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SEISMIC BEHAVIOUR OF RESIDENTIAL WOOD-FRAME CONSTRUCTION IN BRITISH COLUMBIA: PART I – MODELING AND VALIDATION

T. W. White[1] and C. E. Ventura[2]

ABSTRACT

Many single-family, wood-frame houses in British Columbia were built before the National Building Code of Canada mandated seismic design provisions for these types of structures. This suggests that there are likely many houses with inadequate seismic performance, because parts of British Columbia are regions of moderate to high seismicity. A simple and rational method is required to fairly assess and to provide an adequate measure of safety for seismic retrofits. A set of pre-generated seismic performance curves, specifically targeting the types of structures in question and accounting for local seismic conditions, is one such method.

This paper presents an overview of the development of performance curves for single-family houses in BC. Static test results and results from the program CASHEW are used to build non-linear house models for the non-linear dynamic analysis program SAWS. These SAWS models were then validated against full scale dynamic shake table tests. The final stage was an implementation of the models into a non-linear static procedure from FEMA-440, from which the performance curves were generated.

Introduction

Over 95% of all residential buildings in Canada are of wood-frame construction. The design and construction of wood-frame buildings is governed by the National Building Code of Canada (NBCC, 2005). While seismic provisions are provided for new construction, there is no absolute measurement of seismic performance, and traditionally “non-engineering” materials, such as gypsum wallboard or stucco, are not accounted for. Additionally, many older wood-frame buildings were not specifically designed to resist earthquake shaking. While much of Canada has little to no seismicity, parts of the West Coast of Canada and the St. Lawrence Valley have a moderate to high seismic hazard. The 1994 Northridge earthquake in California demonstrated that both existing and new wood-frame buildings do have a significant seismic risk and when not addressed can lead to the loss of property and life.

In 1999 the University of British Columbia (UBC) and its industry partners undertook an extensive experimental program focusing on local wood-frame construction practices. The tests included static and dynamic testing of stucco, plywood, gypsum and horizontal board panels, as well as shake table testing of a series of full scale two-storey wood-frame houses. The overall testing program was conducted in the

[1] Project Engineer, Bush, Bohlman & Partners, Vancouver, BC, V6G 2Z6
[2] Professor, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC, V6T 1Z4

Earthquake Engineering Research Facility (EERF) and is referred to as Earthquake 99 (EQ-99). A complementary project was conducted at UBC in 2005. The goal of this project, entitled “Seismic Performance of Residential Wood-frame Construction in BC” (EERF Reports No. 06-02 and 06-03, 2006) was to develop performance criteria for wood-frame housing using advanced analysis techniques in conjunction with the seismic hazards from the 2005 (NBCC). The above EERF reports are available from the Canadian Mortgage and Housing Corporation (CMHC).

This paper presents the methodology used to generate performance curves for four types of residential wood-frame construction in British Columbia. This methodology involved building a non-linear model for the computer application SAWS (CUREE, 2003), based on laboratory data of shearwall tests, conducted at UBC (EQ-99), and shearwall load-deformation data from the computer program CASHEW (CUREE, 2001). The model was validated against full-scale dynamic shake table tests (EQ-99). The SAWS model was then adapted to the FEMA-440 Displacement Coefficient Method (ATC, 2005), where it was analyzed using the design spectra from the 2005 NBCC.

The performance curves from this study are presented in a companion paper entitled “Seismic Behaviour of Residential Wood-frame Construction in British Columbia: Part II – Performance Requirements” (White and Ventura, 2007) which also presents a simplified method to assess the performance of existing small residential wood-frame buildings in BC.

Scope

The paper specifically focuses on the seismic performance of two-storey, single-family, wood-frame dwellings in British Columbia. Four types of houses will be considered, based on the sheathing and finishing materials used to construct the shearwalls:

1) Blocked Plywood/OSB Shear walls with exterior stucco cladding and gypsum wall board interior finish.
2) Blocked Plywood/OSB shear walls with gypsum wall board interior finish.
3) Unblocked Plywood/OSB shear walls with gypsum wall board interior finish.
4) Horizontal Board sheathing (shiplap) with gypsum wall board interior finish.

Experimental Data from EQ-99

The Earthquake 99 Wood-frame House Project (EQ-99) commenced in 1999 with the goal to investigate the performance of existing house stock in British Columbia and California. The experimental testing was divided into three phases. Phase I consisted of quasi-static and dynamic testing of 8 ft. by 8 ft. walls with different sheathing configurations. A full report on the data can be found in Rudolf et al. (1998). Phase II consisted of shake table testing of a single storey subsystem with different sheathing configurations. Phase III saw shake table testing of a two-storey house with different sheathing configurations. Details of the shake table testing program can be found in Kharrazi et al. (2002). Summaries of relevant information on the phases are discussed below.

Static Test Results

A total of 23 different panels (8 ft by 8 ft) were tested. The sheathing materials included: oriented strand board (OSB), plywood, horizontal board (shiplap), and gypsum wallboard (GWB). Hold downs and blocking where not used, and thus the OSB shearwalls are considered to be “non-engineered”.

Static test data from this experiment was used to develop models of the different sheathing materials. Fig. 1 shows one side of the load-deformation curves for (a) non-engineered OSB (1 side), (b) OSB/GWB, (c) GWB (2 sides) and (d) shiplap/GWB respectively. Due to instrumentation difficulties, only one side of the load-deformation curve was available. Note that these curve display a considerable degree of strength/stiffness degredation, as well as a highly pinched behaviour. For this reason nonlinear analysis should be used to assess the performance of wood-frame structures.

These four experimental tests were used to develop the shear spring wall models for SAWS presented below. These particular tests were used, as the method of construction and materials were the same as those used in the shake table tests, which were then used to validate the house model.

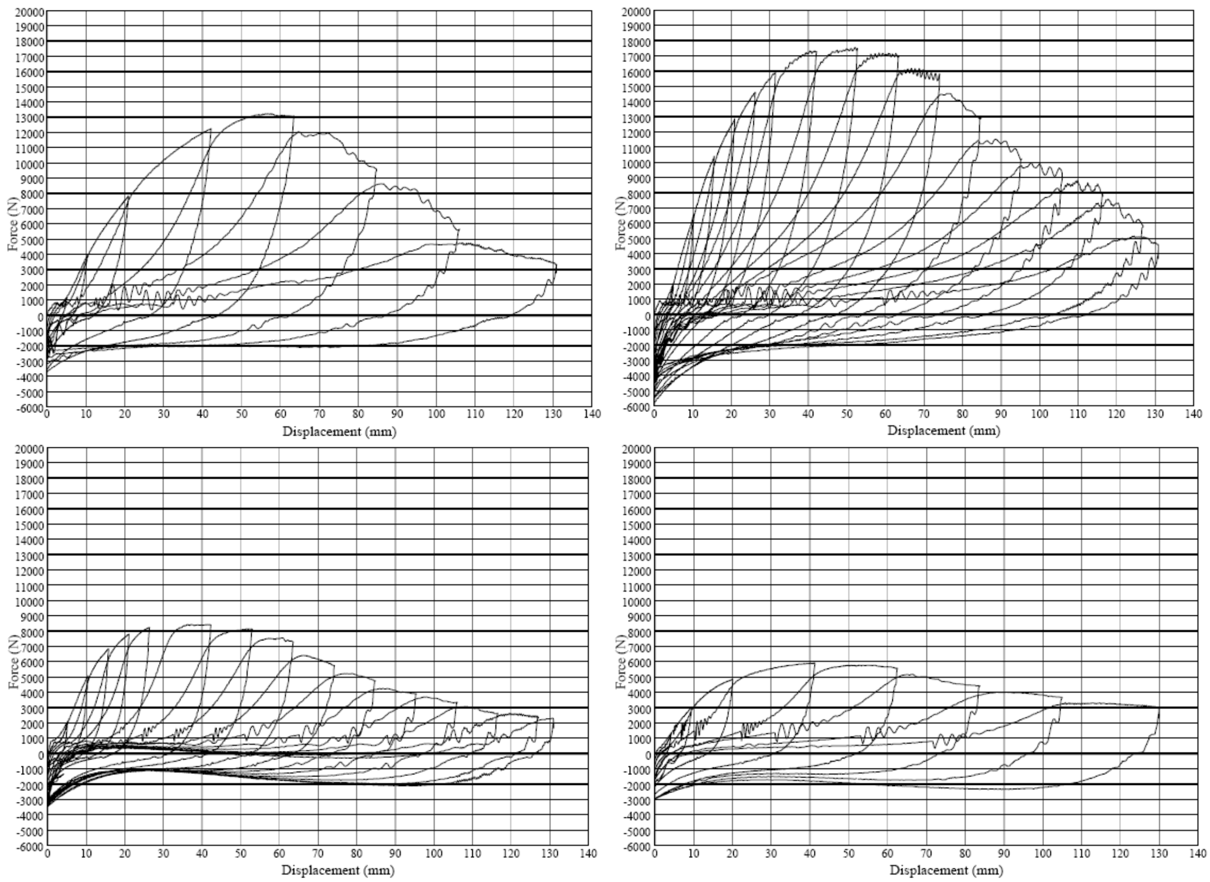

Figure 1. EQ-99 Static Test Results for (a) Panel P2 (8’x8’ Non-engineered OSB 1 side), (b) Panel P3 (8’x8’ Non-engineered OSB/GWB), (c) Panel P7 (8’x8’ GWB both sides) and (d) Panel P10 (8’x8’ Horizontal Shiplap/GWB).

Stucco Tests

Laboratory testing of Stucco consisted of 18 static tests on 8 ft. high by 4 ft. wide panels. All test panels were secured against sliding and overturning by anchor bolts and hold-downs. Each panel in the test had a different strapping and/or lath configuration. See Taylor et al. (2003) for details on the stucco panel static tests.

Fig. 2 below shows monotonic load-displacement (top of panel) behaviour for eight of the panels. Test S16, denoted with an asterisk (*), was for one of the rainscreen test panels. Its results are average for the series.

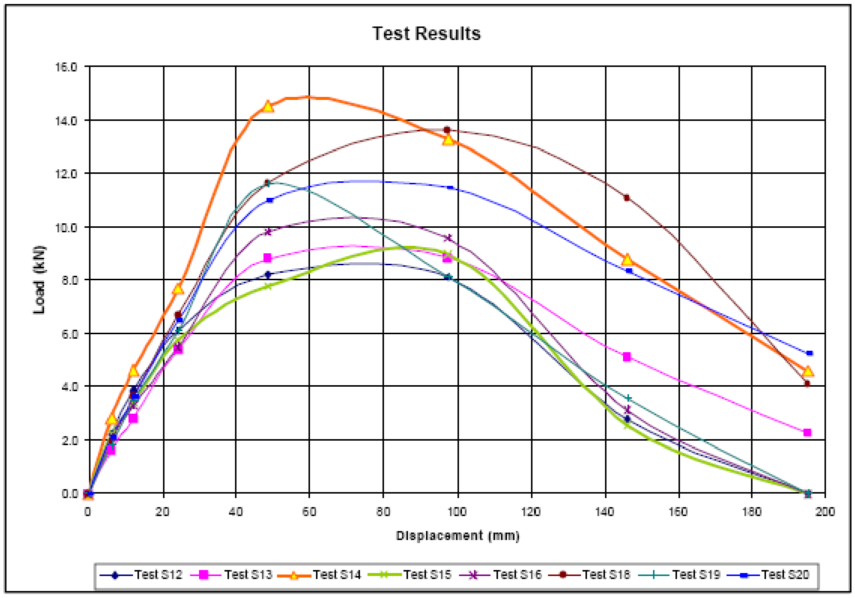

Figure 2. EQ-99 Series 1 Stucco Static Test Results

Dynamic Test Results

Full-scale tests were conducted on two types of buildings. One was a single storey sub-system, and the other a two-storey house. Both types of structures were intended to represent the behaviour of a full-scale two-storey house. The plan dimensions measured 25 ft by 20 ft. Tests were conducted in the long direction only (North-south). The sub-system test structure had concrete masses added on top, to represent the inertia of the 2nd storey. The results of the sub-system tests were used to refine the design and ground motions used in the two-storey tests. Drawings of the two-storey house specimens are shown in EERF Report No. 06-03 (2006).

Seven ground motions were used in the shake table testing program. These ground motions were selected to represent the seismicity in the Lower Mainland of British Columbia (Vancouver and surrounding area). Some ground motions were modified to obtain the appropriate seismic demands. These modifications are discussed in TBG (2002). The ground motions relevant to this study are summarized below in Table 1.

Table 1. Ground Motion Records used in the EQ-99 Shake Table Tests.

Earthquake Record	Station	Max. Disp. *cm*	Max. Vel. *cm/s*	Max. Acc. *cm/s^2*	Type
Nahanni 85	UBC modified Nahanni	11.70	33.44	315.90	Crustal
Northridge 94	Sherman Oaks	13.13	54.90	437.14	Crustal
Kobe 95	KJMA	19.95	74.32	587.14	Crustal
Landers 92	Joshua Tree	15.73	42.71	278.38	Crustal
Chile 85	Llayllay	8.40	41.79	345.47	Subduction

Both the sub-system and the two-storey house were tested multiple times. Each test consisted of running one or more ground motions (to simulate the response to aftershocks). After each test, the specimen was reconstructed with new sheathing material and interior finishes. Table 2 below summarizes the relevant portions of the test program, including the ground motions, the type of construction (refer to Scope), and a summary of the maximum inputs and responses of the specimens. More details of the shake table test results are presented in EERF Report No. 06-03 (2006).

Table 2. Summary of EQ-99 Shake Table Test Program and Results.

EQ-99 Test No.	House Type	Earthquake Record	Maximum Shake Table Input Acc. *g*	Vel. *cm/s*	Disp. *cm*	Initial Period *s*	Maximum Drift *%h*
9	2	Sherman Oaks	0.32	46.9	12.0	0.29	0.51
10	1	Nahanni	0.29	31.3	11.2	0.25	0.19
11	1	Nahanni	0.26	31.6	11.2	0.26	0.25
12	3	Landers	0.31	39.5	15.6	0.36	0.61
13	4	Kobe	0.68	57.9	21.7	0.37	8.19
14	2	Landers	0.39	52.5	21.6	0.32	0.50
15	2	Llayllay 175%	0.76	80.5	16.4	0.36	3.35
16	3	Llayllay 175%	0.75	79.8	16.4	0.38	3.34

Non-linear Models

A necessary step in developing the full-scale house models was to assign characteristics to the various types of shearwalls present within each type of house. Since SAWS was the program that would be used for all of the full house analysis, the material models (i.e. shear wall models) had to be compatible. This meant that all of the models had to use the same element model and hysteretic behaviour, as SAWS has only the one type available. Conveniently, this is also the same model used by CASHEW, and is shown in Fig. 3.

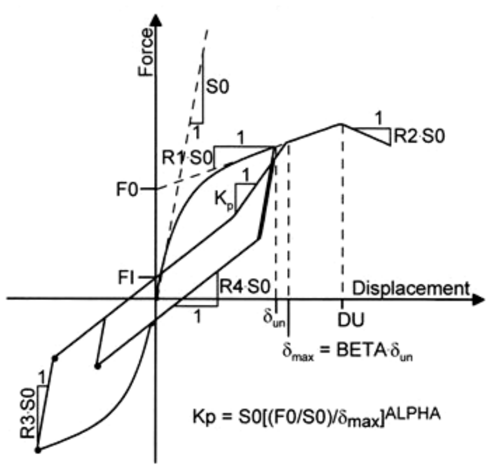

Figure 3. SAWS/CASHEW Backbone Curve and Hysteretic Model (CUREE, 2001).

This model represents the force versus horizontal displacement (at the top of a shear wall) relationship for a shearwall one storey in height. The large number of parameters in this hysteretic rule allows it to be adapted to any type of shearwall material, and not just OSB/plywood connected with nails.

CASHEW Model for Engineered (Blocked) OSB

CASHEW was used to develop the engineered OSB (blocked construction with hold downs) shear wall model. The model included specific material information about the OSB sheathing and the nails. In addition to monotonic and cyclic load-displacement data, CASHEW also provides the parameters for an equivalent single degree of freedom (SDOF) system, as defined above in Fig. 3. The parameters from the CASHEW analysis of the Engineered OSB shear walls, as well as the other material models are listed on Table 3.

Table 3. Shear Wall Material Model Parameters.

Shear Wall Description	CASHEW - SAWS Model Parameters									
	F_o	F_i	D_u	α	β	So	R_1	R_2	R_3	R_4
Blocked OSB	5.163	1.073	2.354	0.736	1.095	14.343	0.080	-0.094	1.293	0.070
GWB 1 Side	1.324	0.204	1.102	0.800	1.100	3.105	0.125	-0.177	1.000	0.005
Stucco 1 Side	7.334	0.815	2.835	0.800	1.100	6.210	0.069	-0.270	1.000	0.005
Unblocked OSB/GWB	6.519	1.121	2.087	0.770	1.100	9.108	0.034	-1.980	1.150	0.030
Horzontal Shiplap/GWB	1.956	0.407	1.575	0.800	1.100	3.519	0.088	-0.200	1.050	0.015

Notes: OSB - Oriented Strand Board
GWB - Gypsum Wall Board

Adaptation of EQ-99 Static Tests

The remainder of the shearwall material models were adapted from the EQ-99 static tests. Using the CASHEW/SAWS backbone curve and hysteretic model (Fig. 3) values for each of the parameters were established. Some of the parameters (α, β, R_3 and R_4) were more difficult to determine that the others. Guidance on these values for GWB and Stucco was taken from Folz and Filiatrault (2004) and CUREE (2002). The parameters F_o, F_i and S_o (strengths and initial stiffness) are assumed to be directly proportional to the length of the wall (a pure shear element), and were thus appropriately scaled, as there was a difference between the experimental wall lengths and the house model wall lengths. The models were based on the static EQ-99 tests, and not other experimental tests, as the type of construction used in the static EQ-99 tests was the same as used in the EQ-99 shake table tests, which would ultimately be used to verify these models.

Where possible, models representing combined wall systems (such as Shiplap/GWB or OSB/GWB) were used instead of modeling each material system individually. It was felt that this method was more representative of the combined systems and also streamlined the analytical process, by reducing the number of elements in the house model.

SAWS Full House Models

The generic structural model for the full house analysis done in SAWS is depicted below in Fig. 4. The shear walls are represented by single storey shear-spring elements, with the hysteretic models outlined above. The full house models also have interior shear walls, which are not depicted in Fig. 4. The floor and roof diaphragms are rigid. Only a two-storey model was used in this study. While the modeling methodology given here readily applies to a wider variety of wood-frame structures, only two-storey shake table test results were available for the validation of the model. An equivalent viscous damping of 3% in the first two modes was used.

Validation of Models

The results of the SAWS full house models were compared against the data obtained from the EQ-99 shake table tests. The SAWS models were run using the actuator motion recorded during the tests, which differs slightly from the original ground motions.

Table 4 compares the differences in initial period and maximum interstorey drift (expressed as a percentage of the storey height) between the test data and the analysis results.

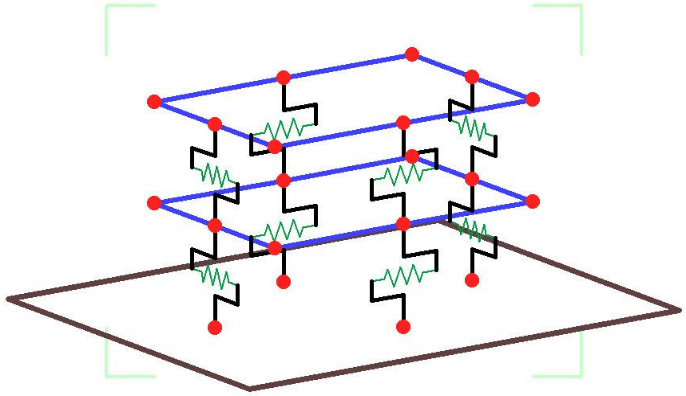

Figure 4. Generic SAWS Structural Model for Full House Analysis.

Table 4. Comparison of Test Data with Analysis Results.

EQ-99 Test No.	House Type	Test Data Period s	Test Data Drift %h	Analysis Results Period s	Analysis Results Drift %h	Test/Analysis Period	Test/Analysis Drift
9	2	0.29	0.51	0.35	0.79	0.84	0.65
10	1	0.25	0.19	0.31	0.39	0.81	0.49
11	1	0.26	0.25	0.31	0.46	0.83	0.54
12	3	0.36	0.61	0.39	0.77	0.93	0.79
13	4	0.37	8.19	0.40	8.87	0.93	0.92
14	2	0.32	0.50	0.35	1.01	0.92	0.50
15	2	0.36	3.35	0.35	3.20	1.02	1.05
16	3	0.38	3.34	0.39	3.19	0.99	1.05

The comparison shows a reasonable estimate of the initial period, within 19% of the test data. The EQ-99 test data indicates a stiffer structure than was modeled. One explanation for this are that the test periods were obtained by ambient vibration tests (see Kharrazi et al. 2002), which measures frequencies at under near rest conditions, where as the analysis stiffness was based on the load-deformation response of the individual shearwalls. A second reason is that the out-of-plane resistance of the walls was not accounted for in the SAWS model.

The difference in the drift values was more pronounced. The analysis over-estimated the drift at the low drift levels (1% or less), but was accurate in predicting the larger drifts (3% or greater). The ability to predict drifts of 1% to 4% is of the most interest for this study, as that is the performance range in which there is a good balance between economy and safety. This is also the drift range of the final performance curves.

In addition to comparing maximum drift and periods, the full drift time history of the 1st storey was compared between the test data and the analysis results, for all cases (see EERF Report No. 06-03, 2006). Fig. 5 shows a portion of the drift time history for EQ-99 Test No. 16 and the corresponding SAWS analysis of the Type 3 House (unblocked OSB and GWB).

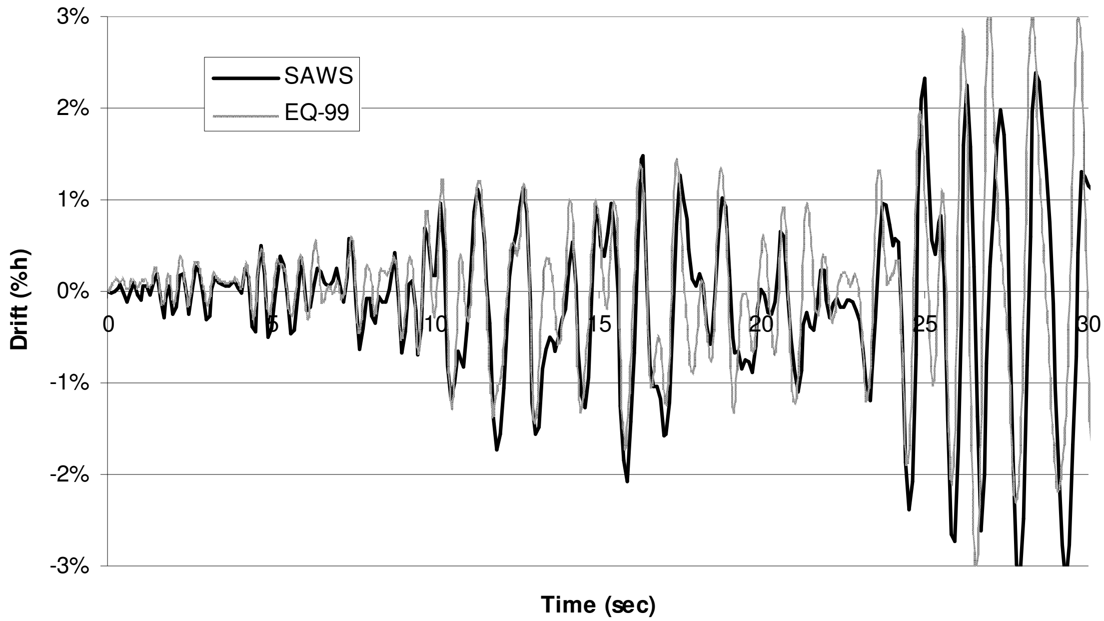

Figure 5. 1st Storey Drift Time History for EQ-Test No. 16 and SAWS Analysis of Type 3 House.

Fig. 5 shows a reasonable match between the drift time histories of the test and the analysis, up to the point of the maximum drift. The remaining 20 seconds of these two records do not show as strong a correlation, indicating that the data used for the model was not accurate enough to represent the heavily damaged house, but well enough to predict the maximum drift.

Development of Performance Curves

The validated models of each house type were subjected to a non-linear static "push-over" analysis to obtain both (a) the relationship between base shear and roof displacement (i.e. statically condensed single degree of freedom backbone curve), and (b) the distribution of drift between the two stories as it varied with the overall displacement. Backbone curve and the initial period were required to estimate the target displacement with FEMA-440, and subsequently the related maximum interstorey drift. Fig. 6 below shows the adaptation of the Type 3 House from SAWS into FEMA-440. A summary of the single degree of freedom (SDOF) properties for all the house types are shown on Table 5.

For each House Type, a range of base shear capacities were analyzed, assuming that strength and stiffness were proportional. This is a reasonable assumption for shear-dominated squat walls. Design spectra from the 2005 NBCC was used to estimate the drift for each house type on Site Class C, D and E for a variety of locations in British Columbia. These details are explained in White and Ventura (2007). Additionally, analysis on Site Class E included the influences of soil-structure interaction. Details of the FEMA-440 analysis (including specific parameters) can be found in EERF Report No. 06-03 (2006).

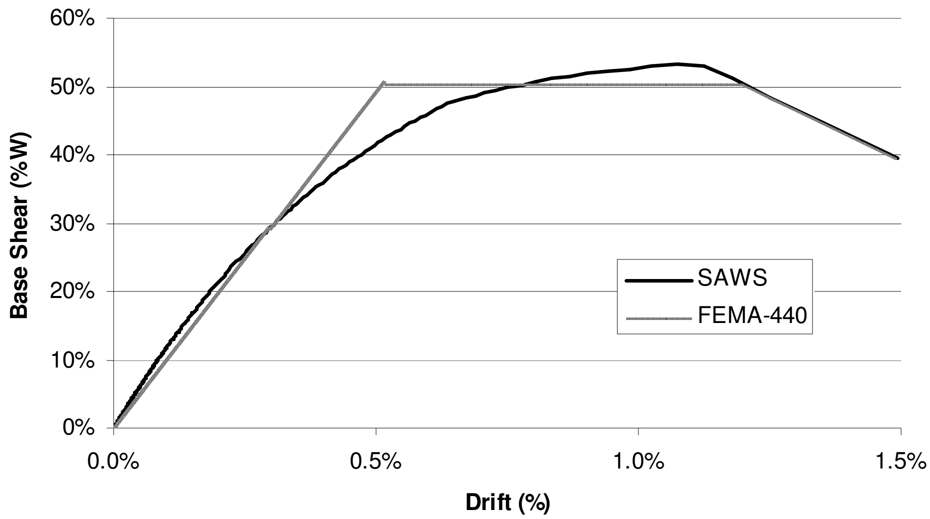

Figure 6. Backbone Curve of Type 3 House for SAWS and FEMA-440.

Table 5. Equivalent SDOF Parameters for FEMA-440.

House Type	Yield Drift	Drift Ductility	Post-yielding Stiffness
1	0.53%	3.0	-0.37
2	0.40%	3.3	-0.29
3	0.52%	2.3	-0.38
4	0.40%	2.2	-0.32

Conclusions

Models of single-family wood-frame houses were developed from experimental results (EQ-99) and implemented in a well-known non-linear dynamic analysis program (SAWS). The models were validated against dynamic shake table tests (EQ-99). After validation the models were adapted into non-linear static procedure (FEMA-440) to facilitate the development of the performance curves.

The following are specific conclusions of the validation of the models implemented in SAWS:

1) The initial periods matched well
2) The models were unable to accurately predict small drifts (<1%) obtained from the experimental test. However the performance curves are primarily interested in drifts ranging from 1% to 4%
3) The models were able to predict the maximum interstorey drift for both crustal and subduction type ground motions with reasonable accuracy, for all tests that had significant maximum interstorey drift (>1.0%).
4) There was strong correlation in the drift time history up to the point of the maximum interstorey drift, but not after. This is a result of the assigned hysteretic parameters, which governed the response of a heavily damaged house, not being accurate enough. Further testing would be required to improve them, but is not necessary for the scope of this project.
5) Overall the SAWS models showed an acceptable level of validation with the EQ-99 shake table tests, and should reliably predict maximum interstorey drifts between 1% and 4% drift, based on crustal earthquakes.

Acknowledgments

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