Seismic Screening, Evaluation, Rehabilitation, and Design Provisions for Wood-Framed Structures

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Abstract: Seismic screening, evaluation, and rehabilitation design of two existing wood-framed structures are performed using FEMA 154, FEMA 356, ASCE/SEI 31, and the 1997 UBC. FEMA 154 screening demonstrates the importance of examining construction drawings. Plans revealed a structural deficiency, but screening alone indicated no further evaluation is needed. Demand-to-capacity ratios for shear walls and roof diaphragms are examined. 1997 UBC design provisions for new buildings are not necessarily conservative compared to FEMA 356 rehabilitation guidelines. Also, FEMA 356 and 1997 UBC design provisions are not necessarily conservative compared to existing building evaluation in ASCE/SEI 31. A likely cause of the unexpected results is the conservative linear static procedure and associated m factors for wood in FEMA 356 and ASCE/SEI 31. Research is needed to better calibrate these factors and acceptance criteria to account for duration of shaking, number of cycles of nonlinear behavior, and redundancy.


CE Database subject headings: Seismic analysis; Seismic design; Wood structures; Shear walls; Diaphragms.

Introduction

Earthquakes cause widespread destruction and loss of life. Some areas are more prone to seismic activity, but earthquakes are a national hazard. Regions previously considered relatively safe from earthquakes actually have potential for extensive damage, such as the Pacific Northwest, with evidence of large historic earthquakes on the Cascadia subduction zone. With education, seismic rehabilitation may increase, as public facilities are evaluated and upgraded, and owners voluntarily strengthen.

New building codes typically focus on life safety in design seismic events. Life safety involves structural damage, but a margin against collapse (FEMA 2000). The design seismic event has evolved, however, and buildings designed to previous codes may not meet current requirements. Other changes have taken place in construction methods, materials, and workmanship.

In this paper, seismic performances of two adjacent wood-framed structures are compared using several methods. One structure was designated a disaster shelter, and is thus a good candidate for seismic evaluation and upgrade. Approaches compared include: FEMA 154, “Rapid visual screening of buildings for potential seismic hazards” (ATC 2002); FEMA 356, “Prestandard and commentary for the seismic rehabilitation of buildings” (FEMA 2000); ASCE/SEI 31, “Seismic evaluation of existing buildings” (ASCE 2003); and 1997 Uniform Building Code (ICBO1997).

Literature Review

Much information is available on seismic research and design, but few comparisons of analysis, evaluation, and design results are found. Model building codes (SBC, BOCA, UBC, and IBC) for new construction are comprehensive, trusted for life-safety performance, and adopted by local authorities. A level of comfort derives from observed performance, experience with design methods, and research. Seismic evaluation and rehabilitation documents are not as widely accepted, are newer, and often guidelines. This paper focuses on seismic screening, rehabilitation, evaluation, and design methods.

Screening: FEMA 154

FEMA 154, “Rapid visual screening of buildings for potential seismic hazards: A handbook” (ATC 2002) identifies structures likely to need rehabilitation. Rapid visual screening (RVS) involves no structural analysis, but basic scores and modifiers from external, visual inspection (15–30 min/building). Lateral-load-resisting system is identified, and basic structural hazard score assigned. Modifications yield a final structural score, S, with higher scores for better expected performance. Building inventories can be readily assessed, with those having an S of 2 or less needing further investigation.

Rehabilitation: FEMA 356

Seismic evaluation may show rehabilitation is needed. FEMA 356, “Prestandard and commentary for the seismic rehabilitation of buildings” (FEMA 2000) gives forces and acceptance criteria
for rehabilitation of identified deficiencies to meet demands at target performance levels. Model codes target life safety performance, but higher or lower levels may be selected in FEMA 356.

**Evaluation: ASCE/SEI 31**

ASCE/SEI 31, “Seismic evaluation of buildings” (ASCE 2003) is a three-tiered process. Evaluation may be indicated by FEMA 154, and progresses from checklist-style (Tier 1) to detailed evaluations of systems (Tiers 2 and 3). Evaluations are at immediate occupancy (IO) and life safety (LS) performance. Checklists are available for “wood light frames” and “wood frames—commercial and industrial.” Tier 1 examines “benchmark building” criteria, deficiencies summarized and evaluated in Tier 2. When more analysis of deficiencies in Tier 2 is needed, Tier 3 uses provisions for rehabilitation or new buildings. Seventy percent of design forces is used, as appropriate for evaluating existing buildings (ASCE 2003).

**Design: 1997 UBC**

The Uniform Building Code (ICBO 1997) was the primary design code in the Pacific Northwest at the time of the study (Baxter 2004), and is compared here with rehabilitation and evaluation documents, although evaluating existing buildings is not a primary intent of the UBC. Wood design in UBC 1997 uses working-stress force levels and FEMA 356 and ASCE/SEI 31 use ultimate strength so direct comparison of forces cannot be made, and comparison of performances is presented. The 1997 UBC targets life safety performance, and to increase performance for essential facilities, it increases design forces using an importance factor, I. For typical structures I = 1.0, but for essential facilities I = 1.5. For wood structures, seismic forces are computed at strength level, and divided by 1.4 for working stresses. Allowable stresses are also at working stress levels. Alternate load combinations are used here for allowable stress design, and the 33% increase in allowable stress is taken for short-term loads.

**Procedure Comparisons**

There is little published comparing design results from existing codes and handbooks. McIntosh and Pezeshk (1997) compare previous editions of the NEHRP specification and UBC. No comparisons were found using the methods in this paper. Anecdotal design evidence shows that “n” factors in linear static procedures (LSPs) of FEMA 356 may be overly conservative compared to the 1997 UBC for reinforced concrete shear walls and other systems. Poland (private communication, Dec. 12, 2003) points to the conservative LSP provisions. UBC R factors are calibrated to produce forces designers are comfortable with, based on postearthquake observations of damage. The m factors are derived and do not include duration of shaking and number of cycles of nonlinear behavior.

**Structure Description**

The structures are portions of a school built in the mid-1960s. Classrooms and gymnasiums are separate structurally, connected by hallways. Structural walls are wood frame, with a plywood roof. Classrooms [Fig. 1(a)] have interior gypsum or wood panel bearing walls. Exterior walls are nominal 51 × 152 mm (2 × 6) vertical studs, with batt insulation, and channeled vertical wood siding, 19–38 mm in thickness. Walls are blocked at 0.91 m for siding edge nailing, with diagonal wood let-in bracing. The roof has open web trusses 0.81 m deep at midspan, spaced at 1.22 m. Glulam beams support the upper roof and span window openings. The gymnasium [Fig. 1(b)] has a 6.10 m high ceiling. The roof is framed with wood trusses at 1.83 m, and nominal 51 × 102 mm (2 × 4) purlins at 0.61 m. Exterior walls are nominal 51 × 152 mm (2 × 6) vertical studs with gypsum board on the interior and wood siding on the exterior.

The classroom area is rectangular in plan (24.4 m N-S × 27.4 m E-W), with low ceilings and significant openings in two opposite walls. For seismic loads in the east/west direction, doors and windows permit analysis of separate “high aspect ratio” wall piers. In the north/south direction there are no openings. The gym is also rectangular (29.3 m N-S × 35.4 m E-W), with high ceilings, doors, and window openings.

Rehabilitation using the 1997 UBC showed that one layer of 13 mm C-D grade plywood on walls and roofs is sufficient. For comparison of analysis, evaluation, rehabilitation, and design provisions, buildings are assumed completely sheathed with one layer of 13 mm C-D plywood. Ratios of element forces divided by capacities are compared. A single layer of 13 mm plywood is inadequate for all methods, so analysis with two layers of 13 mm plywood wall sheathing and additional roof sheathing is also performed. A double-sheathed wall has a single layer of plywood sheathing attached to the inside and outside faces.

**Analysis Assumptions and Methods**

**FEMA 154**

The school is in a zone of high seismicity. Classrooms are 670 m² and the gym is 1,035 m², so each is W2 since wood-framed and greater than 465 m². Basic structural scores are based on probability of collapse given the maximum considered earthquake. For W2 in high seismicity, the basic score is 3.8. For these one-story structures, soil type D is assumed as a default, and a modifier equal to −0.8 is applied.

**FEMA 356**

Before using FEMA 356, a structure must already be shown to need rehabilitation. Evaluation using ASCE/SEI 31 shows the structures are noncompliant, requiring rehabilitation. FEMA 356 defines performance levels including immediate occupancy, life safety, and collapse prevention. Immediate occupancy is where the building is safe to occupy, and retains preearthquake strength and stiffness. Minor structural repairs may be needed. Life safety performance includes significant damage to
structural components but a margin against collapse. Risk of life-threatening injury is low. While possible to repair, it may be impractical (FEMA 2000). Collapse prevention is where the structure continues to carry gravity loads, but is on the verge of collapse.

Seismic Hazard
Two percent/50 year [for the maximum considered earthquake (MCE)] and 10%/50 year maps plot spectral accelerations for 2 and 10% probabilities of exceedance, respectively, in a 50-year period, for the short-period response parameter, $S_S$, and $S_1$, at the 1-s period. Results for the site are $S_S=0.43g$ for 10%/50 years and 1.08g for 2%/50 years; $S_1=0.20g$ for 10%/50 years and 0.47g for 2%/50 years.

Two earthquake hazard levels are defined: basic safety earthquakes 1 (BSE-1) and 2 (BSE-2). BSE-1 uses the lesser of 10%/50 year spectral accelerations, modified for site class, or two-thirds of MCE spectral accelerations, modified for site class. BSE-2 uses MCE spectral accelerations, modified for site class. BSE-1 and BSE-2 response spectra are shown in Fig. 2, assuming 5% effective viscous damping.

Rehabilitation Objectives
Rehabilitation objectives include earthquake hazard level and target building performance. The basic safety objective (BSO) is life safety performance for BSE-1 and collapse prevention for BSE-2 (FEMA 2000). Target performance for the gym is immediate occupancy at BSE-1, an enhanced objective, and collapse prevention at BSE-2. The rehabilitation objective for the classrooms is the BSO.

As-Built Information
A site visit checked construction drawings, connections, and structural systems. Major elements were in good condition. Knowledge factor, $\kappa$, accounts for uncertainty in as-built information. With an enhanced objective for the gym knowledge must qualify as usual or comprehensive based on testing. With drawings, wood strengths were verified by grade stamps for comprehensive testing (FEMA 2000). Thus, $\kappa$, a multiplier on capacity, equals 1.0. If grade stamps are not observed, $\kappa=0.75$.

Systematic Rehabilitation
Systematic rehabilitation includes identification of deficiencies, rehabilitation strategy, and analysis procedure. A lateral-force-resisting system is vertical stud walls with nominal 25 $\times$ 152 mm (1 $\times$ 6) diagonal let-in wood bracing. The 1994 UBC disallowed this in seismic zones 2B and higher (ICBO 1994). FEMA 356 provides no properties. Horizontal lumber sheathing and diagonal let-in bracing are discussed, but braces fail at low loads. The 1971 San Fernando earthquake showed the inadequacy of diagonal let-in braces. Many failed in tension or pulled from the bottom plate. Lateral-load resistance is assumed zero (Breyer 1993). Classroom diaphragms have edges detailed as blocked only. Typical diaphragms have a continuous edge member or “chord” to develop in-plane moment. Stiffnesses for “unchorded” diaphragms are approximately half that of “chorded” ones. Thus, continuous steel strap chords are assumed in the rehabilitation.

Rehabilitation Strategy and Analysis Procedure
Global stiffening and strengthening add structural elements to increase building stiffness or strength. New shear walls are appropriate and necessary with the flexible, weak let-in bracing in place.

The LSP is selected based on building regularity and fundamental period. Lateral seismic load or base shear, $V$(kN) is applied to the structure proportional to its weight

$$V = C_1 \cdot C_2 \cdot C_3 \cdot C_m \cdot S_m \cdot W$$

where $C_1$=inelastic displacement modification factor; $C_2$=modification factor for pinched hysteresis, stiffness and strength deterioration; $C_3=P-\Delta$ modification factor; $C_m$=higher mode mass participation factor; $S_1$=response spectrum acceleration, (g’s); and $W$=effective seismic weight (kN).

Base shears are calculated for BSE-1 and BSE-2 for each structure as shown in Table 1. These are distributed to shear walls and diaphragms in the three-dimensional (3D) models as percentages of self-weight, applied laterally. Wall and diaphragm shears are checked against acceptance criteria. For comparison with other rehabilitation, evaluation, and design documents, all shear wall elements are assumed rehabilitated.

Horizontal torsion must be included for buildings without flexible diaphragms. A flexible diaphragm is where horizontal, in-plane deflection is at least twice the lateral deflection of the story below. A diaphragm is rigid if its deflection is less than half the lateral story deflection, stiff otherwise. 3D models with a single layer of diaphragm sheathing show the classroom diaphragm is rigid and gym diaphragm is stiff, eliminating 2D analysis for both. 3D finite-element models account for actual torsion with 5% accidental torsion added.

Nearly all structural elements affect stiffness and response to lateral loads. Primary elements resist collapse in seismic events. In linear analysis, only primary elements are included. Here, diaphragms and shear walls are primary. The existing “let-in” bracing system is very weak, so only stiffnesses and strengths for new shear wall elements and existing or new roof diaphragms are used in the model.

Shear Wall Element Formulation
The equation for deflection of wood structural panel sheathing (or shear wall)

$$\Delta_y = \frac{667 \cdot v_y \cdot h^3}{E \cdot A \cdot b} + \frac{v_y \cdot h}{G \cdot I_1} + 2.46 \cdot h \cdot e + \left(\frac{h}{b}\right) d$$

Table 1. FEMA 356 Base Shears

<table>
<thead>
<tr>
<th>Structure</th>
<th>BSE-1 V</th>
<th>BSE-2 V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classrooms</td>
<td>0.816 W(LS)</td>
<td>1.497 W(CP)</td>
</tr>
<tr>
<td>Gymnasium</td>
<td>0.651 W(IO)</td>
<td>1.184 W(CP)</td>
</tr>
</tbody>
</table>
where $\Delta_y$=deflection of panel at yield (mm); $v_y$=shear at yield (kN/m); $h$=shear wall height (m); $E$=modulus of elasticity of boundary member (GPa); $A$=cross section of boundary member (mm$^2$); $b$=shear wall width (m); $G$=modulus of rigidity of wood structural panel (GPa); $t$=effective thickness of wood structural panel (mm); $d_y$=deflection at yield of tie-down anchorage or deflection at load level to anchorage at end of wall (mm); and $e_n$=nail deformation at yield load per nail (mm).

Yield strengths are LRFD-based, nominal strengths with $\varphi=1.0$, multiplied by 0.8 for plywood. New shear walls are 13 mm C-D plywood with 8d common nails at 152 on-center edge nailing. Factored shear resistance for 12 mm sheathing is 4.96 kN/m, with $\varphi=0.65$ (BSSC 2001). Classroom shear wall height is $h=2.74$ m, and for gym shear walls $h=6.50$ m. Boundary elements at ends of shear panels are nominal 51 $\times$ 152 mm (2 $\times$ 6) studs, with assumed $E=11.0$ GPa. The Plywood Design Specification (APA 1997) provides a modulus of rigidity, $G=0.62$ GPa, for Douglas-fir in species Group 1, rated APA C-D plugged, with exterior glue specified. This is grade stress Level 2, and panels are assumed dry. Simpson PHD2-SDS3 tie-down anchors are at the ends of new wall panels. Similar to a lag-bolt connection, default deformation at yield of $d_y=2.5$ mm is used. Nail deformation for 8d nails is $e_n=2.0$ mm for Structural I grade panels, multiplied by 1.2 for the C-D panel.

There is a nonlinear relationship between panel deformation and width, so trial calculations and finite-element models calibrated calculated deflection to modeled behavior. Models used 0.30 $\times$ 0.30 m plate elements. For a given width $b$, $E$ was varied until predicted deflection in Eq. (2) was achieved. Calibrated $E$ values show linear variation of stiffness versus wall length.

**Roof Diaphragm Element Formulation**

Deflection of blocked and chocked wood structural diaphragms with constant nailing along the length is given by

$$\Delta_y = \frac{52 \cdot v_y \cdot h^3}{E \cdot A \cdot b} + \frac{v_y \cdot L}{4 \cdot G \cdot t} + 0.62 \cdot L \cdot e_n + \frac{\sum (\Delta_x \cdot X)}{2 \cdot b}$$

(3)

where $\Delta_y$=yield deflection of diaphragm (mm); $A$=cross section of diaphragm chords (mm$^2$); $b$=diaphragm width (m); $E$ =modulus of elasticity of chords (GPa); $e_n$=nail deformation at yield per nail (mm); $G$=modulus of rigidity of wood structural panel (GPa); $L$=diaphragm span, distance between shear walls, or collectors (m); $t$=effective thickness of wood structural panel (mm); $v_y$=shear at yield (kN/m); and $\sum (\Delta_x \cdot X)$=sum of chordsplice slip values (mm) on both sides of diaphragm, each multiplied by its distance to nearest support (m).

Parameters in Eq. (3) are the same as in Eq. (2), except as noted. Diaphragm deflection is at the centerline of a panel with edges parallel to the load pin supported. The diaphragm chord is a 14-gauge $\times$ 76 mm steel strap at classrooms, and wood framing elements at other locations. Chord splice-slip is assumed zero. Factored shear resistance of diaphragms is taken from BSSC (2001). Panels are exterior C-D grade. Edge nailing is 10d at 4 $\times$ 102 mm for 19 mm diaphragms, and 10d at 152 mm for 16 mm diaphragms. Factored shear resistance for 19 mm diaphragms is 12.3 kN/m and factored shear resistance for 16 mm diaphragms is 6.13 kN/m. Shear yield force is equal to factored shear resistance divided by 0.65.

Each diaphragm was modeled and stiffness adjusted until the calculated deflection was achieved. Diaphragms are pinned-connected at walls. Each model is half the roof width for a given direction of load, with symmetry at the center of the diaphragm.

Nodes along that line are restrained against rotation about the “$z$” or “outward” axis, and translate in the direction of load. Shear $v_y$ is a series of point loads along the diaphragm centerline, to induce an equivalent uniform shear at the supported edge. “$E$” is adjusted until lateral deflection matches Eq. (3). Equivalent “$E$” for each diaphragm is different for each load direction as “$L$” and “$b$” reverse. Average “$E$” values for the two directions are used for diaphragms in 3D models.

**Finite-Element Model**

2D analysis is permitted for flexible diaphragms. Model displacements confirm the classroom diaphragm is rigid and gym diaphragm is stiff, so 2D analysis is not appropriate. Each building is modeled using plate elements for walls and roof. Material properties are assigned based on test modeling for material “$E$” stiffness. An average softwood Poisson’s ratio of 0.40 is used (Bodig and Jayne 1993). For models with a single layer of roof sheathing, roof plate elements are 19 mm thick at classrooms and 16 mm thick at the gym. Where double sheathing is used, total dead load increases, and element thickness is doubled. For models with a single layer of wall sheathing, wall plate elements are 13 mm thick.

**Verify Rehabilitation Design**

A typical “design” iterates wall panel lengths (with appropriate material properties) until panel and diaphragm capacities are not exceeded. To compare methods, all wall panels are upgraded. Shear force is compared with capacity in a demand-to-capacity ratio (DCR). DCR greater than 1.0 is not acceptable.

Primary components analyzed using linear procedures are deformation controlled or force controlled. Wood structures and connections are often ductile, and shear walls and diaphragms may deform well beyond the elastic limit and still carry gravity loads. Thus, they are deformation controlled. Component ductility is accounted for using an $m$ factor, based on material, element type, component classification, dimensions, and performance level. Shear wall factors are for “wood structural panel sheathing or siding.” Diaphragm factors are for “wood structural panel, blocked, chored.”

The acceptance criterion for deformation-controlled actions in linear procedures is

$$m \cdot k \cdot Q_{CE} \geq Q_{UD}$$

(4)

where $m$=component demand modifier for ductility; $Q_{CE}$=expected strength of component at deformation level under consideration (kN); $Q_{UD}$=deformation-controlled design action (kN); and $k$=knowledge factor. Deformation-controlled design action is shear for walls and roofs.
In a typical check, dimensions of shear wall or roof elements are increased until the acceptance criterion is met. Here, walls are assumed sheathed with one or two layers of 13 mm C-D plywood, and roof diaphragms assumed single or double sheathed with plywood. $Q_{UD}$ divided by $(m^* \kappa Q_{CE})$ is the DCR.

**ASCE/SEI 31**

Construction drawings were available and checked, and site visits showed the general condition of the structure to be very good. Desired performance must be defined prior to evaluation. Due to the emergency shelter designation, the gym is assigned IO performance. Classrooms are at a LS level. Structures are wood frames (W2), floor area greater than 465 m$^2$.

Two percent/50 year (for the MCE) and 10% /50 year maps plot spectral accelerations for 2 and 10% probabilities of exceedance, respectively, in a 50-year period, for the short-period response parameter, $S_q$, and $S_1$, at a 1-s period.

Spectral accelerations for the maximum considered earthquake $S_2$ (0.2-s period) and $S_1$ (1-s period) are equal to 1.083g and 0.469g, respectively. $S_{DS}$ and $S_{DS}$ are design spectral accelerations modified for soil properties

$$S_{DS} = \left( \frac{2}{3} \right) \cdot F_p \cdot S_1$$

$$S_{DS} = \left( \frac{2}{3} \right) \cdot F_p \cdot S_2$$

The 2/3 provides 50% margin of safety between the force causing loss of the first primary element and that causing collapse (ASCE 2003). Site coefficients are functions of mapped MCE accelerations and site class. Where insufficient soil information is known, site class D is assumed. Interpolation gives $F_p=1.530$ and $F_p=1.068$. Associated spectral accelerations are $S_{DS}=0.479g$ and $S_{DS}=0.769g$. Both parameters designate the level of seismicity as high.

**Screening Phase (Tier 1)—Existing Structure**

Construction drawings are dated 1967, so the design was likely to be the 1964 edition of the UBC. 1976 is the benchmark year for the UBC and building type W2. Thus, the school does not meet benchmark criteria and tier 1 evaluation is needed. In addition, buildings designed to the UBC but evaluated for IO performance are disqualified from benchmark status as UBC designs to the lower, life safety level.

“Let-in” bracing is an inadequate system for resisting lateral loads, and does not meet the shear stress check. New wood shear panels are added, requiring hold-down anchors. Roof diaphragm elements lack a continuous chord, so a continuous steel nailer strap at the edge of the plywood diaphragm is added.

**Evaluation Phase (Tier 2)—Existing Structure**

For all single-story, wood-framed structures a full-building Tier 2 analysis is not required. Thus, only noncompliant Tier 1 statements must be evaluated in Tier 2. Although all Tier 1 statements for the rehabilitated structures are in compliance, Tier 2 is included for comparison with other methods. Since no discontinuities exist, and the building is wood frame, the LSP is used for Tier 2. In Tier 2, load path and shear stress check again require rehabilitation as no viable shear walls are part of the existing structure.

**Evaluation Phase (Tier 2)—Rehabilitated Structure**

Tier 2 LSP assumes addition of structural panel sheathing to create viable shear walls. 2D analysis is allowed for flexible dia-

$$V = \frac{C_v}{R} \cdot \frac{I}{T} \cdot W$$

where $F_p$=total roof diaphragm force (kN); $C=$modification factor=1.3 for wood structures; and $F_l$=lateral force applied at roof level (kN). Lateral force at roof level equals pseudolateral force for the roof plus pseudolateral force for the upper half of the two walls perpendicular to the loading.

Acceptance criteria are based on component classification as deformation controlled or force controlled. Shear in wood shear walls and diaphragms are deformation-controlled actions. Gravity load effects are ignored. For deformation-controlled actions, the acceptance criterion is

$$Q_{CE} = \frac{Q_{UD}}{m}$$

where $Q_{CE}$=expected strength of component=1.25$^*$nominal strength (kN); $Q_{UD}$=load effect from seismic force (kN); and $m$=component demand modifier for ductility.

Nominal strength for shear walls is taken from the BSSC (2001). New shear walls are assumed to be 13 mm C-D plywood with $8d$ at 152 mm on center edge nailing ($i=13$ mm). Factored shear resistance is 4.96 kN/m, with $\varphi=0.65$. Diaphragm sheathing is exterior C-D grade (not Structural 1), and assumed edge nailing is 10d at 102 mm for 19 mm diaphragms, and 10d at 152 mm for 16 mm diaphragms. Factored shear resistance for 19 mm diaphragms is 12.26 kN/m, and 6.13 kN/m for 16 mm.

For comparison with other methods, DCR is calculated for each panel and diaphragm. DCR equals the required length of shear panel divided by actual length. If the resulting DCR is greater than 1.0, panel length is inadequate. All available panels are sheathed.

**Detailed (Tier 3) Evaluation—Rehabilitated Structure**

Tier 3 evaluation is performed according to FEMA 356, except forces are multiplied by 0.75. Thus, the Tier 3 DCR for each component is equal to 0.75 times the DCR from the FEMA 356 analysis.

**1997 Uniform Building Code**

The static force procedure is used. Total design lateral force (at ultimate strength level) is
where $V$=design lateral force (kN); $C_s$, $C_r$=seismic response coefficients; $I$=importance factor; $R$=overstrength and ductility factor; $T$=fundamental building period (s), and $W$=seismic dead load (kN).

Seismic response coefficients are based on seismic zone, $Z$, and soil type. The school is in seismic Zone 3. Default soil profile type $S_D$ is used. The seismic response coefficients are $C_s=0.36$ and $C_r=0.54$.

The local Red Cross has designated the gym as an emergency shelter, so $I=1.25$. Classrooms have $I=1.00$. For the bearing wall system with light-framed shear walls, three stories or less, $R=5.5$.

Fundamental building period, $T$, (s) is

$$T = C_t \cdot (h_n)^{3/4}$$

where $C_t=0.0488$ for wood structures; and $h_n$=height to roof (m).

Using $h_n=2.74$ m for classrooms and 6.50 m for the gym, fundamental periods are $T=0.104$ s for classrooms and 0.199 s for the gym. Total design lateral force=$0.164W$ for the classrooms and 0.205W for the gymnasium.

The earthquake load, $E$, for design is

$$E = \rho \cdot E_b + E_v$$

where $E$=earthquake load (kN); $\rho$=reliability/redundancy factor; $E_b$=earthquake load due to base shear, $V$ (kN); and $E_v$=load effect due to vertical component of ground motion (kN). The vertical seismic load effect may be taken as zero for allowable stress level forces.

The reliability/redundancy factor is

$$\rho = 2 - \frac{6.1}{r_{max} \cdot \sqrt{A_B}}$$

where $\rho$=reliability/redundancy factor, not less than 1.0 nor greater than 1.5; $r_{max}$=maximum element-story shear ratio; and $A_B$=ground floor area ($m^2$). For shear walls, element-story shear ratio is the maximum product of wall shear multiplied by 10/$l_w$, where $l_w$=wall length (m). For flexible diaphragms, wall shear is equal to total diaphragm shear multiplied by the ratio of panel length divided by the sum of all panel lengths. Substitution reduces $r$ to 10 divided by the sum of all panel lengths (in m). For the school, $\rho=1.0$ and $E=E_b$. To reduce strength-level forces to allowable stress level, divide design loads $E$ by 1.4.

Seismic weight for diaphragm shear is the weight of the roof plus the top half of each wall and panel perpendicular to the direction of seismic load, multiplied by 0.164 for classrooms and 0.205 for the gym. The diaphragms do not qualify as flexible, so diaphragm shear force is distributed to each shear panel based on relative rigidities. To account for accidental eccentricity, diaphragm loads increase 10%.

Each shear wall panel also resists its own in-plane seismic load. When added to the shear load from the roof diaphragm, the total load represents the in-plane shear on the panel. This total load is compared to the allowable panel shear. Allowable shear load for a horizontal wood structural panel diaphragm or wood structural panel shear wall depends on panel grade, nail size, width of framing members, edge nail spacing, and panel blocking. Classroom roof sheathing is assumed to be Grade C-D, 19 mm plywood with 10$d$ at 102 mm edge nailing. The thickest panel listed is 15 mm, so the allowable diaphragm shear is 6.20 kN/m. Gymnasium roof sheathing is assumed to be Grade C-D, 16 mm plywood with 10$d$ at 152 mm edge nailing. The value for 15 mm is used, and allowable diaphragm shear is 4.67 kN/m. New wall sheathing is 13 mm plywood. Grade C-D, nailed with 8$d$ at 152 mm, and allowable panel shear is 3.79 kN/m. Where “double panel” (with plywood inside and outside) calculations are performed, allowable shear forces are doubled.

DCR equals the shear in each element (at allowable stress force level), divided by the allowable shear.

**Results and Discussion**

**FEMA 154**

For the school, the final structural score $S=(3.8−0.8)=3.0$, is above the cutoff value and adequate based on RVS. Review of drawings, however, shows the shear walls were constructed with diagonal “let-in” bracing. These perform poorly in seismic events, and are not a viable lateral-force-resisting system (Breyer 1993). Thus, a vertical irregularity modifier $−2.0$ is applied, resulting in a final score of $(3.8−2.0−0.8)=1.0$. Since this is below the cutoff value 2.0, detailed evaluation is needed.

**FEMA 356**

DCRs for classroom and gymnasium diaphragms are shown in Tables 2 and 3, respectively. For each configuration and load direction, the higher DCR represents the controlling performance level. For example, the gymnasium single layer configuration loaded in the east/west direction with $\kappa=0.75$ has an immediate occupancy level DCR of 1.16, and a collapse prevention DCR of 0.79. The higher immediate occupancy DCR controls the design check in the east/west direction. Because the controlling DCR is greater than 1.0, the diaphragm is inadequate and a different rehabilitation method would be required.

DCRs are inversely proportional to the knowledge factor, $\kappa$. If a higher degree of knowledge of materials and construction methods is attained, DCRs are reduced. For the classroom diaphragm, the existing configuration meets the acceptance criteria using
κ = 0.75 or 1.0. Thus, there would be no benefit in conducting testing or demolition of roofing to view sheathing grade stamps. Increasing \( \kappa \) is not required, because even at \( \kappa = 0.75 \), the existing classroom diaphragm is acceptable.

At the gym, the existing diaphragm is not acceptable for \( \kappa = 0.75 \) because DCRs for east/west and north/south load directions both exceed 1.0. If plywood grade was observed for \( \kappa = 1.0 \), then the existing, single layer would be acceptable. Otherwise, rehabilitation adding a second layer of sheathing is needed.

At the classroom roof, collapse prevention DCRs are higher than corresponding life safety DCRs, and control. Conversely, immediate occupancy DCRs at the gymnasium diaphragm are all higher than collapse prevention DCRs. Thus, immediate occupancy performance and associated acceptance criteria control the design of the gym diaphragm. Several factors affect design forces, such as building period, sheathing thickness, and panel aspect ratios. The most significant effect is from the very low \( m \) factors used with the immediate occupancy level, compared to those for collapse prevention and life safety.

DCRs for classroom panels resisting east/west seismic loads are shown in Table 4. Existing panels loaded in the east/west direction fail to meet the collapse prevention (CP) performance for both \( \kappa = 0.75 \) and 1.0. Actually only the longest classroom panels (24.4 m) with north/south loads have DCRs that meet LS and CP performance using a single layer of sheathing. Thus, single-panel rehabilitation is not acceptable for the shorter classroom shear panels. Installation of double layer sheathing results in DCRs less than 1.0, and is needed.

At the gym, single-sheathed panels do not meet the acceptance criteria (Table 5). For \( \kappa = 1.0 \), for new sheathing panels, only longer panels meet the acceptance criteria with two layers. Double sheathing alone would not be acceptable at other locations.

For a typical rehabilitation, the design would be modified until DCRs were below 1.0. Some methods include increasing thickness of plywood, increased nailing, additional interior shear walls or other bracing, reducing performance level, or a combination of these. For comparison with other analysis methods, only \( \kappa = 1.0 \) is used for an actual rehabilitation where grade stamps would be observed. The element DCR for comparison with results from other methods is the higher of the two performance levels.

**ASCE/SEI 31**

**Tier 1**

Tier 1 checks only the wall panels for each seismic load direction. No checks are made for acceptability of the diaphragm. Tier 1 shows that shear force in each wall panel (classrooms and gymnasium) is acceptable as all DCRs are less than 1.0 (0.23–0.76).

**Tier 2**

Tier 2 involves evaluation of diaphragms and wall panels for each seismic load direction. As all DCRs are less than 1.0 (0.08–0.52), existing, single-layer configurations do not require rehabilitation.

DCRs (0.35–0.63) indicate all classroom shear walls meet the acceptance criteria for a single layer of sheathing. At the gym, a
single layer of sheathing would not be adequate, but a double layer meets acceptance criteria. Higher DCR values at the gymnasium are largely due to decreased m factors used for the immediate occupancy level. Factors are approximately half those used for life safety at the classrooms.

**Tier 3**

ASCE/SEI 31 Tier 3 uses the same methods as FEMA 356, with forces multiplied by 0.75. Thus, Tier 3 DCRs equal 0.75 times the associated DCRs from FEMA 356. Linear procedures are used, and linear element behavior assumed. Results show each existing diaphragm as acceptable regardless of knowledge level used (0.75 or 1.0); all DCR values are less than 1.0. Comparing DCR values for the classroom roof, the collapse prevention level controls because DCRs are higher than life safety DCRs. At the gym, immediate occupancy DCRs are higher than collapse prevention DCRs, so immediate occupancy controls.

Classroom wall panels do not meet acceptance criteria for single-layer sheathing with the reduced knowledge factor. Panels do meet acceptance criteria with double sheathing. For the gym, none of the single-layer panels meet the acceptance criterion for \( \kappa = 0.75 \), and only 13.7 and 16.8 m panels meet the single-layer acceptance criteria with \( \kappa = 1.0 \). Rehabilitation of either structure involves addition of new plywood sheathing, so the higher knowledge factor is applicable. Gym panels longer than 4.9 m meet acceptance criteria for double sheathing, but shorter ones do not. Shorter panels have lower m values, reducing capacity and increasing DCRs. For comparison with other methods, only \( \kappa = 1.0 \) is used.

### 1997 UBC

DCRs for classroom and gymnasium roof diaphragms are all less than 1.0, so the existing, single-layer configuration of roof sheathing is acceptable. DCRs for classroom and gym single-layer wall panels are also all less than 1.0, indicating that a single layer of (1/2) in. plywood sheathing is acceptable.

### Comparison of Methods

DCR for each element, and for each method of analysis or evaluation, should be compared to the expected level of conservatism inherent in the method. For example, design methods (1997 UBC and FEMA 356) are expected to be more conservative. Moreover, new buildings would generally be designed for increased forces and longer life expectancy than for rehabilitation of existing structures, more stringent seismic detailing requirements for new elements, etc. Lower requirements for existing building evaluation versus design can be thought of as giving a “break” to existing buildings, evaluating the exemption from all of the requirements in design methods. Therefore, DCRs for new building design using the 1997 UBC are expected to be higher than for FEMA 356, which are expected to be higher than from ASCE/SEI 31.

In general, a measure of conservatism in a given method is the level and complexity of investigation. Where more detailed evaluation or analysis is performed, this generally (but not always) results in a less conservative design check and lower DCR. For ASCE/SEI 31, conservatism from Tier 1 to Tier 3 is expected to decrease as analysis complexity increases. Therefore, DCRs should be highest for Tier 1 (most conservative), lower for Tier 2, and lowest for Tier 3 (most detailed analysis). 3D analysis is generally expected to provide less conservative results, causing a reduction in element DCRs. Because FEMA 356 and ASCE/SEI 31 Tier 3 results are based on 3D models, it is possible that DCRs might be lower than an ASCE/SEI 31 Tier 2 evaluation, based on more conservative 2D analysis. It is still expected that 1997 UBC results will be the most conservative, and have the highest DCRs.

In summary, the expected results of a DCR comparison are that the values would increase in the following order (except FEMA 356, which could be lower than ASCE/SEI 31 Tier 2): ASCE/SEI 31 Tier 3 (lowest DCR); ASCE/SEI 31 Tier 2; ASCE/SEI 31 Tier 1; FEMA 356; 1997 UBC (highest DCR).

While this ranking may not be true for all elements, it is expected that evaluation DCRs (ASCE/SEI 31) should generally be lower than design DCRs (FEMA 356, 1997 UBC). A summary of DCRs for the classroom roof is in Table 6, and for the gym in Table 7. ASCE/SEI 31 Tier 1 has no values as it has no provisions for checking diaphragms. Relative DCRs do not match what was expected. Instead of being highest, 1997 UBC DCRs are lower for the east/west seismic direction than other methods. Thus, the 1997 UBC would actually allow the thinnest diaphragm. In the north/south seismic direction results are about equal. This loading direction is relatively simple because there are no wall openings. In all cases the existing, single-layer configuration would be adequate, even if relative values are not in the expected order.

Differences in roof DCRs are notable at the gym, where FEMA 356 and ASCE/SEI 31 result in much higher values. m factors for immediate occupancy are much lower than for collapse prevention, increasing DCR. The 1997 UBC also requires increased design forces for the gym as an essential facility, but with less effect on DCR than FEMA 356 and ASCE/SEI 31. Higher FEMA 356 values also cause ASCE/SEI 31 Tier 3 DCRs to be higher than for the 1997 UBC. FEMA 356 also causes ASCE/SEI 31 Tier 3 results to be higher than Tier 2, the reverse of what is expected. Tier 3 and FEMA 356 results are higher than the 1997 UBC.

### Table 6. Summary of Classroom Roof Diaphragm DCRs

<table>
<thead>
<tr>
<th>Thickness</th>
<th>FEMA 356</th>
<th>Tier 1</th>
<th>Tier 2</th>
<th>Tier 3</th>
<th>1997 UBC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single layer</td>
<td>0.35</td>
<td>NA</td>
<td>0.23</td>
<td>0.26</td>
<td>0.17</td>
</tr>
<tr>
<td>Double layer</td>
<td>0.21</td>
<td>NA</td>
<td>0.13</td>
<td>0.16</td>
<td>0.10</td>
</tr>
</tbody>
</table>

### Table 7. Summary of Gymnasium Roof Diaphragm DCRs

<table>
<thead>
<tr>
<th>Thickness</th>
<th>FEMA 356</th>
<th>Tier 1</th>
<th>Tier 2</th>
<th>Tier 3</th>
<th>1997 UBC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single layer</td>
<td>0.18</td>
<td>NA</td>
<td>0.13</td>
<td>0.14</td>
<td>0.19</td>
</tr>
<tr>
<td>Double layer</td>
<td>0.11</td>
<td>NA</td>
<td>0.08</td>
<td>0.08</td>
<td>0.11</td>
</tr>
</tbody>
</table>

NA=not available.
UBC, also the reverse of what is expected. FEMA 356 $m$ factors are the main reason for higher values.

Summary DCRs for classroom wall panels are shown in Table 8 for seismic load in the east/west direction. DCRs are comparable between Tier 1 (double panel layer), Tier 2, and 1997 UBC methods, but it was expected that the 1997 UBC would have the highest DCR. Values for the 1997 UBC are lower than FEMA 356 and ASCE/SEI 31 results, which is unexpected. Comparison of ASCE/SEI 31 Tiers 2 and 3 shows Tier 2 values are lower than Tier 3, the reverse of the expected results. This is because FEMA 356 DCRs are also high, and Tier 3 results are 75% of FEMA 356.

Summary DCRs for gym walls are shown in Table 9. As in previous results, 1997 UBC DCRs are not the highest. Except for the longest wall panels with single layer sheathing, 1997 UBC values are lowest. Tier 1 results are comparable to the 1997 UBC for single-layer sheathing, but diverge for a double layer. No reduction is evident in Tier 1 DCRs for double-layer sheathing as acceptance criteria are not based on panel thickness, while allowable forces in the 1997 UBC increase for thicker panels.

Unexpectedly, Tier 3 results are generally higher than for Tier 2 or 1. Tier 3 results are directly proportional to those from FEMA 356, which are significantly higher than other methods. This unexpected result is most likely due to conservatism in FEMA 356 and ASCE/SEI 31 linear static procedures, and $m$ factors used. This was observed in the classrooms also, but differences are more pronounced in the gymnasium. The wider difference is due to more stringent requirements for immediate occupancy (gymnasium) compared to life safety performance (classrooms). The 1997 UBC also increases forces for the gymnasium, but the importance factor only increases forces 25%. The $m$ factors for immediate occupancy are approximately half of life safety $m$ factors, doubling the DCR values.

Conclusions and Recommendations

1. FEMA 154 alone shows that further analysis is not recommended. Plans review indicates further structural analysis is needed, and this check should be made for all buildings, whenever possible;
2. 3D modeling requirements in FEMA 356 need to be clarified. A clear explanation for use of the current deflection equations in determining element properties is needed. Boundary conditions for models should be clarified. A table of stiffness values is needed for finite-element models, with values based on panel thickness, materials, nailing patterns, assumptions for chord-slip, etc.;
3. The process for determining element capacities for wood structural sheathing (horizontal diaphragms and shear walls) is not efficient. A table of expected strengths based on panel thickness, nailing patterns, direction of load, material grade, etc. provided within a single document would make design more streamlined and reduce computational errors. Where detailed information for a panel system is not available, conservative, default expected strength values should be provided;
4. The ASCE/SEI Tier 1 “stress check” is not conservative. The 14.6 kN/m allowable load is not conservative, and not dependent on panel thickness, nailing pattern, connections, etc. This screening check could incorporate panel thickness, and the allowable force reduced to a more conservative value;
5. DCR comparisons showed that the 1997 UBC does not always result in more conservative designs than evaluation/rehabilitation documents, as was expected. FEMA 356, especially for immediate occupancy performance, is typically much more conservative than the 1997 UBC. This is likely caused by the $m$ values currently used in FEMA 356 and ASCE/SEI 31 under the linear static procedures; and
6. $m$ factors should be verified or modified for use in LSP for designs more comparable to those from current design codes, or a clear basis provided to explain design force levels and acceptance criteria.

Table 9. Summary of Gymnasium Wall Panel DCRs, E/W Seismic Direction

<table>
<thead>
<tr>
<th>Panel length (m)</th>
<th>FEMA 356</th>
<th>Tier 1</th>
<th>Tier 2</th>
<th>Tier 3</th>
<th>1997 UBC</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Single panel layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.27</td>
<td>2.34</td>
<td>0.76</td>
<td>1.23</td>
<td>1.76</td>
<td>0.70</td>
</tr>
<tr>
<td>4.88</td>
<td>2.51</td>
<td>0.76</td>
<td>1.19</td>
<td>1.88</td>
<td>0.71</td>
</tr>
<tr>
<td>7.01</td>
<td>2.21</td>
<td>0.76</td>
<td>1.13</td>
<td>1.66</td>
<td>0.74</td>
</tr>
<tr>
<td>7.32</td>
<td>1.65</td>
<td>0.76</td>
<td>1.13</td>
<td>1.24</td>
<td>0.74</td>
</tr>
<tr>
<td>7.92</td>
<td>1.98</td>
<td>0.76</td>
<td>1.13</td>
<td>1.49</td>
<td>0.75</td>
</tr>
<tr>
<td>13.72</td>
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<td>0.76</td>
<td>1.13</td>
<td>0.96</td>
<td>0.78</td>
</tr>
<tr>
<td>(b) Double panel layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.27</td>
<td>1.34</td>
<td>0.76</td>
<td>0.70</td>
<td>1.01</td>
<td>0.41</td>
</tr>
<tr>
<td>4.88</td>
<td>1.44</td>
<td>0.76</td>
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<td>1.08</td>
<td>0.42</td>
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<tr>
<td>7.01</td>
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<td>0.76</td>
<td>0.65</td>
<td>0.95</td>
<td>0.43</td>
</tr>
<tr>
<td>7.32</td>
<td>0.95</td>
<td>0.76</td>
<td>0.65</td>
<td>0.71</td>
<td>0.43</td>
</tr>
<tr>
<td>7.92</td>
<td>1.14</td>
<td>0.76</td>
<td>0.65</td>
<td>0.86</td>
<td>0.43</td>
</tr>
</tbody>
</table>

Table 8. Summary of Classroom Wall Panel DCRs, E/W Seismic Direction

<table>
<thead>
<tr>
<th>Panel length (m)</th>
<th>FEMA 356</th>
<th>Tier 1</th>
<th>Tier 2</th>
<th>Tier 3</th>
<th>1997 UBC</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Single panel layer</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.05</td>
<td>1.12</td>
<td>0.41</td>
<td>0.63</td>
<td>0.84</td>
<td>0.64</td>
</tr>
<tr>
<td>4.57</td>
<td>1.02</td>
<td>0.41</td>
<td>0.63</td>
<td>0.77</td>
<td>0.64</td>
</tr>
<tr>
<td>(b) Double panel layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.05</td>
<td>0.67</td>
<td>0.41</td>
<td>0.37</td>
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<td>4.57</td>
<td>0.61</td>
<td>0.41</td>
<td>0.37</td>
<td>0.46</td>
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References


