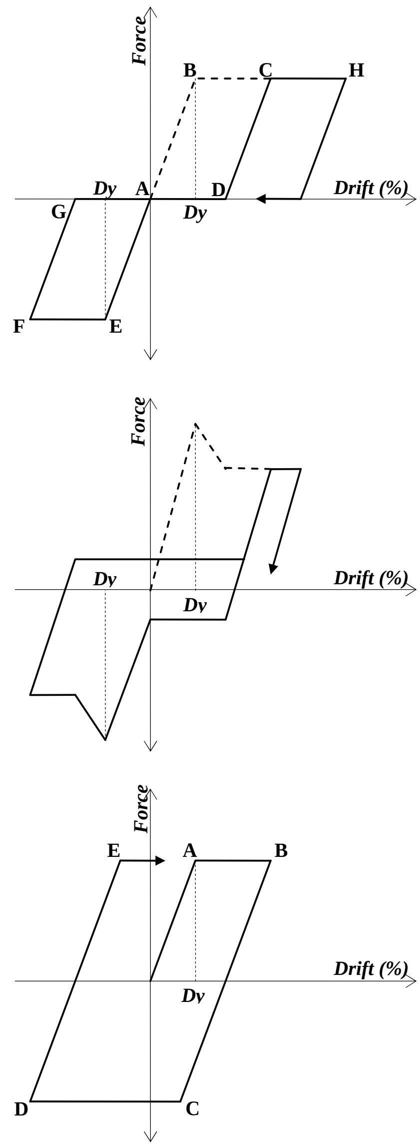


Figure 2. Hysteretic rules for steel prototypes S-1 (top), S-2 (middle), S-3 and S-4 (bottom)

3 ANALYTICAL MODELS

All LDRS prototype models were based on 2-storey structures with equal strength on both stories. This two-dimensional model was comprised of a single degree of freedom (horizontal deformation), with masses lumped at the 2nd floor and at the roof. Each clear storey height was 3 meters. The total weight of the building was divided into three parts. Two parts were placed at the 2nd floor and the third at the roof. The P-delta influence was properly modeled for moment frame prototypes S-3 and S-4, with a frame structural model (3 meters wide). The inelastic elements of prototypes S-3 and S-4 were the beam and the column elements, respectively.

3.1 *Non-linear Dynamic Analysis*

Non-linear dynamic analysis (NLDA) was used to predict inelastic deformations between stories, which are required for a precise measurement of the life-safety performance of a structure. Computer program CANNY (Kangning 2006) was used to perform the NLDA. CANNY uses an explicit stiffness-based approach (direct stiffness method) with the seismic input based on acceleration time histories. CANNY has many different types of structural elements and hysteretic models to choose from.

3.2 *Input Motions*

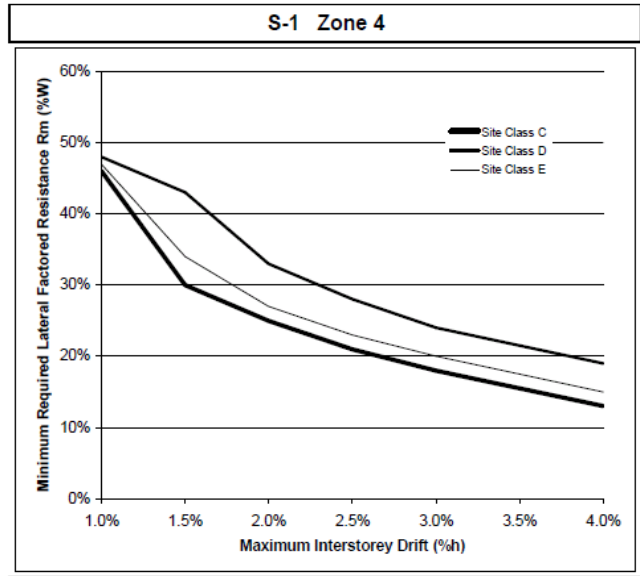
In the first two editions of the guidelines, a suite of 10 crustal earthquake motions were used as the input motions to run the NLDA. This suite is mainly based on the Northridge 1994 and Loma Prieta 1989 earthquake. Each record was corrected by a single scaling factor that allows the matching of spectral velocities within a range of period with the target velocity spectrum derived from the code (NBCC 2005). It is believed that this matching procedure be a better predictor of the potential damage in the structure. The range of period is defined according to the population of prototypes to assesses or retrofit. A range of 0.5 to 1.5 seconds was defined as a proper range of periods for most of the prototypes defined in the guidelines.

4 RESISTANCE TABLES

The Resistance Tables represent the resistance required to limit the drifts to desired level. . This level ranges from the ISDL down to a recommended level, below which the prototype can no longer efficiently limit the drift. A sample of a resistance table for an S-1 prototype is shown in Figure 3.

A Resistance Table is comprised of a plot of the Minimum Required Lateral Factored Resistance, *Rm*, versus the Maximum Interstorey Drift. The *Rm* is the maximum resistance (as a percentage of the total weight) defined in the backbone curve divided by the overs-strength factor, R_o. The Maximum Interstorey Drift is the maximum drift (as a percentage of the interstorey height) of the first or second storeys obtained from the nonlinear dynamic analysis, NLDA. Some specific *Rm* values are also given at the bottom of these tables for each site class.

NLDA were used to generate the resistance values. Each prototype was analyzed multiple times for each combination of seismic zone and site class. These multiple analyses provided data to develop a relationship between strength (resistance) and maximum interstorey drift. For the suite of 10 ground motions, the mean plus one standard deviation was defined as the risk and retrofit values shown in the Resistance Tables.



Site Class	1.0%	1.5%	2.0%	2.5%	3.0%	4.0%
C	46%	30%	25%	21%	18%	13%
D	48%	43%	33%	28%	24%	19%
E	47%	34%	27%	23%	20%	15%

Figure 4-1(c) *Minimum Required Lateral Factored Resistance R_m (%W)*
Steel Prototype S-1 Concentric Braced Frame (Tension Only)
Seismic Zone 4 (Vancouver)

Figure 3. Sample of a Resistance Table for Prototype S-2 in seismic Zone 4

The Resistance Tables provide engineers the rational means to assess or retrofit structural systems to a desired level of performance (i.e. drift). These tables are the product of a very large analysis program that has been validated and thoroughly reviewed by an external peer review committee.

5 THE TOOLBOX METHOD

The Toolbox method is a simplified method for combining resistance contributions from mixed lateral systems within a low-rise building in a drift-compatible manner. This method is based on the ability of the mixed lateral systems to redistribute inertia mass within their group.

Low-rise school buildings often comprise more than one LDRS. The retrofit guidelines present the minimum factored resistance requirements in a format that permits the design engineer to treat each LDRS individually and then combine the LDRSs in a drift-compatible manner for overall building performance.

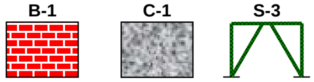
This method is restricted in application to (1) low-rise buildings (1-3 stories), (2) well-defined load path, (3) diaphragms with adequate strength and wall connections, (4) plan eccentricity not greater than 30% in one direction and 20% in the orthogonal direction, (5) no diaphragm torsional redistribution of inertia forces in steel or wood frame buildings.

5.1 *Example*

An example of the application of the "Toolbox Method" is presented in Figure 4. In this case a two-storey building located in Seismic Zone 4 on Site Class C soils is considered. The second storey of the building has been assessed as having an acceptable level of risk. The evaluation will focus on the first storey of the building. The two predominant structural systems in the building are unreinforced clay brick masonry load bearing wall, B-1, and moderately ductile reinforced concrete shearwalls, C-1. The existing and required resistances for each prototype are named as *Re* and *Rm*, respectively, and the resistance ratio is given by *Rr*. The following are the results from this assessment/retrofit example:

- The first set of calculations checks the level of risk. Prototype B-1 determines the GDL (1.0%) in this case. The existing building requires upgrading because the sum of the *Rr* (*Rrt*) is less than 1 (0.77 in this case).
- The first retrofit option is to upgrade both prototypes as illustrated. In this case, an increment on the required resistance, *Re*, of both prototypes is needed to reach an *Rrt* above one (1.01 in this case).
- The second retrofit option is to demolish or isolate the URM clay brick masonry wall, thereby increasing the GDL to 2.0% (governed by concrete shearwall). Upgrading with an eccentrically steel braced frame now results in an acceptable retrofit solution.

Prototypes



	B-1	C-1	S-3	
Risk Assessment - GDL 1%				
Re	9	10		
Rm	22	28		**Total Rrt**
Rr	0.41	0.36		**0.77**
Retrofit Option 1 - Upgrade Both Systems - GDL 1%				
Re	12	13		
Rm	22	28		**Total Rrt**
Rr	0.55	0.46		**1.01**
Retrofit Option 1 - Remove B-1/Upgrade with S-3 - GDL 2%				
Re		10	10	
Rm		21	19	**Total Rrt**
Rr		0.48	0.53	**1.00**

Figure 4. Assessment and retrofit example of a low-rise building

6 SENSITIVITY STUDY

Many more analyses were performed than are shown on the resistance tables, for the purposes of defining the trends and validation. A number of the modeling assumptions for the steel prototypes were investigated in a sensitivity study. Below is a brief summary and details can be found in the Commentary section of the 2nd Editions of the Bridging Guidelines (APEGBC 2006). The following results are presented using the same format as in the Resistance Tables and correspond to the mean plus one standard deviation obtained from the suite of 10 ground motions.

6.1 *Yield Drift*

Variations in the yield drift for the S-1 and S-2 prototypes can be associated with a difference in the angle of the brace, as well as the relative stiffness of the rest of the frame. Figure 4 indicates that the response of the steel braced frames (tension only) is mildly sensitive to the variation in stiffness. It is assumed that this behaviour would also be similar for prototype S-2, as they have similar characteristics and the same stiffness (yield drift).

The results for S-4 prototype show that there is a significant amount of scatter for the yield drifts ranging from 0.8% to 1.2%. There is no clear trend, but it appears that the default yield drift (1%) is a reasonable estimate for this range of stiffness.

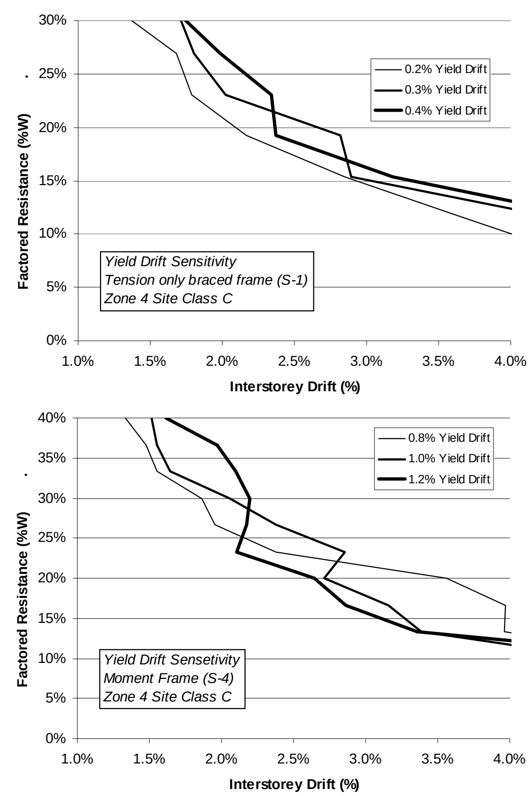


Figure 4. Factored resistance versus drift curves of prototypes S-1 and S-4 for different yield drifts

6.2 *Strain Hardening*

The default strain hardening for S-1 is 0.1%. Modeling a low rate of strain hardening is undoubtedly conservative. This sensitivity study investigates the degree of conservatism. Figure 5 shows that there is a small sensitivity in the response of prototype S-1 to the level of strain hardening, ranging from 0.1% to 2%. The default value of 0.1% results in a slightly conservative response compared to the higher levels of strain hardening, but it is not significant enough to modify the existing model.

The default strain hardening for S-4 is 0.1%. Modeling a low rate of strain hardening is conservative. Figure 5 shows that there is a small sensitivity in the response of prototype S-4 to the level of strain hardening, ranging from 0.1% to 5%. The default value of 0.1% results in a conservative response compared to the higher level of strain hardening.

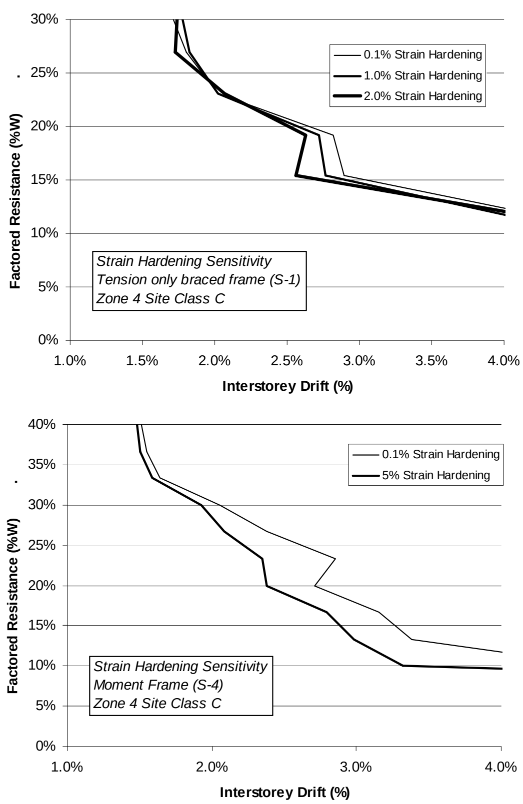


Figure 5. Factored resistance versus drift curves of prototypes S-1 and S-4 for different strain hardening values

6.3 *Compression to Tension Strength Ratio*

The default yield drift for S-2 is 60%, which means the strength of the compression brace is 60% of the strength of the tension brace. This value was taken as it was felt that it was the most representative of tension/compression braced frames. Note that the factored resistance is based solely on the strength of the tension brace (i.e. the strength of the compression brace is not included in the factored resistance or demand). The plot indicates that there is some variation in the response of the system based on the relative strength of the compression brace, as one might expect. However, this variation is small. Braced frame systems with high relative compression strength will be conservative if designed to the Resistance Tables.

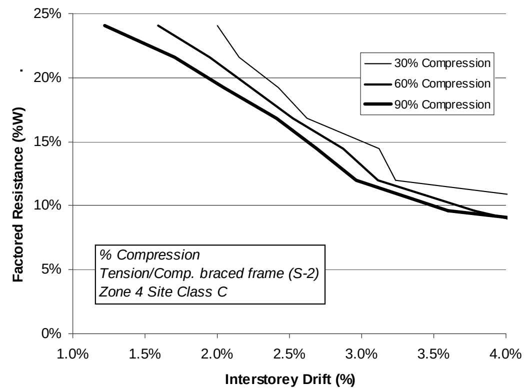


Figure 6. Resistance versus drift curves of prototype S-2 for different compression to tension ratios.

7 FUTURE STUDIES

It has been found that the structural responses are very sensitive to the type of earthquake motion used in the nonlinear analyses. The province of BC is indeed located in a very complex tectonic setting where different earthquake sources can be allocated. The hazard of any of these sources is different and could trigger different responses in the structures. The project group is currently working on a probabilistic-based assessment approach that allows the use of different seismic sources in the nonlinear analyses and defines the structural risk accordingly.

In the example provided above, the two retrofit solutions may have to follow a feasibility study in order to make the final decision. The project group is also working on the development of economic and technical recommendations for the feasibility evaluation of multiple retrofit options.

8 FURTHER REMARKS

The professional and research group is currently preparing the first edition of the Technical Guidelines for the Assessment and Retrofit of BC Schools. This new document will supersede the past editions of the BG, but it will keep the basis of the “Toolbox Method” and the Resistance Tables due to its simple, easy-to-follow and highly accepted procedure for the professional community in the province.

It is expected that the retrofit plan be strongly commenced by later this year by following a risk ranking also prepared for the research group of this project. The retrofit plan will benefit from the recommendations and solutions that are being developed and tested as part of this project.

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