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PRELIMINARY *R*-VALUES FOR SEISMIC DESIGN OF STEEL STUD SHEAR WALLS

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ABSTRACT

Design codes recognize the ability of some structures to undergo significant inelastic deformation during a seismic event without reaching the point of collapse. In consideration of this behaviour, building codes provide force modification factors (*R*-values) to determine the reduced lateral loads that engineers may use in design. This paper presents an overview of the seismic requirements for various design standards and an explanation of how *R*-values may be determined from test results. The findings of an evaluation of existing steel stud shear wall test data, in addition to preliminary force modification factors for use in seismic design, are presented.

INTRODUCTION

The use of steel stud framing in homes and multiple storey buildings is increasing, due in part to the escalating construction costs of lumber structures, the scarcity of adequate lumber products, and in addition because of concerns such as pest resistance, product quality, *etc.* In future years the construction of steel stud framed buildings will, in all probability, increase across North America, including earthquake prone areas such as the west coast. Hence, it is of great importance to design engineers and contractors, as well as future building owners and occupants, that the lateral performance characteristics of steel stud structures are understood before their use in areas of known earthquake activity becomes widespread.

Under seismic ground motion, horizontal inertia forces develop at the roof and floor levels as a result of the accelerations experienced by the building mass. To resist these lateral loads the structure may include diagonal steel bracing, plywood sheathing, oriented strand board sheathing, gypsum wallboard or sheet steel sheathing in the walls (Fig. 1). These structural shear wall systems maintain the structural integrity of the building by transferring the seismic loads from the diaphragms at the roof and floor levels to the foundations.

Design codes recognise the ability of some structures to undergo significant inelastic deformation during a seismic event without reaching the point of collapse. In view of this behaviour, engineers may reduce the design forces associated with seismic action to values significantly less than those consistent with elastic theory. However, the lateral deflections of a building that is designed to behave inelastically can be several times that of the same structure when designed using an elastic analysis with reduced loads (Fig. 2). Most building codes provide force and deflection modification factors to determine the reduced loads and increased deflections that occur during a seismic event. An overview of various design standards has been included in this paper in order to compare the different methods used in the equivalent static approach to seismic design.

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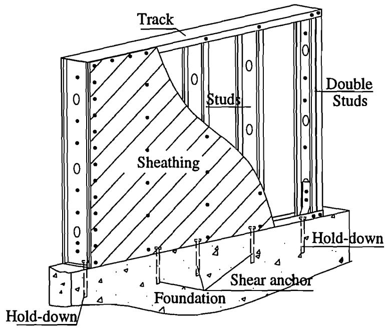

Fig. 1: Typical Cold-Formed Steel Shear Wall

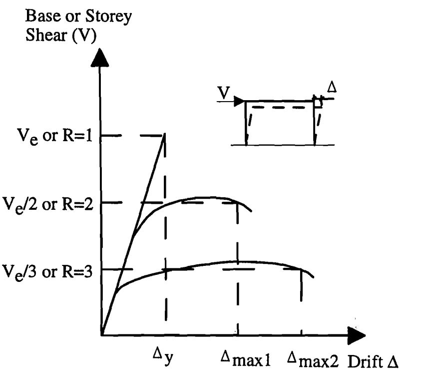

Fig. 2: Design Shear *vs.* Required Drift

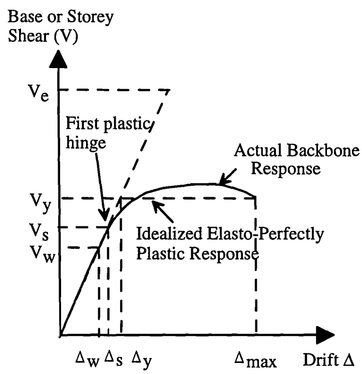

Fig. 3: Base or Storey Shear *vs.* Drift

DESIGN STANDARDS

NEHRP (National Earthquake Hazards Reduction Program)

The National Earthquake Hazards Reduction Program design method *(FEMA, 1997a,b)* accounts for the ability of a structure to behave inelastically through the use of a response modification factor, R, which is used to reduce the elastic design spectral response acceleration in the short period range. The design seismic base shear, V, in a given direction and the associated inelastic displacement can be expressed as follows:

$$V = \frac{V_e}{R} \qquad \Delta_{max} = \Delta_s \times C_d$$

where V_e is the base shear corresponding to an elastic response of the structure, C_d is the displacement amplification factor, and Δ_s is the elastic drift under the design seismic base shear.

Uang *(1991)* showed that if the actual response envelope of the structure, considering drift *vs.* base or storey shear (Fig. 3), can be idealized as an elasto-perfectly plastic response curve, the system ductility modification factor, R_μ, and the structural over-strength factor, Ω, can be defined as:

$$R_\mu = \frac{V_e}{V_y} \qquad \Omega = \frac{V_y}{V_s}$$

where V_y is the actual yield strength level and V_s is the first significant yield strength level (prescribed design force). Therefore, the response modification factor can be defined as:

$$R = \frac{V_e}{V_s} = R_\mu \Omega$$

The values of R vary from 1.25 to 8.0, where the lower is recommended for an ordinary masonry wall and the higher for a ductile moment frame *(FEMA, 1997a)*. Uang also stated that the displacement amplification factor, C_d, could be determined from Fig. 3 as follows:

$$C_d = \frac{\Delta_{\max}}{\Delta_s} = \frac{\Delta_{\max}}{\Delta_y}\frac{\Delta_y}{\Delta_s} = \frac{\Delta_{\max}}{\Delta_y}\frac{V_y}{V_s} = \mu\Omega$$

The over-strength factor, Ω, may also be increased based on the material over-strength, the resistance factor used in design, the designer who intentionally introduces additional over-strength, *etc.* Furthermore, a structure that is designed with a high degree of redundancy, structurally optimised, and/or whose member sizes are controlled by a drift limit could be expected to have additional over-strength.

UBC (Uniform Building Code)

The Uniform Building Code *(ICBO, 1994)* specifies a design seismic force level for working stress design. The required elastic seismic force level can be reduced by a force modification factor R_w *(SEAOC, 1996)*.

$$V = \frac{V_e}{R_w}$$

Uang *(1991)* proposed that an additional factor, Y, is required for allowable stress design.

$$Y = \frac{V_s}{V_w} \qquad R_w = \frac{V_e}{V_w} = \frac{V_e}{V_y}\frac{V_y}{V_s}\frac{V_s}{V_w} = R_\mu \Omega Y$$

where V_w is the corresponding design force level for allowable stress design (Fig. 3). The R and R_w factors in the two design documents differ by a constant load factor Y ($\approx$1.4), which is dependent on the type of structural system. The values of R_w range up to 12.0, which is the factor specified for ductile moment frames.

As an estimate of the maximum inelastic deflection, Δ_{max}, that may develop in a design earthquake, the design lateral deflection computed using an elastic structural analysis, Δ_w, is amplified by a factor as follows:

$$\Delta_{max} = \Delta_w \times \frac{3}{8} R_w$$

The base shear equation in the UBC *(ICBO, 1997)* is presented in the load and resistance factor design format and is consistent with the NEHRP equation, where the R-factor has an approximate value of $R_w/1.4$. The maximum inelastic displacement Δ_{max} is calculated as follows:

$$\Delta_{max} = \Delta_s \times 0.7R$$

NBCC (National Building Code of Canada)

For regular structures, the NBCC *(NRCC, 1995)* allows engineers to use an equivalent static design approach to determine the seismic load. The NBCC states that regular structures must be designed to sustain a base shear, V, in each of their principal directions and to resist maximum lateral deflection without collapse. The following formulas are provided:

$$V = \frac{V_e}{R} U \qquad\qquad \Delta_{max} = \Delta_y \times R$$

where $U = 0.6$, V_e is the base shear corresponding to an elastic response of the structure, R is the force modification factor, and Δ_y is the lateral deformation of the structure based on an elastic analysis. The factor, R, depends on the ability of the structure to maintain load carrying capacity over extended lateral displacement; hence, it will vary depending on the type of structural system that is specified. R-factors are determined based on experience acquired in terms of design and construction and also from the study of experimental structures, analytical results, and building behaviour during earthquakes. The values of R vary from 1.0 to 4.0 (R/0.6 vary from 1.7 to 6.7), where the lower is recommended for an unreinforced masonry wall and the higher for a ductile moment frame.

PROCEDURE TO DETERMINE R-VALUES FROM TESTS

In this section a description of a possible method that can be used to determine force modification factors, R, from quasi-static reversed cyclic tests for use with the various design standards is provided. The steps are as listed:

i. Depict the Unidirectional "Actual Response" (Backbone Curve)

The backbone curve is taken as the envelope of cyclic curves based on the highest strength hysteretic response from a plot of storey shear *vs.* storey deflection (Fig. 4) (recommended by SEAOSC *(1997)*).

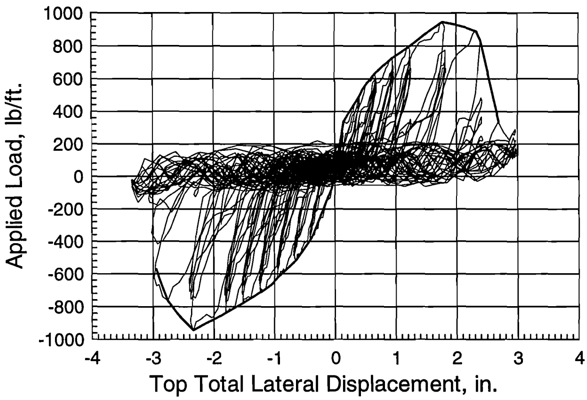

Fig. 4: Typical Steel Stud Shear Wall Load *vs.* Displacement Hysteresis *(Serrette et al., 1996b)*

ii. Evaluate the Ideal Bilinear Curve and Compute the Ductility Factor μ

An ideal bilinear curve is comprised of two segments, where the first segment represents the shear stiffness of the wall, which is dependent on the definition of the yield displacement. Park *(1989)* recommended various definitions for the yield displacement. In consideration of the difficulty in accurately defining the first yield point of a cold-formed steel shear wall sheathed with wood panels, sheet steel or steel straps, the equivalent elasto-plastic yield, which is defined as having the same elastic stiffness as the real structure, was incorporated into this study (Fig. 5).

The second segment of the ideal bilinear curve consists of a plateau over which the shear displacement of the wall increases while the applied load remains constant. A lower and upper bound value for the plateau portion can be established from the storey shear *vs.* storey deflection envelope curves (Fig. 6). The lower and upper bounds occur when the plateau portion of the curve (plastic behaviour) intersects the peak load in the envelope and the failure load, respectively. An idealized bilinear elasto-plastic shear *vs.* deflection plateau can be selected within these bounds by taking into consideration the hysteretic energy dissipation, the resistance to degradation, the inherent redundancy, the number of cycles resisted, the failure mode, *etc (Driver et al., 2000)*. The ductility factor, μ, can be determined using the bilinear curve:

$$\mu = \frac{\Delta_{max}}{\Delta y}$$

where Δ_{max} is the measured deflection at the intersection of the actual and idealized bilinear response curves (Fig 6), and Δ_y is the pseudo yield deflection, which occurs at the intersection of the two segments of the idealized bilinear curve.

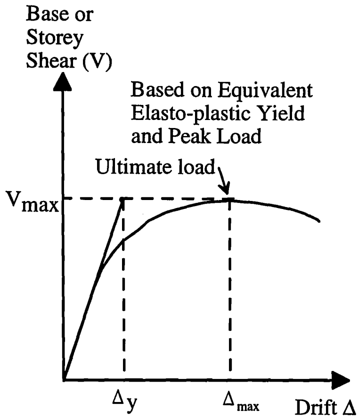

Fig. 5: Definition for Δ_y and Δ_{max}

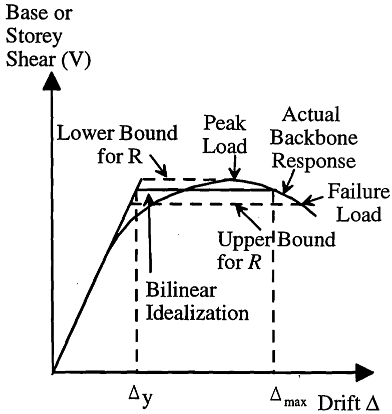

Fig. 6: Backbone Curve for R-Value Calculation

iii. Estimate the Ductility Modification Factor R_μ

The relationship between the ductility modification factor, R_μ, and the ductility demand factor, μ, must be determined. Newmark and Hall *(1982)* demonstrated that for a single degree of freedom system with a period greater than 0.5 seconds, the maximum lateral displacement of a non-linear system is almost equal to the maximum displacement of a corresponding linear system. However, for a system with a period from approximately 0.125 to 0.5 seconds, the strain energies for the elastic and the elasto-plastic cases are approximately the same. Hence, the following relationships were recommended:

$R_\mu = \mu \quad (T \geq 0.5)$ $\qquad$ $R_\mu = \sqrt{2\mu - 1} \quad (T < 0.5)$

iv. Establish the Over-Strength Factor Ω

It is possible to determine the nominal strength, F_n, of a shear wall from the load reached during the last stable loop used to define the backbone curve. A stable loop is obtained when the shear force in successive cycles of a given displacement amplitude is within 5%. The first significant yield strength, F_s, occurs at the point in the elastic segment of the bilinear curve beyond which the backbone curve deviates significantly from this idealized curve. Thus, as shown for the definition of ductility factor, one can define the over-strength factor, Ω_0, as a function of the nominal and first significant yield strengths. This over-strength factor can also be further increased by other sources as noted previously.

$$\Omega_0 = \frac{F_n}{F_s}$$

v. Evaluate the Force Modification Factor

The definition of the force modification factor depends on the corresponding building code. For the three codes that are under consideration R is defined as:

$R = R_\mu$ (NBCC) $\quad$ $R_w = R_\mu\,\Omega\,Y$ (UBC-1994) $\quad$ $R = R_\mu\,\Omega$ (NEHRP; UBC-1997)

CALCULATED R-VALUES OF COLD-FORMED STEEL SHEAR WALLS

A number of experimental research programs have been conducted to investigate the behaviour of single-storey cold-formed steel shear walls with different sheathing material, and to provide design capacities for a variety of wall types. A partial listing includes: Serrette *(1994, 1997)*, Serrette & Ogunfunmi *(1996)*, Serrette *et al. (1996a,b, 1997a,b)*, Salenikovich *et al. (2000)*, and COLA-UCI *(2001)*. The AISI has published a shear wall design guide *(1998)* in which the shear capacities of walls subjected to wind load were based on monotonic tests, while those under seismic load were derived from the results of cyclic tests. In this paper, the results of cold-formed steel stud shear wall tests carried out by Serrette *et al. (1996b, 1997b)* and COLA-UCI *(2001)* were used in the

Table 1: Calculated R-Values for Steel Stud Shear Walls with Wood Sheathing

Specimen	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)	*Specimen*	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)
AISI-OSB1[1]	5.9	4.2	2.3	AISI-B3	5.3	3.8	2.1
AISI-OSB2	6.3	4.5	2.5	AISI-B4	5.8	4.1	2.3
AISI-OSB3	4.8	3.4	1.9	AISI-E1	8.3	5.9	3.2
AISI-OSB4	6.1	4.4	2.4	AISI-E2	7.2	5.1	2.8
AISI-OSB5	5.0	3.6	2.0	AISI-E3	6.8	4.9	2.7
AISI-OSB6	5.1	3.6	2.0	AISI-E4	6.8	4.9	2.7
AISI-OSB7	3.9	2.8	1.6	AISI-E5	4.6	3.3	1.8
AISI-OSB8	4.1	2.9	1.6	AISI-E6	5.0	3.6	2.0
AISI-PLY1	4.7	3.3	1.8	Group14A[2]	6.6	4.7	2.6
AISI-PLY2	5.7	4.1	2.2	Group14B	6.5	4.6	2.5
AISI-PLY3	5.9	4.2	2.3	Group14C	6.0	4.3	2.4
AISI-PLY4	5.0	3.6	2.0	Group15A	6.3	4.5	2.5
AISI-PLY5	5.0	3.6	2.0	Group15B	6.0	4.3	2.4
AISI-PLY6	5.2	3.7	2.0	Group15C	6.1	4.4	2.4
AISI-PLY7	4.1	2.9	1.6	Group16A	6.0	4.3	2.4
AISI-PLY8	4.1	3.0	1.6	Group16B	5.2	3.7	2.0
AISI-A1	6.1	4.4	2.4	Group16C	5.1	3.6	2.0
AISI-A2	5.1	3.7	2.0	Group17A	5.7	4.0	2.2
AISI-A3	5.0	3.6	2.0	Group17B	6.6	4.7	2.6
AISI-A4	4.8	3.5	1.9	Group17C	6.2	4.4	2.4
AISI-A5	5.0	3.6	2.0	Group18A	6.9	4.9	2.7
AISI-A6	4.8	3.4	1.9	Group18B	6.6	4.7	2.6
AISI-A7	5.1	3.6	2.0	Group18C	6.9	5.0	2.7
AISI-A8	5.1	3.7	2.0	Group19A	7.0	5.0	2.8
AISI-B1	5.4	3.8	2.3	Group19B	6.5	4.7	2.7
AISI-B2	6.2	4.4	2.4	Group19C	5.8	4.2	2.3

[1]AISI tests by Serrette *et al. (1996b, 1997b)* [2] Group tests by COLA-UCI *(2001)*

calculation of force modification factors. The test specimens were 8' in height, 2', 4', and 8', in length and were usually framed with 0.033" thick studs (0.043" and 0.054" thick studs were also specified in some cases), which were sheathed with wood panels (OSB or plywood) that were attached with screws spaced from 6"/12" to 2"/12". The wood panels were situated parallel to the studs, and hence had full blocking along their edges.

The overstrength factor can be defined as the ratio of the nominal strength to the code defined design capacity. Considering that the Uniform Building Code *(ICBO, 1997)* specifies that the shear capacity for load and resistance factor design is the nominal strength multiplied by a resistance factor of 0.55, the overstrength, Ω, can be taken as 1/0.55 = 1.82. The maximum displacement is defined based on the peak load (the lower bound) as shown in Fig. 5, which is also one of the possible definitions recommended by Park *(1989)*. The period ($0.125 \text{ s} \leq T \leq 0.5\text{s}$) was empirically calculated for the various test walls according to the NBCC *(NRCC, 1995)*. Table 1 contains a listing of the calculated R-values for the different codes following the procedures defined above. The mean R-values for the 52 shear wall specimens following the UBC 94, NEHRP & UBC 97, and NBCC are 5.7, 4.1 and 2.2, respectively, each with a coefficient of variation of 0.16.

COMPARISON WITH WOOD WALLS AND MASONRY WALLS

Wood and masonry shear walls have often been used to resist lateral loads; as well methods to determine their design strength and ductility are available *(Easley et al., 1982)* and documented in codes and standards *(CSA, 1994, 2001)*. Therefore, the results of tests on these wall types were used in an attempt to compare and evaluate the seismic behaviour of cold-formed steel stud shear walls.

Wood Shear Walls

The Canadian Wood Council Wood Design Manual *(CWC, 1995)* prescribes the use of different R-values depending on the type of lateral load resisting system and its ability to absorb earthquake induced energy. An R-value of 3.0 is assigned to all nailed plywood, waferboard and oriented strandboard shear walls which satisfy certain requirements including: panel orientation and configuration, panel thickness, width of framing member, fastener schedule, *etc.* A more recent document from the CWC *(2001)* recommends that a lower force modification factor (R = 2) be used for the design of shear walls sheathed with a combination of wood-based panel and gypsum wallboard, where the gypsum wallboard is considered to provide a shear resistance for the wall. Walls that are sheathed with wood-based panels alone, or where the effect of the gypsum is neglected, may still be designed with R = 3. Wood shear wall specimens that were tested at Virginia Polytechnic Institute and State University *(Salenikovich et al., 2000)* and the University of California at Irvine *(COLA-UCI, 2001)* were used in the comparison with steel stud shear walls.

Masonry Shear Walls

The National Building Code of Canada *(NRCC, 1995)* contains three classifications for masonry shear wall structures; each with a different value for the force modification factor to account for the inelastic performance offered by the reinforcing steel. R-values are defined as follows: i) R = 1.0 for unreinforced masonry, ii) R = 1.5 for reinforced masonry, and iii) R = 2.0 for reinforced masonry walls with nominal ductility. Masonry shear wall specimens tested at the University of

Colorado at Boulder *(Shing et al., 1989, 1990a,b, 1991)* were used in a comparison of the behaviour of wood stud, masonry and steel stud walls. Force modification values for the masonry shear wall test specimens were assigned according to the requirements of the CSA S304.1 Masonry Design Standard *(1994)*.

Comparison of Steel, Wood and Masonry Force Modification Factors

Force modification factors were determined for the wood and masonry shear walls following the procedure detailed previously. These test-based values were then compared with those specified in the National Building Code of Canada *(NRCC, 1995)* (Table 2). The current NBCC does not list an R-value for steel stud shear walls, hence $R = 1$ must be used for design. In general, the use of the procedure described in this paper to determine force modification factors for masonry walls yields a high ductility ratio because these walls are significantly stiffer, and hence, have small yield displacement values in comparison with steel and wood walls. Thus, a direct comparison of the lateral ductility cannot be made because of the substantial variation in behaviour of the wall types. In contrast, the construction of wood-stud walls and steel-stud walls is similar; hence, a direct comparison of the force modification values calculated using test results may be carried out. As shown in Table 2, the calculated test-based R-values for wood walls are relatively similar to those defined in the NBCC. The calculated R-values for steel walls range from 1.6 to 2.8 (except AISI-E1) while those of wood walls range from 2.1 to 3.3 (except 12A). In general, the steel stud walls exhibit a slightly lower ability to maintain load-carrying ability in the inelastic lateral deformation range.

Table 2: Calculated R-Values for Wood and Masonry Walls

Specimen	***R* (Cal.)**	***R* (NBCC)**	***Sample***	***R* (Cal.)**	***R* (NBCC)**	***Specimen***	***R* (Cal.)**	***R* (NBCC)**
Group 03A[w,1]	2.9	3.0	Group 23A	3.0	3.0	Specimen 1[m,3]	2.8	1.5
Group 03B	2.5	3.0	Group 23B	2.8	3.0	Specimen 2	4.0	1.5
Group 03C	3.0	3.0	Group 23C	2.8	3.0	Specimen 3	3.8	1.0
Group 04A	2.8	3.0	Group 34A	2.5	2.0	Specimen 4	2.6	1.0
Group 04B	2.6	3.0	Group 34B	3.0	2.0	Specimen 5	2.0	1.0
Group 04C	2.7	3.0	Group 34C	2.7	2.0	Specimen 6	2.6	1.5
Group 06A	2.4	2.0	Group 35A	2.6	3.0	Specimen 7	2.8	1.0
Group 06B	2.3	2.0	Group 35B	2.7	3.0	Specimen 8	3.1	2.0
Group 06C	2.4	2.0	Group 35C	2.4	3.0	Specimen 9	2.9	1.0
Group 09A	2.5	3.0	Group 36A	2.9	3.0	Specimen 10	3.4	1.5
Group 09B	2.5	3.0	Group 36B	2.1	3.0	Specimen 11	3.1	1.5
Group 09C	2.4	3.0	Group 36C	2.5	3.0	Specimen 12	3.7	1.5
Group 12A	3.7	3.0	04FAc1[w,2]	3.0	3.0	Specimen 13	3.0	1.0
Group 12B	3.0	3.0	04FAc2	2.9	3.0	Specimen 14	2.6	1.0
Group 12C	3.3	3.0	08FAc1	3.2	3.0	Specimen 15	4.5	1.5
Group 13A	2.6	3.0	08FAc2	2.9	3.0	Specimen 16	2.9	1.0
Group 13B	2.6	3.0	12FAc1	3.3	3.0			
Group 13C	2.7	3.0	12FAc2	3.3	3.0			

[w]wood walls [m]masonry walls [1]COLA-UCI *(2001)* [2]Salenikovich *et al. (2000)* [3]Shing *et al. (1989, 1990a,b, 1991)*
Note: All R-values are based on peak load

PARAMETER STUDY

While it is of importance that a structure exhibits high ductility to resist possible earthquake forces; other parameters, including hysteretic energy dissipation, resistance to degradation, inherent redundancy, load level, failure mode, *etc*, also play an important role and should be considered in the determination of force modification values for design. Furthermore, the calculated R-values listed in this paper were based on the measured peak load during a test without consideration of the subsequent degradation in load, and hence, post peak load behaviour of the shear walls has been overlooked. In an attempt to better understand the behaviour of laterally loaded shear walls, and to assess the appropriateness of the calculated R-values for cold-formed steel shear walls, a parameter study was completed. Comments on the various parameters that were taken into consideration are provided in the following sections and a listing of the characteristics that were compared is located in Table 3.

i. The Ratio of Displacements at Failure to that at Peak Load ($r = \Delta_{fail}/\Delta_{peak}$)

The ratio $\Delta_{fail}/\Delta_{peak}$ is an indication of the ability of a shear wall to limit the amount of load degradation with increasing lateral deflection. In a best-case scenario the capacity of a structure should be roughly maintained, with no sudden decrease, if earthquake energy is to be more efficiently absorbed. The failure load for wood structures and masonry structures is $0.8F_{peak}$ *(ISO, 1998)* and $0.9F_{peak}$ *(CSA S304, 1994)*, respectively. For this comparison, the failure load for cold-formed steel shear walls was assumed to be $0.9F_{peak}$. In terms of performance, the higher the r-value, the better the resistance to load degradation.

ii. Normalised Energy (E_{nor})

The dissipation of hysteretic energy during cyclic loading is an important attribute for a structure to possess if it is to survive an earthquake. Favourable energy dissipation characteristics enable a better seismic response and thus, support the assignment of higher R-values. The energy dissipated in one complete cycle is measured as the area enclosed by the storey shear *vs.* deflection curve, which can be obtained by carrying out a numerical integration of the recorded test results. The cumulative dissipated energy of all cycles up to the peak load cycle was calculated and normalised by the peak load in order that a comparison of the different tests could be made.

iii. Overstrength Factor (Ω_M)

Generally, design codes and standards prescribe a process with which one can determine the capacity of a shear wall with respect to the nominal strength (F_n), however, the maximum load that a structure can carry may be much higher. Uang *(1994b)* stated that the actual strength of the structure greatly contributes to its ability to survive severe earthquakes. It was also recommended that a balance between the strength and ductility requirements should be made to take advantage of the reserve strength when considering the assignment of an R-factor. The material over-strength factor, Ω_M, is defined as the ratio of the maximum strength the system can attain, F_{max}, to the nominal strength that is used in design:

$$\Omega_M = \frac{F_{\max}}{F_n}$$

iv. Failure Mode

Acceptable seismic performance requires a ductile failure mode without a rapid or a complete loss of load carrying capacity. Driver *et al. (2000)* state that a conservative value of R is suitable for a structure that fails in a nonductile fashion, whereas for a structure that exhibits a gradual degradation of load before final failure, the use in design of a more elevated R-value will still result in adequate performance during an earthquake. With respect to masonry walls, where R = 1.5 in the NBCC, flexural failure of the wall is expected with yielding of the tension reinforcement. In contrast, the unreinforced masonry walls that must be designed with R = 1.0 fail in a brittle shear mode. As recorded during testing, the steel stud shear walls failed when one of the following took place: screws pulled through the wood sheathing, studs buckled, screws pulled out of the studs and/or tracks, screws sheared, tracks pulled out of the plane, *etc (Serrette et al., 1996b, 1997b; COLA-UCI, 2001)*. For wood walls, the failure modes that were most frequently observed were: nails failed in fatigue, nails pulled out of wood studs and/or through the panels, nails tore through the sheathing edge, and combinations of these modes *(Salenikovich et al., 2000; COLA-UCI, 2001)*. In general, the loss of load-carrying capacity is similar in steel stud and masonry walls (except for those with R = 2.0), and typically occurs at a lower lateral displacement from that measured for wood stud walls *(Zhao, 2002)*.

Results of Parameter Study

The parameters discussed above were determined based on the results of shear wall tests completed by Serrette *et al. (1996b, 1997b)*, COLA-UCI *(2001)*, Salenikovich *et al. (2000)*, and Shing *et al. (1989, 1990a,b, 1991)*. As shown in Table 3, wood walls with R = 3.0 have similar r-values to wood walls with R = 2.0 (*i.e.* walls with gypsum sheathing), because these test walls were constructed with a combination of wood and gypsum sheathing, which increased the lateral stiffness and also, decreased the displacement at peak load. Masonry walls are significantly stiffer than wood and steel walls; thus, the measured r-values are high even though the shear resistance diminished rapidly after the peak load was reached. Steel stud walls generally have lower r-values than wood walls, which is an indication that the steel walls do not have the same capacity to resist shear loads in the post peak range.

Walls with higher measured R-values tend to have an increased ability to absorb energy, hence, possess elevated normalised energy values (E_{nor}). For example, the normalised energy values of the wood shear walls with R = 3.0 are in the range of 20 lbs in./lbs (except 04Fac-1), and those of the wood shear walls with R = 2.0 are noticeably lower (≈ 11 lbs in./lbs) (except Group 34B). The E_{nor} values for the masonry shear walls were determined for the 50% degradation post peak load position, rather than at the peak load, which would provide a slight advantage in terms of energy dissipation. However, in comparison with the steel stud and wood shear walls, the normalised energy values are dramatically lower, especially for the unreinforced walls with R = 1.0. The steel wall normalised energy values are in the range of 8 lb in./lb for the specimens tested by Serrette *et al. (1996b, 1997b)*, and approximately 14 lb in./lb for those tested by COLA-UCI *(2001)*. This discrepancy may have resulted from the use of different aspect ratios (height *vs.* length) for the test specimens in the two test programs. The steel stud shear wall tests with lower aspect ratios tended to display a better ability to dissipate energy. In general, the wood stud walls were able to dissipate the greatest amount of energy, followed by the steel stud walls, with the masonry walls showing only minimal energy absorption ability.

In terms of over-strength, the masonry shear walls have higher factors (Ω_m) than both steel and wood walls. This characteristic may aid in their ability to survive severe earthquakes. The over-strength-values for wood walls are stable (1.1 ~ 1.2), while those for steel walls are in the same overall range, although the results fluctuate to a larger degree.

Table 3: Parameters for Steel, Wood and Masonry Walls

Specimen	*r*	$E_{nor.}$	Ω_m	*Specimen*	*r*	E_{nor}	Ω_m	*Specimen*	*r*	$E_{nor.}$	Ω_m
R = 3.0				Grp. 34C[w,3]	1.29	11.0	1.15	AISI-A2[s,1]	1.12	7.5	1.08
Grp. 03A[w,3]	1.41	26.8	1.15	Spec. 8[m,4]	2.01	2.4	1.39	AISI-A3[s,1]	---	15.2	1.07
Grp. 03B[w,3]	1.68	16.1	1.16	**R = 1.5**				AISI-A4[s,1]	1.14	---	1.12
Grp. 03C[w,3]	1.24	25.0	1.19	Spec. 1[m,4]	1.26	1.36	1.38	AISI-A5[s,1]	1.07	5.4	1.08
Grp. 04A[w,3]	---	29.1	1.19	Spec. 2[m,4]	1.39	1.02	1.37	AISI-A6[s,1]	1.12	5.5	1.09
Grp. 04B[w,3]	1.28	24.3	1.14	Spec. 6[m,4]	2.73	3.58	1.65	AISI-A7[s,1]	1.04	---	1.11
Grp. 04C[w,3]	1.32	23.0	1.20	Spec. 10[m,4]	1.65	0.72	1.48	AISI-A8[s,1]	1.04	---	1.09
Grp. 09A[w,3]	1.51	25.5	1.13	Spec. 11[m,4]	1.44	0.58	1.46	AISI-B1[s,1]	1.26	---	1.13
Grp. 09B[w,3]	---	23.2	1.11	Spec. 12[m,4]	1.48	0.72	1.54	AISI-B2[s,1]	1.28	---	1.15
Grp. 09C[w,3]	1.46	19.4	1.10	Spec. 15[m,4]	1.43	0.91	1.52	AISI-B3[s,1]	1.04	---	1.21
Grp. 12A[w,3]	1.27	33.7	1.14	**R = 1.0**				AISI-B4[s,1]	1.16	---	1.14
Grp. 12B[w,3]	1.51	27.9	1.15	Spec. 3[m,4]	1.52	0.61	1.28	AISI-E1[s,1]	---	17.3	1.13
Grp. 12C[w,3]	1.43	22.7	1.14	Spec. 4[m,4]	1.27	0.43	1.22	AISI-E2[s,1]	---	---	1.10
Grp. 13A[w,3]	---	17.2	1.14	Spec. 5[m,4]	1.19	0.38	1.05	AISI-E3[s,1]	---	---	1.17
Grp. 13B[w,3]	1.44	32.5	1.14	Spec. 7[m,4]	1.09	0.39	1.17	AISI-E4[s,1]	---	---	1.12
Grp. 13C[w,3]	1.23	26.2	1.14	Spec. 9[m,4]	1.18	0.32	1.26	AISI-E5[s,1]	---	11.7	1.09
Grp. 23A[w,3]	1.38	34.5	1.16	Spec. 13[m,4]	1.63	0.55	1.25	AISI-E6[s,1]	1.08	11.5	1.13
Grp. 23B[w,3]	1.46	19.6	1.17	Spec. 14[m,4]	1.23	0.39	1.24	Grp. 14A[s,3]	1.20	12.6	1.25
Grp. 23C[w,3]	1.15	25.1	1.17	Spec. 16[m,4]	1.05	0.39	1.19	Grp. 14B[s,3]	1.28	13.1	1.23
Grp. 35A[w,3]	1.25	21.8	1.11	**R Undefined**				Grp. 14C[s,3]	1.23	14.6	1.23
Grp. 35B[w,3]	1.42	17.6	1.16	AISI-OSB1[s,1]	1.26	5.8	1.35	Grp. 15A[s,3]	1.33	14.6	1.20
Grp. 35C[w,3]	1.39	16.3	1.14	AISI-OSB2[s,1]	1.15	9.8	1.31	Grp. 15B[s,3]	1.22	14.3	1.21
Grp. 36A[w,3]	1.40	25.5	1.13	AISI-OSB3[s,1]	1.07	8.0	1.30	Grp. 15C[s,3]	1.21	13.7	1.21
Grp. 36B[w,3]	1.59	16.5	1.14	AISI-OSB4[s,1]	1.13	8.8	1.50	Grp. 16A[s,3]	1.11	14.7	1.14
Grp. 36C[w,3]	1.41	18.9	1.12	AISI-OSB5[s,1]	1.13	---	1.21	Grp. 16B[s,3]	---	11.7	1.23
04Fac-1[w,2]	1.82	9.7	1.10	AISI-OSB6[s,1]	1.05	---	1.54	Grp. 16C[s,3]	---	13.2	1.09
04Fac-2[w,2]	1.41	16.9	1.13	AISI-OSB7[s,1]	1.16	8.0	1.19	Grp. 17A[s,3]	1.22	12.2	1.25
08Fac-1[w,2]	1.45	15.4	1.15	AISI-OSB8[s,1]	1.03	---	1.15	Grp. 17B[s,3]	1.23	13.9	1.20
08Fac-2[w,2]	1.61	15.8	1.14	AISI-PLY1[s,1]	1.28	5.6	1.29	Grp. 17C[s,3]	1.12	13.1	1.23
12Fac-1[w,2]	1.47	16.1	1.15	AISI-PLY2[s,1]	1.06	8.5	1.25	Grp. 18A[s,3]	1.27	11.7	1.21
12Fac-2[w,2]	1.43	15.8	1.15	AISI-PLY3[s,1]	1.05	8.5	1.24	Grp. 18B[s,3]	1.26	11.9	1.21
R = 2.0				AISI-PLY4[s,1]	1.04	8.2	1.42	Grp. 18C[s,3]	---	13.4	1.18
Grp. 06A[w,3]	1.57	11.1	1.14	AISI-PLY5[s,1]	1.02	---	1.20	Grp.19A[s,3]	1.17	15.0	1.17
Grp. 06B[w,3]	1.36	11.8	1.14	AISI-PLY6[s,1]	---	---	1.20	Grp. 19B[s,3]	1.08	15.0	1.17
Grp. 06C[w,3]	1.32	8.8	1.14	AISI-PLY7[s,1]	1.06	---	1.16	Grp. 19C[s,3]	---	14.9	1.16
Grp. 34A[w,3]	1.39	12.5	1.15	AISI-PLY8[s,1]	1.04	---	1.21				
Grp. 34B[w,3]	1.26	24.4	1.15	AISI-A1[s,1]	---	14.3	1.10				

[w]wood walls [m]masonry walls [s]cold-formed steel walls

[1]Serrette *et al. (1996b, 1997b)* [2]Salenikovich *et al. (2000)* [3]COLA-UCI *(2001)* [4]Shing *et al. (1989, 1990a,b, 1991)*

Note: E_{nor} of wood and steel walls are based on P_{peak}, while those of masonry walls are based on 0.5 post P_{peak}

DISCUSSION OF ALTERNATIVE METHODS

Additional Method to Determine an Over-Strength Factor

The Applied Technology Council *(1995)* recommends the use of another method with which the over-strength factor can be evaluated. Once the backbone curve is plotted it is possible to calculate the base shear force, V_0, at the drift corresponding to the limiting state of response. The typical limiting responses include maximum inter-storey drift and maximum plastic hinge rotation. The design base shear force at the working stress level is specified by the 1985 UBC *(ICBO, 1985)* as:

$$V_D = (ZIKCS)\ W$$

The parameters Z and I are used to quantify the seismic zone and the importance of the building, respectively. The parameter S is used to account for site characteristics and C is a numerical period of vibration of the building and the defined spectral shape. K is a numerical coefficient referred to as the horizontal force factor, and W is the mass of the building.

The design base shear at the strength level (V_d) can be calculated by multiplying the working-stress design base shear V_w by a seismic load factor ($\approx$ 1.40). It follows that the over-strength factor can be calculated using the following expression:

$$\Omega = \frac{V_0}{V_d}$$

Other Methods to Determine an R-Value

It must also be noted that the R-values presented in this paper were evaluated from the results of quasi-static reversed cyclic single-storey shear wall tests, which may not necessarily correspond to the behaviour of a real building including inertia effects. The contributions of various boundary conditions, connections and non-structural components, gravity loads, *etc*, may impact on the distribution of seismic forces to the structural shear walls. Ceccotti and Karacabeyli *(2000)* presented a methodology to assess R-values, for which full-size shear wall specimens are tested under both static and cyclic load to determine the near-collapse criterion. A hysteretic model is then fit to the cyclic test data; the walls are designed for use in a selected building according to the code peak ground acceleration following various design scenarios; then a nonlinear dynamic analysis is carried out to obtain the ultimate peak ground acceleration for the different design scenarios. With this type of study the performance of a shear wall when subjected to seismic inertia loading can be predicted and more appropriate R-values may be selected. Shake-table tests of the shear walls would also need to be performed to verify the analytical conclusions. Ceccotti and Karacabeyli confirmed with this procedure that wood-stud shear walls sheathed with gypsum wallboard should be designed using an R-value of 2.0.

CONCLUSIONS

A procedure was presented to determine force modification factors for use in the design of lateral load-resisting systems based on the results of quasi-static cyclic shear wall tests. According to a comparison of calculated R-values, and other parameters of steel, wood, and masonry shear walls, at this time a preliminary R-value of 2.0 is suggested for use with the National Building Code of Canada in the design of single-storey cold-formed steel shear walls sheathed with wood panels. Further studies are necessary to evaluate the effects of aspect ratio and construction configuration, as well as the influence of dynamic forces, multiple storey walls, *etc.* A more advanced study that includes nonlinear dynamic analyses of different design scenarios and a comparison with additional test data must be carried out to confirm this suggested force modification factor. The expected displacement of the structure must also be adjusted accordingly if an R-value greater than 1.0 is used in design.

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