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Key Seismic Design Parameters: Steel and Wood Light-frame Shear Walls

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research report

Key Seismic Design Parameters: Steel and Wood Light-Frame Shear Walls

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American Iron and Steel Institute

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PREFACE

The primary objective of this project was to compare the requirements for the use of the overstrength factor, resistance and safety factors for seismic design and identify differences between steel and wood framing in industry documents.

The primary purpose of the report was to provide the AISI Codes and Standards staff and the Committee on Framing Standards with recommendations for the development of a road map to achieve parity between wood and cold-formed steel framing in design standards. The report may also be a useful reference for other researchers and for the development of design aides, as it illustrates the application of the various regulations.

Research Team
Steel Framing Alliance

Report on Key Seismic Design Parameters
Steel and Wood Light-Frame Shear Walls

11 August 2005

Prepared For:
AISI Committee on Framing Standards
And
Steel Framing Alliance
Washington, DC

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Report on Key Seismic Design Parameters
Steel and Wood Light-Frame Shear Walls

Introduction
This report details current United States building standard requirements for seismic forces used in design and detailing of steel and wood light-frame shear wall buildings. Building standards considered are:

1. ASCE Minimum Design Loads for Buildings and other Structures, 2005 Edition with Supplement 1 (ASCE 7-05),
2. AISI Standard for Cold Formed Steel Framing—Lateral Design, 2004 Edition (AISI Lateral Standard), and
3. AF&PA 2005 Edition Supplement Special Design Provisions for Wind and Seismic (AF&PA Wind and Seismic).

These standards are anticipated to be required under the 2006 International Building Code (IBC). A comparison of steel and wood light-frame design requirements is presented as a step towards identification of: differences, reasons for differences, and approaches to achieve parity where appropriate.

For illustration of the design parameters and their impact, an example building has been developed. It is a multi-family residential complex with stacked units (same plan at each story). Included in Appendix A are details of the example building including plans, elevation, assembly weights, building weights and base shears. A three-story version of this building is used to illustrate collector and shear wall chord stud and anchorage design. Two and three story versions are used in non-linear time-history analysis, presented in Appendix B.

I) Shear Wall Collectors and Top Chords

Comparison of Requirements
Under ASCE 7-05 Section 12.10.1.1, diaphragms and their collectors are designed using a secondary vertical distribution that varies between the story shear in the top story and the story weight times the base shear coefficient in the bottom story, as has been used in earlier ASCE 7 and UBC provisions. These diaphragm forces are used for collector design with load and resistance factor design (LRFD) or allowable stress design (ASD) basic load combinations. Use of overstrength, Ω_0, factors is not required. The same force level would be required by ASCE 7 for splices in wall top chords (top tracks) that serve as an extension of the collector member.

There are two circumstances in which ASCE 7 requires forces for collectors to be modified. First, in accordance with Plan Irregularity Table 12.3-1, where Irregularities 1 through 4 exist, collectors, collector splices and collector connections to vertical elements are required to be designed for 1.25 times the otherwise calculated force. This is per ASCE 7 Section 12.3.3.4, and applies in Seismic Design Categories (SDC) D and up.

Second, ASCE 7 Section 12.10.2.1 requires that collectors, collector splices and collector connections to vertical elements be designed using the load combinations with overstrength in SDC C through F, however structures and portions thereof braced entirely by light-frame shear walls are exempt from this requirement. This requirement is not triggered for the example building, but would be if bracing systems such as braced frames or flat straps were introduced.

For illustration of the impact of ASCE 7 collector requirements, the collector forces for the Line B transverse shear wall are provided in Table 1, below. The Line B shear wall is designed to take 37% of the story shear at each level. The maximum collector force occurs where the collector beam meets the shear wall. The ASCE 7 LRFD force (Table 1, column 1) is calculated as 37% of the F_{px} diaphragm force (Page A10) times the 17-foot collector length divided by the 25-foot building width. Note that horizontal forces are tabulated. If the collector also carries gravity loads, the vertical reaction would need to be determined using appropriate load combinations.

Table 1. Example Building Line B Collector Force (kips)				
Column No.	**1**	**2**	**3**	**4**
Diaphragm Level	**ASCE 7 LRFD Force 1.0 E**	**ASCE 7 ASD Force 0.7 E (1)**	**ASCE 7 Overstrength Force - LRFD Combination Ω_o Q_E (2)**	**ASCE 7 Overstrength Force - ASD Combination 0.7 Ω_o Q_E (3)**
ASCE 7 Section	**2.3.2**	**2.4.1**	**12.4.3.2**	**12.4.3.2**
ASCE 7 Equation	**5**	**5**	**Strength 5**	**ASD 5**
Roof	2.18	1.53	6.54	4.58
3rd Floor	2.49	1.74	7.47	5.23
2nd Floor	1.89	1.32	5.67	3.97

Notes:

1) Column 2 = Column 1 x 0.7.
2) Column 3 = Column 1 x System Overstrength Factor, Ω_o (3.0 per ASCE 7 Table 12.2-1).
3) Column 4 = Column 2 x System Overstrength Factor, Ω_o (3.0 per ASCE 7 Table 12.2-1).

LRFD required strengths (Column 1 or 3) are compared to LRFD strengths. ASD forces (Column 2 or 4) are compared to ASD allowable strengths. For the example building, under ASCE 7 requirements only the Column 1 or 2 ASCE 7 basic load combination forces would be required because the building is braced entirely by light-frame shear walls and has no plan irregularities.

Section C5.2 of the AISI Lateral Standard incorporates requirements for shear wall design that are in addition to the ASCE 7 requirements. The AISI requirement applies when the R-factor (seismic response modification coefficient) is taken as greater than three, and would be applicable to the example building with an R-factor of 6.5. These requirements are specific to connections, not members. Either the nominal tensile strength of the member or the overstrength force (Column 3 or 4 of Table 1 above) needs to be used for design of the following:

- Connections in shear wall top chord (top track) splices, and
- Connections in shear wall collectors

The wording used in the AISI Lateral Standard is not completely parallel to ASCE 7, in that consideration of overstrength is not specifically required for "collector connections to vertical elements." AISI is less stringent than ASCE 7 in that overstrength forces are not required for collector members. If the ASCE 7 requirement is triggered, checking of both collector members and connections to vertical elements for overstrength forces is required in addition to AISI C5.2.

The AF&PA Wind & Seismic Supplement makes no modification to the ASCE 7 requirements.

Conclusions: For buildings and portions thereof braced entirely by light-frame shear walls, the AISI Lateral Standard currently imposes more restrictive requirements than wood design. For other buildings, ASCE 7 requirements will govern and design will be roughly equivalent for both wood and steel buildings. See Section IV of this report for recommendations.

Areas for Discussion: In addition to Section IV of this report, discussion of the following is suggested:

- ASCE 7-05 Section 12.4.3.2 includes both LRFD (strength) and ASD load combinations for use with the overstrength, Ω_0, forces. As stated in AISI Lateral Standard Section C.5.2, the required strength for connections is the lesser of the nominal tensile strength of the member or the amplified seismic load. Following definitions in Section A.2, the design strength (nominal strength times resistance factor) must equal or exceed the required strength. This means that a phi factor is required for this provision, where it is not required for shear wall boundary members in Section C5.3. It should be confirmed that there is a specific reason for differentiation between C5.2 and C5.3 requirements.
- It should be decided whether use of either ASCE 7 LRFD or ASD overstrength load combinations should be acceptable for steel design. If use of ASCE 7 ASD overstrength load combinations is acceptable, it should be decided whether use of the ASCE 7 1.2 allowable strength increase is acceptable. If the 1.2 factor is not acceptable, a recommendation to ASCE 7 to delete the factor should be considered.
- ASCE 7 requirements differ from AISI in that 1) design of collector member using overstrength forces is required by ASCE 7 and not AISI, and 2) ASCE 7 uses the wording "…collectors and their connections to vertical elements..." Both of these differences should be confirmed and, if appropriate, clarification made to the AISI Lateral Standard. Connection to vertical elements may not be an issue where the collector is the shear wall top chord (top track), but could be an issue in more complex shear transfer connections such as a ledger on the side of a wall.
- AISI Lateral Standard Section C5.2 currently requires amplified seismic loads be used for connections for "boundary members." This is interpreted by some designers to be shear wall chord stud connections. Recommend striking or clarifying this term, as shear wall chord stud connections are already covered in Section C5.3.
- The AISI Lateral Standard should consider referencing use of overstrength load combinations (ASCE 7 or code) when using overstrength factors, to provide clarity for the designer.

Background Collector and Top Chord Requirements

Use of overstrength factors for design of collector members, splices, and their connections to vertical elements was first introduced into the 1997 UBC, with SEAOC Seismology as the proponent. Observations of damage to a hot-rolled steel frame building in the Northridge earthquake initiated the discussion. The building had a significant in-plane offset in bracing between stories (Vertical Irregularity 4) and suffered collector damage in the load path between upper and lower elements. This concern evolved into a broader concept of wanting to ensure that

collectors to vertical elements are not the weak link in lateral force resisting systems. The exemption of light-frame shear wall construction was on the basis that 1) collectors in light-frame systems were less critical due to more bracing lines and smaller areas being supported, and 2) that use of the overstrength factors was going to significantly change wood light-frame detailing, construction complication, and cost without evidence of this being a weak link in this building type. It was also pointed out during development of these provisions that steel light-frame requirements already required the overstrength factor, and it should not be imposed twice.

The ASCE 7 requirement for collector members comes from Section 4.6.2.2 of the 2003 NEHRP Provisions. There is no corresponding NEHRP commentary. This requirement is in Section 108.2.6 of the SEAOC Blue Book (SEAOC, 1999). The commentary to this Blue Book section notes the intent to ensure that inelastic energy dissipation occurs in the vertical elements rather than the collector and collection connections.

II) Shear Wall Boundary Members (Chord Studs)

Comparison of Requirements

For light-frame shear wall systems designed under ASCE 7-05, the only time that overstrength forces are required is for elements supporting discontinued shear walls (Section 12.3.3.3). The overstrength factors apply to the beams and columns supporting the discontinued shear walls and their interconnection. Applicability to the anchorage of the discontinued wall boundary member to the supporting system is the subject of interpretation; those interested are referred to the SEAOC Seismology web site for discussion. ASCE 7 does not require application of overstrength forces to boundary members of light-frame shear walls.

Section C5.3 of the AISI Lateral Standard requires that studs or other vertical shear wall boundary members and their anchorage shall have the nominal strength to resist amplified seismic loads (Ω_o overstrength loads), but need not be greater than the force the system can deliver. This requirement applies when the R-factor (seismic response modification coefficient) is taken as greater than three, and would be applicable to the example building with an R-factor of 6.5. Definitions indicate that the amplified seismic load is the horizontal component of seismic load E multiplied by the overstrength factor Ω_0. Because use of the ASCE 7 overstrength load combinations is not referenced, implications for other loads acting concurrently are not clear. The three-story stacked shear wall at Line B of the example building is used to illustrate the impact of overstrength factors on boundary member forces and design. The shear wall length is eight feet. The design unit shears for the shear walls are in a moderate range for this type of construction. The unit shears at the three stories and sheathing and fastening requirements are summarized in Table 2. See Appendix A for plans and an elevation of the Line B wall.

Table 2. Example Line B Shear Wall							
Story	Shear Wall ASD Force (kips)	Shear Wall ASD Unit Shear (plf)	Steel Shear Wall[1, 2]			Wood Shear Wall	
			Sheathing	Studs	Fastening	Sheathing	Fastening
3rd	2.42	280	7/16 OSB	33 mil	No. 8 @ 4"	7/16 OSB	8d @ 4"
2nd	4.18	520	7/16 OSB	54 mil	No. 8 @ 3"	15/32 STR I	10d @ 3"
1st	5.11	640	7/16 OSB	54 mil	No. 8 @ 2"	15/32 STR I	10d @ 2"

Notes:

1) Nominal strength (pounds per foot) from AISI Lateral Standard Table C2.1-3 must be greater than or equal to the Shear Wall ASD Unit Shear (plf) multiplied by 2.5.
2) Use of thicker steel framing members may permit greater fastener spacing. (Later in Table 4, thicker members are indeed selected to resist boundary forces.)

Table 3 illustrates design uplift and downward forces for the boundary members (chord studs). Downward forces are tabulated first without the beam reaction at Lines B and 1.4, and second with the beam reaction (The "beam reaction" is concentrated dead plus live load from the Line B beam spanning between Lines 1.4 and 2). ASCE 7 load combinations have been used for Table 3 forces. Spreadsheets detailing force calculations are included in Appendix A.

The LRFD overstrength load combination (Column 3) is the preferred method for meeting this requirement, because AISI Section C5.3 permits the LRFD overstrength load combination to be compared to nominal strength (use of a phi factor is not required). ASCE 7 Section 12.4.3.2 also includes ASD overstrength load combinations and permits use of a 1.2 increase in strength (shown here as a decrease in demand to permit comparison with other demands) when using ASD design procedures. AISI Section C5.3 does not address what strength is to be used with ASD overstrength load combinations. These combinations are shown in columns 4 and 5 of Table 3, but are not recommended for use with the AISI Lateral Standard.

Table 3. Shear Wall Boundary Member Forces, ASCE 7-05					
Column No.	**1**	**2**	**3**	**4**	**5**

Uplift Force (kips)					
Story	**LRFD Force (1)**	**ASD Force (2)**	**Overstrength Force - LRFD Combination (3)**	**Overstrength Force - ASD Combination (4)**	**Overstrength Force - ASD Combination / 1.2 (5)**
3rd	3.93	2.79	13.5	9.5	7.9
2nd	10.27	7.28	34.8	24.5	20.4
1st	18.27	12.92	61.1	42.9	35.8

Downward Force (kips) Without Beam Reaction					
Story	**LRFD Force (6)**	**ASD Force (7)**	**Overstrength Force - LRFD Combination (8)**	**Overstrength Force - ASD Combination (9)**	**Overstrength Force - ASD Combination / 1.2 (10)**
3rd	6.56	4.79	16.2	11.5	9.6
2nd	17.32	11.85	41.9	29.0	24.2
1st	29.74	20.08	72.5	50.0	41.7

Downward Force (kips) With Beam Reaction					
Story	**LRFD Force (6)**	**ASD Force (7)**	**Overstrength Force - LRFD Combination (8)**	**Overstrength Force - ASD Combination (9)**	**Overstrength Force - ASD Combination / 1.2 (10)**
3rd	8.71	7.34	18.3	13.3	11.1
2nd	25.06	20.48	49.6	33.6	28.0
1st	43.05	34.49	85.9	57.5	47.9

Notes:

1) LRFD basic load combination, ASCE 7 Section 2.3.2, Combination 7: 0.9D +1.0E.
2) ASD basic load combination, ASCE 7 Section 2.4.1, Combination 8:0.6D + 0.7E.
3) LRFD overstrength load Combination, ASCE 7 Section 12.4.3.2, Combination 7: $(0.9\text{-}0.2\ S_{DS})\ D + \Omega_o\ Q_E$, $\Omega_o = 3.0$.
4) ASD overstrength load combination, ASCE 7 Section 12.4.3.2, Combination 8: $(0.6\text{-}0.14\ S_{DS})\ D + 0.7\ \Omega_o\ Q_E$, $\Omega_o = 3.0$. This method is NOT recommended.
5) Column 4 divided by 1.2 based on ASCE 7 Section 12.4.3.2 strength increase when using ASD load combinations. This method is NOT recommended
6) LRFD basic load combination, ASCE 7 Section 2.3.2, Combination 5: 1.2D + 1.0E + 0.5L.
7) ASD basic load combination, ASCE 7 Section 2.4.1, most critical of Combination 5: D + 0.7E and Combination 6: D + 0.75 (0.7 E) + 0.75 L + 0.75 L_r.
8) LRFD overstrength load combination, ASCE 7 Section 12.4.3.2, Combination 5: $(1.2 + 0.2\ S_{DS})\ D + \Omega_o\ Q_E + 0.5\ L$, $\Omega_o = 3.0$.
9) ASD overstrength load combination, ASCE 7 Section 12.4.3.2, most critical of Combination 5: $(1.0 + 0.14\ S_{DS})\ D + 0.7\ \Omega_o\ Q_E$ and Combination 6: $(1.0 + 0.105\ S_{DS})\ D + 0.525\ \Omega_o\ Q_E + 0.75\ L + 0.75\ L_r$, $\Omega_o = 3.0$.
10) Column 4 divided by 1.2 based on ASCE 7 Section 12.4.3.2 strength increase when using ASD load combinations. This method is NOT recommended

Table 4A includes possible tie-down devices, wood posts and steel studs needed to carry the design forces for boundary members for the different ASD load combinations. This is done to give the reader a feel for the construction implied by the force levels. *Note that the ASD*

overstrength load combinations are not the recommended approach for boundary member chord studs. Stud requirements for LRFD overstrength load combinations are given in Table 4B.

For steel shear walls, where Tables 4A and 4B indicate multiple pairs of studs, it is relatively common for cold-formed tubular sections to be used. AISI Lateral Standard Table C2.1-3, Footnote 5, prohibits increasing steel stud thickness beyond that specified in Table 2. See *Areas for Discussion* below.

For wood shear walls a significant amount of shear wall length will be taken up by the required number of posts and tie-down devices noted in Table 4A if the overstrength load combinations are used. Use of the E_m forces is not current design practice, and at lower stories of the example building, construction fitting in the required hardware is not feasible. LRFD values are not currently available for tie-down devices; if available, use of LRFD methodology could potentially reduce the number of required tie-down devices. Otherwise, use of another bracing system (such as steel braced frame) at lower stories would be a likely consequence.

Table 4A. Shear Wall Boundary Member Forces "ASD" ASCE 7-05

Uplift Force (kips)

Story	ASD Force			Em ASD Force	E_m ASD Force / 1.2		
ASCE 7 Sec.	2.4.1	Post	Tie-Down	12.4.3.2		Post	Tie-Down
ASCE 7 Equ.	8	DF-L No. 2		ASD 8		DF-L No. 2	
3rd	2.79	2-2x4	PHD2	9.5	7.9	4x4	2-PHD5
2nd	7.28	4x4	2-PHD5	24.5	20.4	2-4x4	4-HDQ8
1st	12.92	4x4	2-HDQ8	42.9	35.8	3-4x4	5-HDQ8

Downward Force (kips) Without Beam Reaction

Story	ASD Force			Em ASD Force	E_m ASD Force / 1.2		
ASCE 7 Sec.	2.4.1	Post	Stud (No-Mils)	12.4.3.2		Post	Stud
ASCE 7 Equ.	5	DF-L No. 2	350S162	ASD 5		DF-L No. 2	350S162
3rd	4.79	2-2x4	2-43	11.5	9.6	2-4x4	3-54
2nd	11.85	2-4x6	4-54	29.0	24.2	3-4x6	5-68
1st	20.08	3-4x6	5-68	50.0	41.7	5-4x6	10-68

Downward Force (kips) With Beam Reaction

Story	ASD Force			Em ASD Force	E_m ASD Force / 1.2		
ASCE 7 Sec.	2.4.1	Post	Stud (No-Mils)	12.4.3.2		Post	Stud (No-Mils)
ASCE 7 Equ.	5 & 6	DF-L No. 2	350S162	ASD 5		DF-L No. 2	350S162
3rd	7.34	4x6	2-54	13.3	11.1	2-4x4	3-68
2nd	20.48	3-4x6	5-68	33.6	28.0	4-4x6	6-68
1st	24.49	4-4x6	8-68	57.5	47.9	6-4x6	10-68

Notes:

1) Increasing steel stud thickness to resist boundary forces, versus steel stud thickness selected in Table 2 is prohibited by AISI Lateral Standard Table C2.1-3, Footnote 5.
2) Steel stud ASD capacities were determined using tables from ICBO ER-4943P (SSMA, 2001). For 9 foot stud height, ASD allowable compression loads are: 1.95 k for 33mil, 2.66 k for 43 mil, 3.83 k for 54 mil, and 4.80 k for 68 mil.

Table 4B. Shear Wall Boundary Member Forces "LRFD" ASCE 7-05				
Uplift Force (kips)				
Story	**LRFD Force**		**Em LRFD Force**	
ASCE 7 Sec.	**2.3.2**	**Studs**	**12.4.3.2**	**Studs**
ASCE 7 Equ.	**7**	**350S162**	**7**	**350S162**
3rd	4.72	2-33	15.9	2-33
2nd	12.31	2-43	40.9	4-43
1st	21.82	2-54	71.7	6-68
Downward Force (kips) Without Beam Reaction				
Story	**LRFD Force**		**Em LRFD Force**	
ASCE 7 Sec.	**2.3.2**	**Studs**	**12.4.3.2**	**Studs**
ASCE 7 Equ.	**5**	**350S162**	**5**	**350S162**
3rd	7.34	2-43	18.4	2-68
2nd	19.35	2-43	48.0	6-68
1st	33.29	4-43	83.2	8-68
Downward Force (kips) With Beam Reaction				
Story	**LRFD Force**		**Em LRFD Force**	
ASCE 7 Sec.	**2.3.2**	**Studs**	**12.4.3.2**	**Studs**
ASCE 7 Equ.	**5**	**350S162**	**5**	**350S162**
3rd	9.5	2-43	20.7	2-68
2nd	27.1	4-43	55.7	6-68
1st	46.6	4-54	96.5	10-68

The AF&PA Wind & Seismic Supplement makes no modification to the ASCE 7 requirements.

Conclusion: The AISI Lateral Standard currently imposes requirements that are more restrictive than in wood design. See Section IV of this report for recommendations.

Areas for Discussion: In addition to Section IV of this report, discussion of the following is suggested:

- Design of shear wall boundary member studs for compression will generally either require use of thicker studs (increased mils) or use of more than two studs if the thickness is maintained. This is true for design using both ASD and overstrength forces. Footnote 5 of AISI Table C2.1-3 prohibits use of a greater stud thickness than tabulated, making the use of groups of studs mandatory. Footnote 5 is likely an appropriate limitation because the change in fastener load deflection behavior in thicker studs suggests that premature failure would possibly result. Current design practice needs to be identified, and appropriate guidance given for selection of boundary member studs. Testing of shear wall configurations that meet boundary member design requirements may be appropriate.
- It has been suggested that the use of ASCE 7 LRFD overstrength load combinations should be recommended by AISI, and the use of ASCE 7 ASD overstrength load combinations and the ASCE 7 ASD capacity increase of 1.2 should be prohibited. If true, this needs to be made clear in the AISI provisions.

- The AISI Lateral Standard should consider referencing use of overstrength load combinations (ASCE 7 or code) when using overstrength factors, to provide clarity for the designer.

Background on Shear Wall Boundary Member Requirements

Application of overstrength factors to the design of boundary members in wood light-frame shear walls has been discussed at length within the NEHRP wood subcommittee, and provisions are included in the 2003 NEHRP Provisions and commentary (BSSC, 2004). NEHRP Section 12.2.3.11 includes the following requirement: "Nominal strength of tie-down assemblies [device plus post] shall be equal or greater than the forces resulting from the nominal strength of the wall." The commentary to this section states: "The capacity-based nominal strength has been introduced primarily as a statement of design philosophy, with the intent of forcing the sheathing nailing to be the controlling failure mechanism. The complexity of load paths in wood frame buildings suggests that additional study is needed to achieve reliable development of desired failure mechanisms." Based on this being a statement of philosophy, this NEHRP requirement has not yet been moved forward into the AF&PA Wind and Seismic Supplement, ASCE 7 or the IBC. Alluded to in the commentary language is the great complexity of applying this provision to shear wall systems with both in-plane and out of plane offsets. Also considered was the cost and complexity of construction introduced (see Table 4 for example) and whether the added cost would realistically result in reasonable gains in building performance.

Further discussion considered what overturning forces are actually experienced by light-frame buildings. As one reference point from the CUREE-Caltech woodframe project, the tie-downs in the two-story house tested at UC San Diego (Filiatrault et Al., 2002) only went slightly above ASD design forces, while the base shear went well above 1g. This behavior was attributed to the structure behaving as a monolithic box rather than a series of individual shear walls and the stiffness of finish materials. It must be imagined that for larger light-frame buildings the finish materials will play a much smaller role in building behavior, and overstrength boundary member forces could potentially develop. There is little information at this time to determine at what building size or design the behavior might change.

Also raised as an issue was the varying level of overstrength that would need to be considered, not only in the wood structural panel shear walls themselves (factors of up to 5.8 reported by APA), but also with the influence of finish materials (up to 1.6g base shears seen in CUREE shake table testing for a building designed at approximately 0.18g).

There are other significant issues that have come up recently regarding design of wood shear wall boundary members and their connections. Prominent among them is tie-down eccentricity, and to what point this influences the stresses, deformations, and failure modes of tie-down posts. Limited testing and analytical studies have been conducted on this topic; however no specific design requirements have been put forward. It is difficult to address overstrength factors for wood boundary members without also considering the design effect of tie-down eccentricity. Similar eccentricity issues exist where steel or wood studs are connected from story to story using a strap on one face; this could have a significant impact on design for overstrength forces, but is not commonly considered in design calculations.

III) Comparison of Shear Wall Sheathing Design Values

Comparison of Design Values

Both the AISI Lateral Standard and the AF&PA Wind and Seismic Supplement tabulate wood structural panel shear wall values in terms of nominal shear strength. For seismic forces, AISI Section C2.1 specifies division by a safety factor, Ω, of 2.5 to determine available strength for ASD, OR multiplication by a resistance factor, Φ, of 0.6 to determine available strength for LRFD. Similarly Section 4.3.3 of the 2005 AF&PA wind and seismic supplement specifies dividing by a reduction factor of 2.0 for ASD OR multiplying by a resistance factor of 0.80 for LRFD. Comparison of design values and actual factors of safety is much more complex than simply comparing these factors because the tabulated nominal values have very different origins and built in assumptions. The most direct way to compare the parity of design values for wood and steel light-frame shear wall systems is to compare the ratio of design values to test values. Table 5 provides a comparison of selected shear wall assemblies from available testing.

When comparing the results of different shear wall tests it is important to acknowledge that the test procedure can greatly influence the test results. One significant influence is the protocol used for load or displacement history imposed on the shear wall. Variables include single direction monotonic loading versus reverse cycling. Also important are the number and magnitude of cycles imposed. Loading protocol is a detailed area of study that is beyond the scope of this discussion; those interested are referred to Gatto and Uang (2002) and Cobeen, Russell and Dolan (2004).

In the Table 5 data, adjustments for loading history suggested by Gatto and Uang (2002) have been used to normalize the peak test capacities to the SPD protocol. SPD protocol was chosen as the basis only because the majority of tabulated values had used this protocol; use of the SPD protocol as a basis of comparison is not intended to reflect preference for this or any other protocol. To adjust to anticipated SPD protocol results, peak capacities for APA and Serrette monotonic loading are multiplied by 0.78 and peak capacities for Gatto & Uang CUREE loading are multiplied by 0.75. Cyclic test results from CoLA and Serrette are not adjusted, as these used an SPD protocol. The SPD protocol was observed by Gatto and Uang to be the most punishing, resulting in both lower peak capacities and lower displacement at peak capacity. The adjustments must be recognized as approximate because of the small data sample Gatto & Uang had to work with, but gives some basis for direct comparison of the test results. Some variations also occur due to the number and type of displacement cycles imposed within the SPD protocol; however these variations are not significant relative to the variations between SPD, monotonic and CUREE loading.

Also important are boundary conditions on the specimen being tested. Included are the uplift anchorages, the stiffness and attachment of the loading beam and the ability for sheathing materials to slide past the framing or bear on perpendicular construction or the test jig. All of these are important variables that can result in a range of test results. There are no right or wrong conditions since all boundary conditions occur in the range of possible behavior, however the effect of using upper or lower bound boundary conditions should be kept in mind when using the results.

Some variations also occur in the methodology for design values assigned based on test data. The SPD protocol test values given in Table 5 are actual peak capacities read off of hysteresis loops where those were available, and reported "strength limit state" values for CoLA testing (also peak capacity). Consistent use of peak capacities normalizes the comparison made in Table 5.

The peak capacities reported in Table 5 differ in some cases from capacity values reported in research reports. In particular, Serrette (1996) Table 6 reports the nominal load capacity as the average of the negative and positive strengths using the lowest of the last complete set of stable hysteretic loops. Serrette (2002) Tables 9 & 10 report capacity based on the peak capacity from the second cycle envelope. The CoLA report (2001) went through a complex process of identifying a "yield" condition in order to assign recommended design values. The method used to recommend design values does not have an effect on the comparison made in Table 5.

The ratios of average tested value to assigned ASD value vary between 2.08 and 3.22 for wood and between 2.59 and 3.57 for steel. The value of 1.69 for wood is ignored since the tested configuration is not permitted to be built. Similarly steel test data that involved compression chord buckling failures have not been included, as these configurations are not permitted to be built. Overall there is not a tremendous difference between the ranges of assigned values, especially considered in light of all of the variations in test protocol that might be contributing to reported differences; however, the overall ratio of average tested value to assigned ASD value for the steel systems is higher by approximately 20%. A change in the phi factor for wood shear walls between 0.65 for the 2001 AF&PA Wind and Seismic Supplement and 0.80 for the 2005 AF&PA standard will widen the separation slightly when LRFD is used.

Table 5. Comparison of Wood Structural Panel Sheathed Shear Walls to ASD Design Values

Framing	Sheathing	Fastening	Edge Dist	Studs	Code Assigned Value (plf)		Tested Value plf)							Adjusted Tested / ASD	Average Tested / ASD
					Nominal	Adjusted to ASD	APA E72	Uang CUREE	Pardoen CUREE	CoLA SPD	Serrette Monotonic	Serrette SPD	Adjusted to SPD		
Wood	3/8 Rated	8dc at 6	3/8		440	220	600						468	2.127	2.418
					440	220	764						596	2.709	
	3/8 Rated	8dc at 4	3/8		640	320	964						752	2.350	2.350
	3/8 Rated	8db at 4	3/8		760	380		1090					818	2.151	2.191
					760	380		1130					848	2.230	
	3/8 Rated	8dc at 2*	3/8		1060	530	938						732	1.380	1.693
					1060	530	1363						1063	2.006	
	7/16 Rated	8dc at 3	3/8		900	450	1860						1451	3.224	3.224
	7/16 Rated	8dc at 4	1/2		760	380				916			916	2.411	2.075
					760	380				718			718	1.889	
					760	380				731			731	1.924	
	15/32 STR I	10dc at 6			680	340				818			818	2.406	2.474
					680	340				799			799	2.350	
					680	340				906			906	2.665	
	15/32 STR I	10dc at 4			1020	510				1211			1211	2.375	2.248
					1020	510				1164			1164	2.282	
					1020	510				1065			1065	2.088	
	15/32 STR I	10dc at 3			1330	665	2026						1580	2.376	2.376
	15/32 STR I	10dc at 2			1740	870				1874			1874	2.154	2.196
					1740	870				1951			1951	2.243	
					1740	870				1907			1907	2.192	
Steel	7/16 OSB	No.8 at 6	1/2		700	280				776			776	2.771	2.624
					700	280				720			720	2.571	
					700	280				708			708	2.529	
	7/16 OSB	No. 8 at 4	1/2		915	366				1070			1070	2.923	2.883
					915	366				1120			1120	3.060	
					915	366				975			975	2.664	
	7/16 OSB	No. 8 at 2	1/2		1625	650				1990			1990	3.062	3.062
					1625	650				1956			1956	3.009	
					1625	650				2024			2024	3.114	
	7/16 OSB	No. 8 at 6		33 mil	700	280					846		660	2.357	2.397
					700	280					875		683	2.438	
	7/16 OSB	No. 8 at 4		33 mil	915	366					1473		1149	3.139	3.008
					915	366					1350		1053	2.877	
	7/16 OSB	No. 8 at 6		33 mil	700	280						1000	1000	3.571	3.571
	7/16 OSB	No. 8 at 4		33 mil	915	366						1300	1300	3.552	3.484
					915	366						1250	1250	3.415	
	7/16 OSB	No. 8 at 2		54 mil	2350	940						2500	2500	2.660	2.660
					2350	940						2500	2500	2.660	
	7/16 OSB	No. 10 at 2		68 mil	3080	1232						3000	3000	2.435	2.597
					3080	1232						3400	3400	2.760	

*Tested without 3x stud now required at adjoining panel edges.

Background on Design Values

The background for wood structural panel shear walls in wood light-frame construction going back to values in the 1955 UBC are document by APA (1999). Original ASD design values were determined by a combination of calculation using nail values and testing to verify adequate factors of safety. Testing was not conducted for all of the cells in the shear wall table. Construction was adjusted in the several cases where the factor of safety was not judged to be adequate. As a result, to this day, testing does not exist for every cell in the wood shear wall tables. Also, test-based nominal strength tables do not exist. Current nominal strength tables are soft-conversions of previous ASD tables. Original testing was conducted using single direction monotonic testing with heavy tie-down rods in accordance with the ASTM E72 procedure. This has been supplemented by a number of cyclic tests with realistic tie-down conditions in recent years. Testing conducted by APA is summarized in Appendix A of the APA report. Ratios of test strength to ASD design range up to 5.8 in the APA report, while most fall between 2.5 and 4. It should be noted that the wide range of overstrength is an impediment to predicting maximum capacity for use with overstrength design. Cyclic testing since has confirmed that notable variation is commonly seen in test capacities for wood shear walls. Table 5 shows similar variability in ratios for steel shear walls.

IV) Discussion and Recommendations

Background

Parts I through III of this report describe a series of differences in design requirements between wood and steel light-frame shear wall buildings. The basis of current differences is largely a function of the different time and level of code scrutiny at the time the provisions were adopted. At this time, acceptance of wood light-frame construction is based primarily on a long history of generally good performance for one to three story buildings, with a few configurations that have shown to be vulnerable. The primary focus for wood design has been on understanding of the vulnerable configurations rather than introduction of overstrength requirements.

The steel provisions were introduced into the codes at a time when a great deal of attention was focused on the use of overstrength forces to allow development of a planned, stable weak link in the structural system. The introduction of overstrength factors was a logical design methodology at that point in time. This is still the currently accepted direction for development of seismic-resistant systems and design procedures, and would be anticipated for a system being introduced today.

The goal of design procedures should be acceptable performance, and the approach to acceptable performance should be similar for wood and steel shear wall construction because of their similar behavior. Wood and steel light-frame shear wall buildings are recently being extended to four, five and more stories, expand significantly beyond our knowledge of performance based on past earthquakes. At this time it is appropriate to consider the need for and benefit of design for overstrength forces for wood and steel light-frame shear wall buildings of all sizes. The issues suggested as key in this consideration are 1) whether and when use of design for overstrength forces contributes to building seismic performance and 2) if use of overstrength forces is the appropriate approach to developing reliable energy dissipation for wood and steel light-frame shear wall buildings.

As discussed in Part II, discussion within the NEHRP wood subcommittee has been that it is desirable for the design of boundary members to support the sheathing to framing connection as the primary source of non-linear activity and energy dissipation. The same mode has been identified as desirable by the AISI Lateral Standard developers. Countering this is 1) the variability in overstrength for wood shear walls (Part III) further increased by the strength contribution of finish materials, 2) the increased complication of detailing, and 3) the question of whether use of overstrength forces contributes to building performance based on observations from testing and analysis.

Overstrength Forces and Story Drift

One possible approach to judging the benefit of design for overstrength forces is to look at story drift as an indicator of deformation demand on the shear walls. Design for overstrength is very likely to be of benefit where actual drift in a building or story is going to be in the range of the peak shear wall capacity. In this case shear wall forces significantly above code design levels would be anticipated, and shear wall collectors and boundary members would be anticipated to see similar large forces. Conversely, story deflections in the range of 0.005 times the story and less can be recognized as representing near-elastic behavior for most shear wall systems. Somewhere between 0.005 and 0.02 times the story height, significant inelastic demands are made on the shear wall system. Above approximately 0.02 times the story height, most conventional shear wall systems have exceeded their peak capacity and are likely to exhibit significant drops in both capacity and stiffness. Where maximum anticipated seismic drift can be defined for buildings or classes of buildings, this could provide an appropriate trigger for requiring use of overstrength forces.

Two important characteristics of building deformation were observed in the CUREE-Caltech Woodframe Project results: 1) the contribution of finish materials and partitions was seen to greatly reduce story drift in testing and analytical studies to the point that inclusion of these materials was recommended for analysis for performance based design, and 2) concentration of building drift in the first story of multi-story buildings was repeatedly seen in testing and analysis. Discuss of these results can be found in Section B of Cobeen, Russell and Dolan (2004). These results suggest that it may be possible to identify buildings or portions of buildings where drift under design level earthquakes suggest that use of overstrength forces is not required or of particular benefit.

The contribution of non-structural finishes and partitions were seen as contributing greatly to the observed patterns of deformation demand. In order to identify buildings or portions of buildings based on this contribution, however, a method of quantification is needed. Section B.9 of Cobeen, Russell & Dolan proposes a method for prediction of story drift by dividing the total supported roof and floor area by the linear feet of structural wall plus partition. The resulting "area demand factor" is calculated in each direction at each story. Table B-6 in Section B.9 correlates the ratio to story drift from Woodframe Project testing analysis and field data. While this approach provides a good correlation with observed drifts, it is broad-brush in that it does not differentiate the strength and stiffness of varying materials. An alternate measure of nonstructural contribution to strength and stiffness (Richards, 2005) is the sum of capacity of all structural walls plus finishes divided by the sum of capacity of structural walls. This would again

be repeated in each direction at each story. A high ratio of finish capacity to structural capacity would suggest very little story drift might be expected. These are two of the possible approaches that could be considered.

To increase the information available for anticipated story drift, an analytical study of the example building is included in Appendix B. The analytical study used the SAWS computer program (Folz & Filiatrault, 2002) to perform non-linear time-history analysis on models that included shear walls, finish materials and partitions. This program was developed as part of the CUREE-Caltech Woodframe Project and used for some of the Woodframe Project analytical studies.

For the analytical study, building designs were developed in accordance with the ASCE 7-05, the AISI Lateral Standard and the AF&PA Wind & Seismic Supplement. The designs were developed for SDC D (S_{DS} = 1.0) and using R=6.5 for wood structural panel shear walls. Shear walls were selected in accordance with AISI and AF&PA. Descriptions of hysteretic behavior of wall structural and finish material were developed. The earthquake time history used was a 1994 Northridge earthquake Canoga Park record, scaled to a peak ground acceleration of 0.40g, anticipated to produce a short period spectral acceleration in the range of 1.0g.

Table 6 summarizes peak story drifts for two and three-story steel and wood shear wall buildings using both upper and lower bound gypsum wallboard properties (based on variation in test boundary conditions). The drift in the first story of the two-story building can be seen to be quite modest, and average to 0.5 inches for the upper and lower bound models. The three-story building has higher drifts, pushing towards peak capacity.

Table 6. Drift Results From Non-Linear Time-History Analysis

Building Configuration	Light Frame System	Peak Story Drift (in)		
		1st Story	2nd Story	3rd Story
2-Story -- Lower Bound Gypboard	Steel	0.65	0.39	
	Wood	0.77	0.29	
2-Story -- Upper Bound Gypboard	Steel	0.47	0.38	
	Wood	0.48	0.30	
3-Story -- Lower Bound Gypboard	Steel	0.85	0.53	0.66
	Wood	1.94	1.02	0.49
3-Story -- Upper Bound Gypboard	Steel	0.81	0.60	0.51
	Wood	1.75	1.21	0.43

The analysis results suggest that significant drift is likely to occur in the first story of the three-story configuration. This is only one specific building configuration, and broader consideration is needed before guidance can be given. Testing and analysis by Simpson Strong-Tie has suggested that it may be possible to have peak drifts occur in other than first story walls. This behavior also requires further study.

Other Issues Regarding Shear Wall Boundary Member, Chord and Collector Design

Test reports by Serrette (1996 and 2002) reported chord stud crippling (crushing at the tie-down or at the web cut-out immediately above the tie-down). In review, the shear wall chord studs used in testing did not meet ASD requirements for shear wall design, let alone overstrength

requirements. To date, chord stud crushing has not been seen in test assemblies that meet ASD design requirements. No testing has occurred of shear wall assemblies with chord studs designed for overstrength forces. Very limited facilities are available that would be capable of testing multi-story shear wall assemblies with cumulative overturning forces; Tables 4 and 5 illustrate that such multi-story assemblies can generate very high chord stud demands.

The three different possible levels of chord stud capacity need to be clearly differentiated for discussion of shear wall requirements 1) not meeting ASD requirements, 2) meeting ASD requirements, and 3) meeting overstrength requirements.

Suggested Approach to Resolving Overstrength Differences

The following is suggested as a route towards exploring and resolving the difference in overstrength requirements between wood and steel light frame shear wall buildings for shear wall collectors and boundary members:

1. Divide light-frame shear wall buildings into two or three groups based on estimated deformation demand. Conceptually these would represent high medium and low deformation demand systems (possibly special, intermediate and ordinary systems in NEHRP terminology):
    - Collect available test and analysis data on building deformation demand
    - Try use of the two suggested methods of predicting deformation demand (and any others identified) to establish the best correlation with deformation data
    - Establish a criterion for assigning buildings to groups
2. Define a philosophy for performance of wood structural panel shear wall buildings in each of the two or three groups.
    - Accept or modify shear wall sheathing to framing fastening as the desired mode of behavior. Include consideration of:
        - Failure modes for fastener connection to sheathing – brittle or ductile, and implications of differences in failure modes between steel and wood studs
        - Fastener and framing implications for shear wall deformation capacity – in some cases drift at peak capacity drops significantly with higher shear wall capacity. Is a minimum drift at peak capacity needed as part of this philosophy?
    - Consider other philosophies where small anticipated story drifts suggest that overstrength force design would give little benefit.
3. Identify how to develop or support the desired mode of failure or other design philosophy, considering
    - Overstrength of shear wall and finish materials
    - Ductility and reliability of desired mechanism
    - Mode of failure – local versus global
        - In particular the possibility of local buckling of compression members is of concern as a weak link. Buckling may become more likely as member fastening to sheathing is lost due to fastener pull-through.
    - Member value variability – wood is set at 5% exclusion
4. Review testing and acceptance criteria relative to performance philosophy.
5. Conduct component testing that reflects configurations proposed for use when overstrength forces are required. Testing would preferably include some multi-story walls.

6. Conduct analytical studies to confirm that performance is achieved. Review building groups as a function of performance.
7. Develop design guidance.

Preliminary Recommendations for Wood Structural Panel Shear Values

Section III of this report pointed out that on average there is about a 20% variation in the ratio between assigned ASD design strength and peak test strength between wood and steel stud shear walls. This raises the question of whether adjustment of values should be recommended.

At a quick glance, it may seem that adjustment of (one or the other sets of) shear wall values is appropriate, however there are several characteristics of sheathing and fastening behavior that might bear consideration.

First, it should be considered whether the failure modes of sheathing fastening produce equivalent results. Failure methods for sheathing to framing fastening in wood light-frame shear walls include nail withdrawal, nail pull through (head pulls through sheathing), edge tear out, and low-cycle fatigue failure of the fastener (very occasionally). Cobeen, Russell and Dolan (2004) recommend increasing the distance from the nail center-line to panel edge from 3/8 inches (current standard practice) to ¾ inch. This edge distance generally eliminates edge tear-out as a failure mode. The primary modes of sheathing to fastening failure noted by Serrette (1996, 1997, 2002) are screw pull-though and edge tear-out. Overall none of the light-frame shear walls exhibit the stable energy-dissipating behavior that is being achieved in some of the other structural systems. The fastener modes seen in the steel walls are somewhat more damaging in that each cycle causes permanent gouging of the panel, increasing deformation, and flattening of the hysteresis loops (indicating reduced energy dissipation). Somewhat broader hysteresis loops can occur in wood shear walls.

Second, it should be considered that local crippling of steel studs could potentially occur as fasteners fail, leaving stud flanges unsupported. The wood studs can theoretically remain more stable with loss of fasteners.

In the end it is equivalent performance of the building that is desired, however at this time we do not have a broad basis for comparison. Given the relative closeness of the ratios between test and design values for wood and steel light-frame shear walls (Table 5 and Part III), the variations that are inherent due to testing protocol, and the issues mentioned above, further study is suggested before adjustment of design values is undertaken.

Preliminary Recommendation For Review of Shear Wall Test Data

Some of the cyclic test results in LGSRG-06-02 (Serrette, Morgan & Sorhouet, 2002) show a notable reduction in shear wall deflection at peak capacity relative to other steel and wood shear wall tests. Walls with No. 8 screws at 2 inches on center have deflections of approximately 1.2 and 1.4 inches at peak capacity, followed by significant drops in strength and stiffness. Because this is less than code-permitted peak story drifts of 0.02h, further study of potential implications for reduced performance is recommended.

Preliminary Recommendations for Review of Current AISI Lateral Standard (Summary From Parts I and II)

Collector Provisions

- Use of a phi factor is required for collector overstrength design per AISI S5.2, where it is not required for shear wall boundary members in Section C5.3. It should be confirmed that there is a specific reason for differentiation between C5.2 and C5.3 requirements.
- Is use of either ASCE 7 LRFD or ASD overstrength load combinations acceptable for steel design? If use of ASCE 7 ASD overstrength load combinations is acceptable, it should be decided whether use of the ASCE 7 1.2 allowable strength increase is acceptable. If the 1.2 factor is not acceptable, a recommendation to ASCE 7 to delete the factor should be considered.
- ASCE 7 requirements differ from AISI in that 1) design of collector member using overstrength forces is required by ASCE 7 and not AISI, and 2) ASCE 7 uses the wording "…collectors and their connections to vertical elements..." Both of these differences should be confirmed and, if appropriate, clarification made to the AISI Lateral Standard.
- AISI Lateral Standard Section C5.2 currently requires amplified seismic loads be used for connections for "boundary members." This is interpreted by some designers to be shear wall chord stud connections. Recommend striking or clarifying this term, as shear wall chord stud connections are already covered in Section C5.3.
- The AISI Lateral Standard should consider referencing use of overstrength load combinations (ASCE 7 or code) when using overstrength factors, to provide clarity for the designer.

Shear Wall Chord Provisions

- Design of shear wall boundary member studs for compression will generally either require use of thicker studs (increased mils) or use of more than two studs if the thickness is maintained. This is true for design using both ASD and overstrength forces. Footnote 5 of AISI Table C2.1-3 prohibits use of a greater stud thickness than tabulated, making the use of groups of studs mandatory. Footnote 5 is likely an appropriate limitation because the change in fastener load deflection behavior in thicker studs suggests that premature failure would possibly result. Current design practice needs to be identified, and appropriate guidance given for selection of boundary member studs. Testing of shear wall configurations that meet boundary member design requirements may be appropriate.
- It has been suggested that the use of ASCE 7 LRFD overstrength load combinations should be recommended by AISI, and the use of ASCE 7 ASD overstrength load combinations and the ASCE 7 ASD capacity increase of 1.2 should be prohibited. If true, this needs to be made clear in the AISI provisions.
- The AISI Lateral Standard should consider referencing use of overstrength load combinations (ASCE 7 or code) when using overstrength factors, to provide clarity for the designer.

V) References

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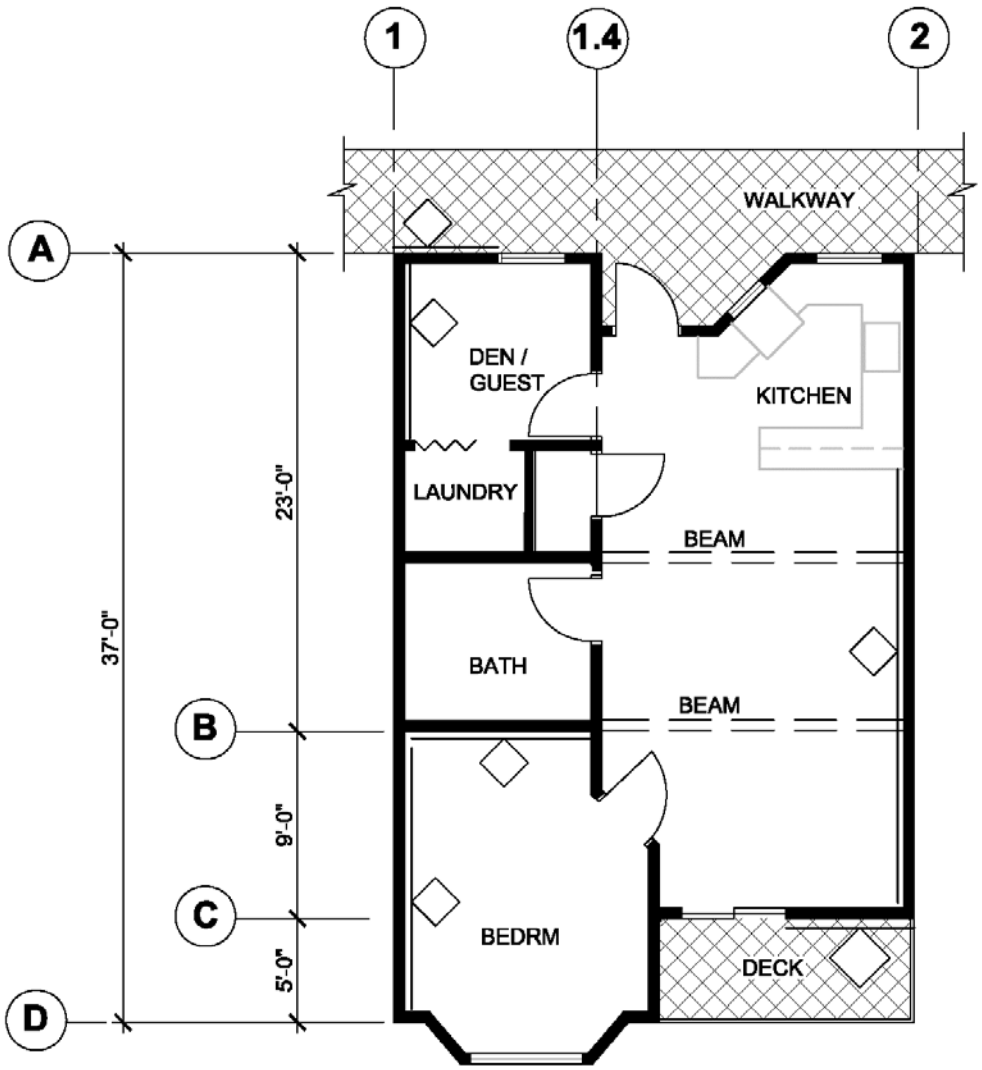
APPENDIX A

to

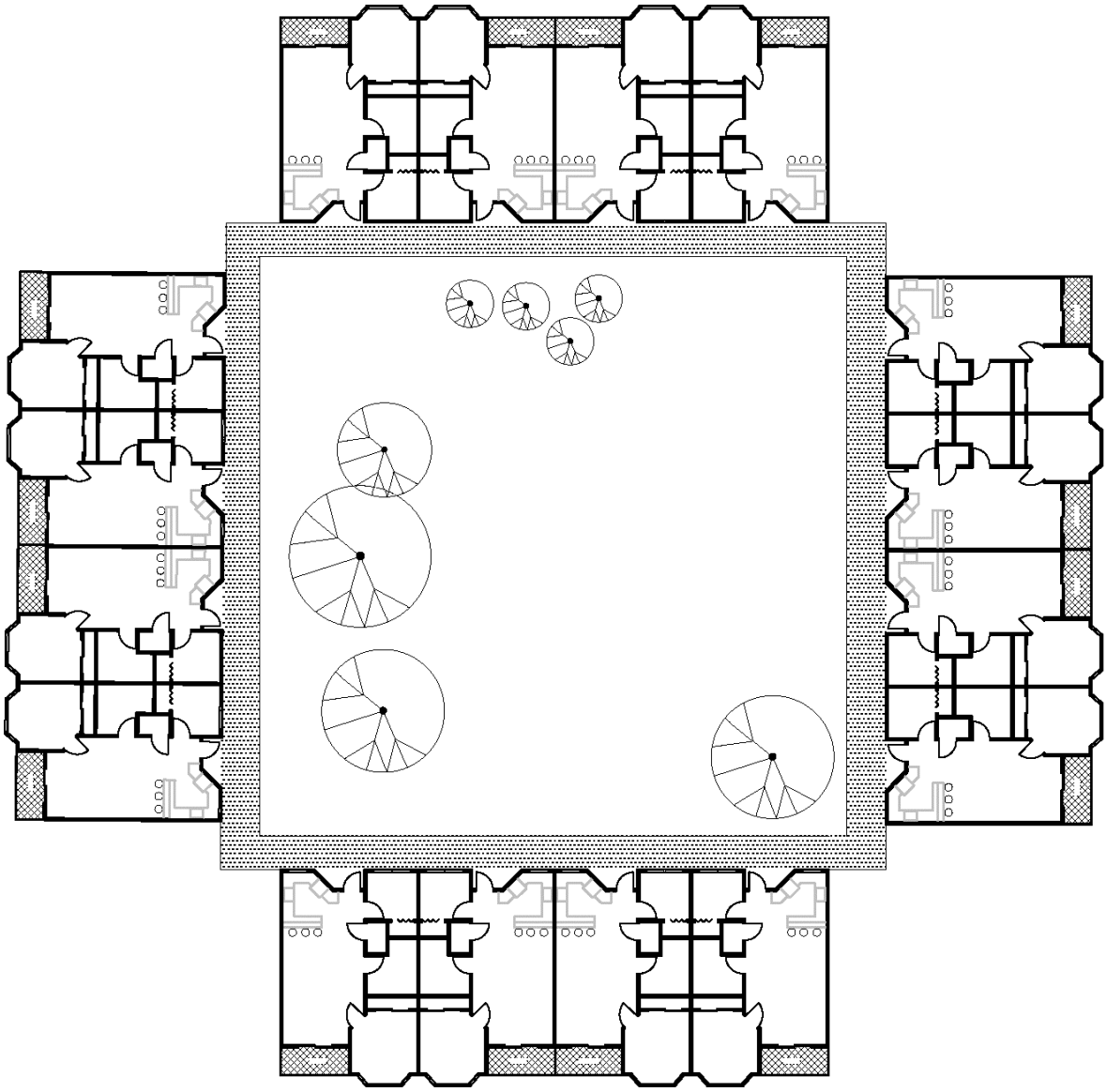
Report on Key Seismic Design Parameters Steel and Wood Light-Frame Shear Walls

11 August 2005

Prepared For:
AISI Committee on Framing Standards
And
Steel Framing Alliance
Washington, DC



UNIT PLAN
NOT TO SCALE



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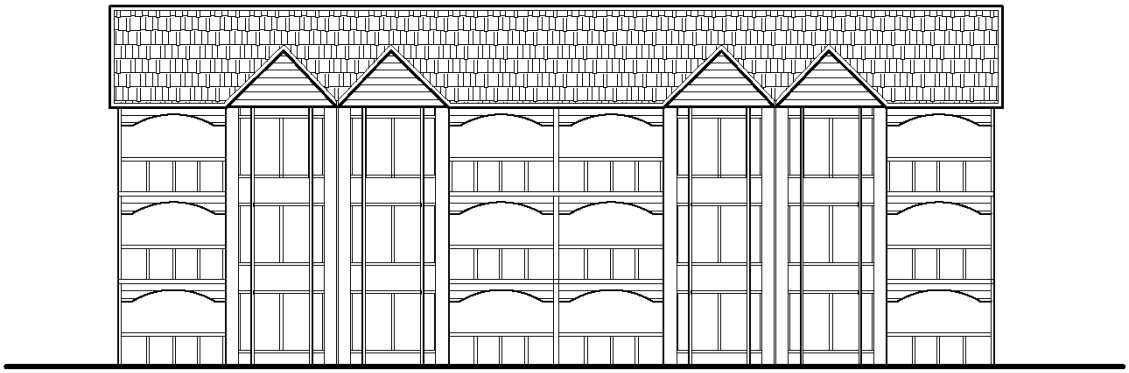
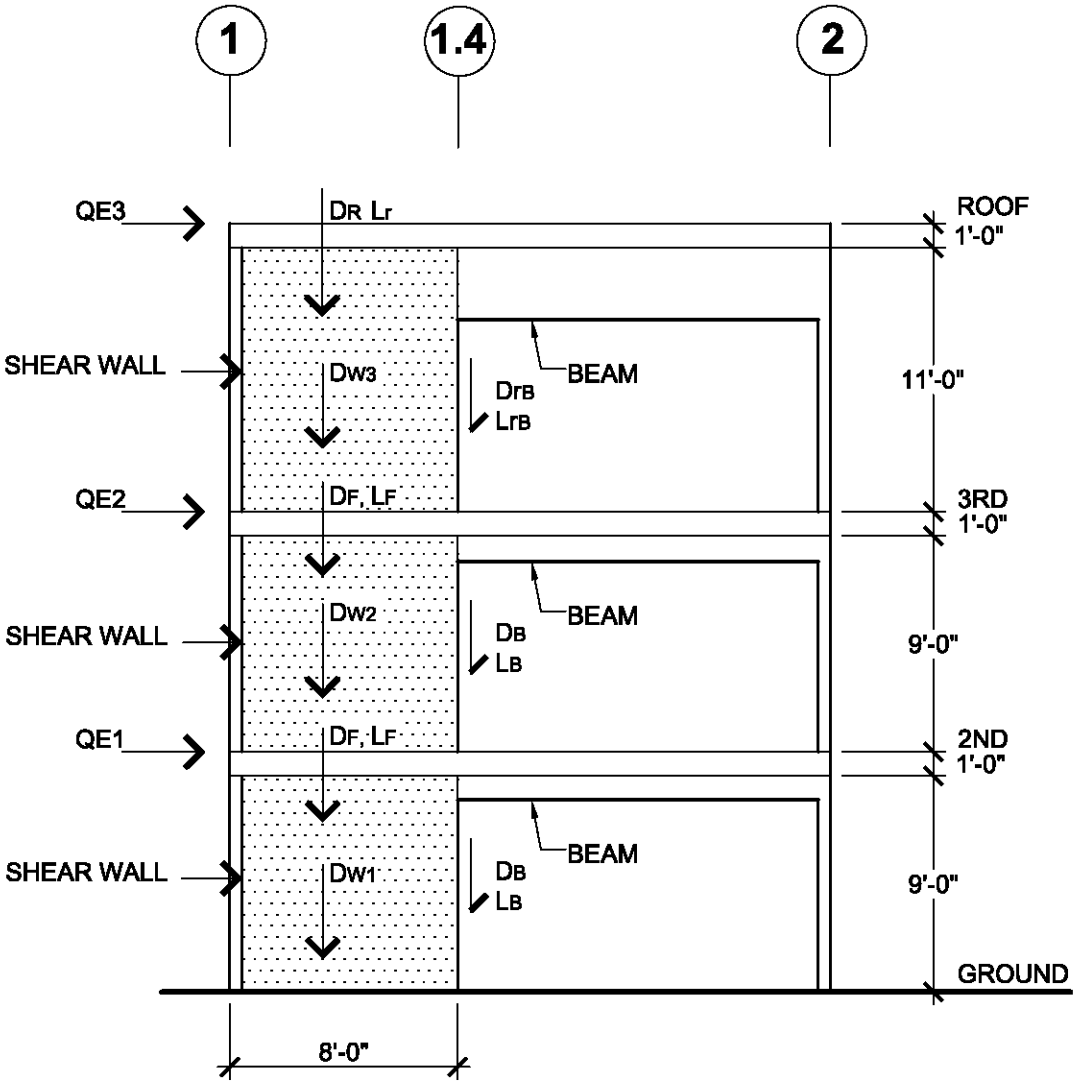
NOT TO SCALE

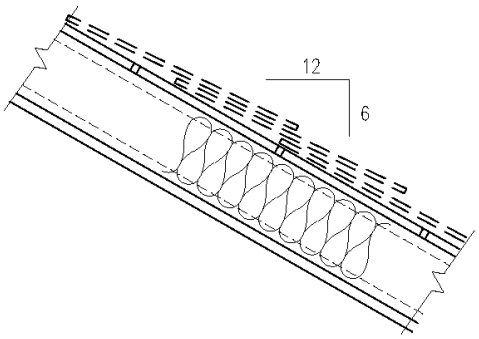


ELEVATION
3 STORY RESIDENTIAL
NOT TO SCALE



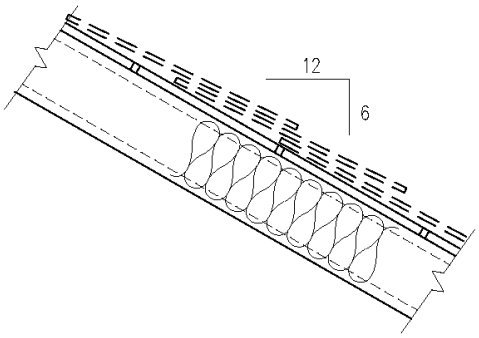
SHEAR WALL ELEVATION - LINE B

ROOF WITH CATHEDRAL CEILING



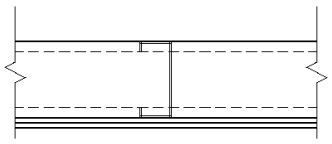
	PSF
COMP SHINGLE	3.5
REROOF	3.0
5/8 OR 19/32" WOOD STRUCTURAL PANEL SHEATHING	1.9
R30 BATT INSULATION	1.4
600C162, 33 MIL	0.6
BLK @ 8'	0.6
5/8" GYP.	3.1
MISC	0.3
TOTAL	14.4
ADJUST FOR SLOPE X 1.12	16.1
USE	17

TRUSS ROOF WITHOUT CEILING



	PSF
COMP SHINGLE	3.5
REROOF	3.0
5/8 OR 19/32" WOOD STRUCTURAL PANEL SHEATHING	1.9
R30 BATT INSULATION	1.4
TRUSS @ 24	0.7
BLK @ 8'	0.6
MISC	0.4
TOTAL	12.1
ADJUST FOR SLOPE X1.12	13.6
USE	14.0

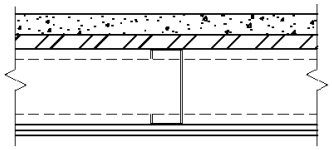
CEILING AT TRUSSES



	PSF
TRUSS @ 24	0.7
BLKG	0.6
2 X 5/8" GYPSUM WALLBOARD	6.3
MISC	0.4
TOTAL	8.0
USE	8

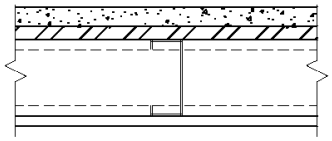
7/6/05

RESIDENCE FLOOR



	PSF
FINISH	1.0
1 1/2" LT. WT. CONC.	15.0
¾" OR 23/32" COMBINED SUB–FLOOR & UNDERLAYMENT	2.3
JOIST @ 16"	2.0
BLKG & BEAMS	1.0
⅝" GYPBD	3.1
MISC/ MEP/ HUNG SOFFIT	1.6
TOTAL	26
USE	26

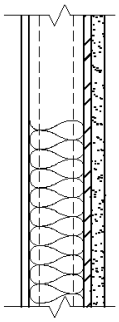
DECK AND COMMON AREA FLOOR



	PSF
1 1/2" LT. WT. CONC.	15.0
¾" OR 23/32" COMBINED SUB–FLOOR & UNDERLAYMENT	2.3
JOIST @ 16"	2.0
BLKG & BEAMS	1.0
STUCCO	10.0
MISC	1.3
TOTAL	33
USE	33

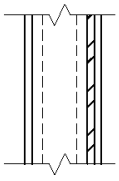
7/6/05

EXTERIOR WALL WITH WSP BRACING



	PSF
7/8" 3–COAT STUCCO	10.0
1/2" OR 15/32" WOOD STRUCTURAL PANEL SHEATHING	1.5
350S162, 33 STUDS@16"	0.7
INSUL	0.8
TOP & BOTT PLATES	0.3
5/8" GYPBD	3.1
MISC	0.9
TOTAL	17.0
USE	17

INTERIOR WALL



	PSF
350S162, 33 STUDS@16"	0.7
1/2" OR 15/32" WOOD STRUCTURAL PANEL SHEATHING	1.5
TOP & BOTT PLATES	0.3
2 X 5/8" GYPBD	6.2
MISC	0.9
TOTAL	9.5
USE W/ PLWD	10
USE W/O PLWD	8

7/6/05

BUILDING WEIGHT SUMMARY									
Stucco Wall Finish									
End Unit									
Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #	
Sloped Ceiling	typ				390	17.0	6,630	6,630	**Roof+Ceil**
Flat Roof/Ceil	bath,kitch,				673	22.0	14,806	14,806	21,436
Walls 3-R	Line 1	Total	37	9	333			5,301	
	Ext	window/door			30	5.0	150		
		typ exterior			303	17.0	5,151		
	Line 2	Total	32	9	288			2,304	
	Int Party	open & door			0	2.0	0		
		interior party			288	8.0	2,304		
	Lines 1-2	Total	37	9	333			2,214	
	Int Partition	open & door			75	2.0	150		
		typ int			258	8.0	2,064		
	Line A	Total	27	7	189			2,625	
	Ext	open & door			49	5.0	245		
		typ exterior			140	17.0	2,380		
	Line C	Total	32	9	288			3,936	
	Ext	open & door			80	5.0	400		
		typ exterior			208	17.0	3,536		
									Walls 3-R
	Lines A-C	interior	40	9	360			2,448	18,828
					72	2.0	144		
					288	8.0	2,304		
2nd or 3rd Fl	Deck				60	33.0	1980	1980	
	Unit				860	26.0	22360	22360	**2nd, 3rd Fl**
	Walkway				146	33.0	4818	4818	29158
Walls 1-2	Line 1	Total	37	9	333			5,301	
	Ext	window/door			30	5.0	150		
		typ exterior			303	17.0	5,151		
	Line 2	Total	32	9	288			2,304	
	Party	open & door			0	2.0	0		
		typ interior			288	8.0	2,304		
	Lines 1-2	Total	37	9	333			2,214	
	Partition	open & door			75	2.0	150		
		typ interior			258	8.0	2,064		
	Line A	Total	27	9	243			3,543	
	Exterior	open & door			49	5.0	245		
		typ exterior			194	17.0	3,298		
	Line C	Total	32	9	288			3,936	
	Exterior	window/door			80	5.0	400		
		typ interior			208	17.0	3,536		
	Lines A-C	Total	40	9	360			2,448	
	Partition	window/door			72	2.0	144		**Walls 1-2, 2-3**
		typ exterior			288	8.0	2,304		19,746

Sum of Weight for Seismic Forces (lb)		
Roof/Ceiling		
Roof + Ceiling		21436
1/2 walls 2-R		9414
Sum		**30850**
Third Floor		
Third Floor		29158
1/2 walls 3-R		9414
1/2 walls 2-3		9873
Sum		**48445**
Second Floor		
Second Floor		29158
1/2 walls 2-3		9873
1/2 walls 1-2		9873
Sum		**48904**
At Grade		
1/2 walls 1-2		9873
Sum		**9873**
Sum 2nd, 3rd & Roof		**128199**
Sum at Grade		**138072**

Primary & Diaphragm Vertical Distribution - Strength Level Forces

Level	W (K)	h (ft)	Wxh	Wxh/Sum	Str V (k)	Sum V (k)	SumV/Sum w	Fpx
Roof	30.85	34.5	1064	0.439	8.66	8.66	0.281	8.66
Third	48.45	19.0	920	0.380	7.49	16.14	0.204	9.86
Second	48.90	9.0	440	0.182	3.58	19.72	0.154	7.52
Sum	128.20		2425	1.0				

2003 IBC			
Assume Site Class D			
SS	1.5	Cap per Sec. 9.5.5.2.1	
Fa	1		
SMS	1.5		
SDS	1		
Rho	1		
Eh	ρ^*S_{DS}/R		
	0.154	Strength Level	
	0.108	ASD Level= 0.7*Str	
S1	0.6		
Fv	1.5		
SM1	0.9		
SD1	0.6		
T	0.28	SEC	
Rho	1		ASCE 7-05
R	6.5		Table 12.2-1
Eh	$\rho^*S_{D1}/(R^*T)$		
	0.330	but not more than 0.165	

OVERTURNING USING ASCE 7-05 LRFD LOAD COMBINATIONS WITHOUT OVERSTRENGTH FACTORS

GRAVITY LOADS				HORIZONTAL SEISMIC			STORY	SDS
DEAD LOAD		LIVE LOAD			INCREM.	TOTAL	HEIGHT	1.00
Dr	1.54	Lr	1.54	QE			h	**WALL**
Df	2.5	Lf	4.16	3rd	3.204	3.204	12	**WIDTH**
Dw3	0.96			2nd	2.771	5.976	10	w
Dw12	0.72			1st	1.325	7.300	10	8

UPLIFT Sec. 2.3.2 EQ. 7 $0.9*D- 1.0*E = 0.9*D - 1.0*Q_E - 1.0*(0.2\ S_{DS}\ D)$

		Vetical Force at Wall End					
	STORY	**D**	**QE**	**$0.2S_{DS}D$**	**L**	**Lr**	**Sum**
		+0.9*D/2	-1.0*QE*h/w	$-(1.0)*0.2S_{DS}D/2$	NA	NA	
					NA	NA	
	3rd	1.125	-4.806	-0.250	0.000	0.000	-3.93
	Δ 2nd	1.449	-7.469	-0.322	0.000	0.000	
	Σ 2nd	2.574	-12.276	-0.572	0.000	0.000	-10.27
	Δ 1st	1.449	-9.125	-0.322	0.000	0.000	
	Σ 1st	4.023	-21.401	-0.894	0.000	0.000	-18.27

DOWN Sec. 2.3.2 EQ. 5 $1.2D+ 1.0*E + 0.5L= 1.2D + 1.0*Q_E + 1.0*(0.2\ S_{DS}\ D) + 0.5L$

		Vetical Force at Wall End					
	STORY	**D**	**QE**	**$0.2S_{DS}D$**	**L**	**Lr**	**Sum**
		+1.2 * D/2	1.0*QE*h/w	$+(1.0)*0.2S_{DS}D/2$	0.5*L/2	NA	
			V*h/w		NA	NA	
	3rd	1.500	4.806	0.250	0.000	0.000	6.56
	Δ 2nd	1.932	7.469	0.322	1.040	0.000	
	Σ 2nd	3.432	12.276	0.572	1.040	0.000	17.32
	Δ 1st	1.932	9.125	0.322	1.040	0.000	
	Σ 1st	5.364	21.401	0.894	2.080	0.000	29.74

OVERTURNING USING ASCE 7-05 LRFD LOAD COMBINATIONS WITHOUT OVERSTRENGTH FACTORS

GRAVITY LOADS w/ beam reaction				HORIZONTAL SEISMIC			STORY	SDS
DEAD LOAD		LIVE LOAD			INCREM.	TOTAL	HEIGHT	1.00
Dr	4.62	Lr	4.62	QE			h	**WALL**
Df	7.5	Lf	12.48	3rd	3.204	3.204	12	**WIDTH**
Dw3	0.96			2nd	2.771	5.976	10	w
Dw12	0.72			1st	1.325	7.300	10	8

UPLIFT Sec. 2.3.2 EQ. 7 $0.9*D- 1.0*E = 0.9*D - 1.0*Q_E - 1.0*(0.2\ S_{DS}\ D)$

		Vetical Force at Wall End					
STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
	+0.9*D/2	-1.0*QE*h/w	$-(0.7)*0.2S_{DS}D/2$	NA	NA	
				NA	NA	
3rd	2.511	-4.806	-0.558	0.000	0.000	-2.85
Δ 2nd	3.699	-7.469	-0.822	0.000	0.000	
Σ 2nd	6.210	-12.276	-1.380	0.000	0.000	-7.45
Δ 1st	3.699	-9.125	-0.822	0.000	0.000	
Σ 1st	9.909	-21.401	-2.202	0.000	0.000	-13.69

DOWN Sec. 2.3.2 EQ. 5 $1.2D+ 1.0*E +0.5L = 1.2*D + 1.0*Q_E + 1.0*(0.2\ S_{DS}\ D) + 0.5*L$

		Vetical Force at Wall End					
STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
	1.2*D/2	1.0*QE*h/w	$+(1.0)*0.2S_{DS}D/2$	0.5*l/2	NA	
					NA	
3rd	3.348	4.806	0.558	0.000	0.000	8.71
Δ 2nd	4.932	7.469	0.822	3.120	0.000	
Σ 2nd	8.280	12.276	1.380	3.120	0.000	25.06
Δ 1st	4.932	9.125	0.822	3.120	0.000	
Σ 1st	13.212	21.401	2.202	6.240	0.000	43.05

OVERTURNING USING ASCE 7-05 ASD LOAD COMBINATIONS WITHOUT OVERSTRENGTH FACTORS

GRAVITY LOADS				HORIZONTAL SEISMIC - Strength Level			STORY	SDS
DEAD LOAD		LIVE LOAD			INCREM.	TOTAL	HEIGHT	1.00
Dr	1.54	Lr	1.54	QE			h	WALL
Df	2.5	Lf	4.16	3rd	3.204	3.204	12	WIDTH
Dw3	0.96			2nd	2.771	5.976	10	w
Dw1 Dw2	0.72			1st	1.325	7.300	10	8

Note: dead and live loads are total, D/2 and L/2 are reactions at each end of the wall.

UPLIFT Sec. 2.4.1 EQ. 8 $0.6*D - 0.7*E = 0.6*D - 0.7*Q_E - 0.7*(0.2\ S_{DS}\ D)$

		Vetical Force at Wall End					
	STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
		+0.6*D/2	-0.7*QE*h/w	-(0.7)*0.2S_{DS}D/2	NA	NA	
			-V*h/w		NA	NA	
	3rd	0.750	-3.364	-0.175	0.000	0.000	-2.79
Δ	2nd	0.966	-5.229	-0.225	0.000	0.000	
Σ	2nd	1.716	-8.593	-0.400	0.000	0.000	-7.28
Δ	1st	0.966	-6.388	-0.225	0.000	0.000	
Σ	1st	2.682	-14.981	-0.626	0.000	0.000	-12.92

DOWN Sec. 2.4.1 EQ. 5 $D + 0.7*E = D + 0.7*Q_E + 0.7*(0.2\ S_{DS}\ D)$

		Vetical Force at Wall End					
	STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
		+D/2	0.7*QE*h/w	+(0.7)*0.2S_{DS}D/2	NA	NA	
			V*h/w		NA	NA	
	3rd	1.250	3.364	0.175	0.000	0.000	4.79
Δ	2nd	1.610	5.229	0.225	0.000	0.000	
Σ	2nd	2.860	8.593	0.400	0.000	0.000	11.85
Δ	1st	1.610	6.388	0.225	0.000	0.000	
Σ	1st	4.470	14.981	0.626	0.000	0.000	20.08

DOWN Sec. 2.4.1 EQ. 6 $D + 0.75*0.7*E + 0.75L + 0.75Lr = D + 0.525*Q_E + 0.525*(0.2\ S_{DS}\ D) + 0.75L + 0.75Lr$

		Vetical Force at Wall End					
	STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
		+D/2	+0.525*QE*h/w	+(0.525)*0.2S_{DS}D/2	+0.75L/2	+0.75Lr/2	
			+V*h/w				
	3rd	1.250	2.523	0.131	0.000	0.578	4.482
Δ	2nd	1.610	3.921	0.169	1.560	0.000	
Σ	2nd	2.860	6.445	0.300	1.560	0.578	11.743
Δ	1st	1.610	4.791	0.169	1.560	0.000	
Σ	1st	4.470	11.235	0.469	3.120	0.578	19.872

OVERTURNING USING ASCE 7-05 ASD LOAD COMBINATIONS WITHOUT OVERSTRENGTH FACTORS

GRAVITY LOADS w/ beam reaction				HORIZONTAL SEISMIC			STORY	SDS
DEAD LOAD		LIVE LOAD			INCREM.	TOTAL	HEIGHT	1.00
Dr	4.62	Lr	4.62	QE			h	WALL
Df	7.5	Lf	12.48	3rd	3.204	3.204	12	WIDTH
Dw3	0.96			2nd	2.771	5.976	10	w
Dw12	0.72			1st	1.325	7.300	10	8

UPLIFT Sec. 2.4.1 EQ. 8 $0.6*D- 0.7*E = 0.6*D - 0.7*Q_E - 0.7*(0.2\ S_{DS}\ D)$

	Vetical Force at Wall End					
STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
	+0.6*D/2	-0.7*QE*h/w	-(0.7)*0.2S_{DS}D/2	NA	NA	
		-V*h/w		NA	NA	
3rd	1.674	-3.364	-0.391	0.000	0.000	-2.08
Δ 2nd	2.466	-5.229	-0.575	0.000	0.000	
Σ 2nd	4.140	-8.593	-0.966	0.000	0.000	-5.42
Δ 1st	2.466	-6.388	-0.575	0.000	0.000	
Σ 1st	6.606	-14.981	-1.541	0.000	0.000	-9.92

DOWN Sec. 2.4.1 EQ. 5 $D+ 0.7*E = D + 0.7*Q_E + 0.7*(0.2\ S_{DS}\ D)$

	Vetical Force at Wall End					
STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
	+D/2	0.7*QE*h/w	+(0.7)*0.2S_{DS}D/2	NA	NA	
		V*h/w		NA	NA	
3rd	2.790	3.364	0.391	0.000	0.000	6.55
Δ 2nd	4.110	5.229	0.575	0.000	0.000	
Σ 2nd	6.900	8.593	0.966	0.000	0.000	16.46
Δ 1st	4.110	6.388	0.575	0.000	0.000	
Σ 1st	11.010	14.981	1.541	0.000	0.000	27.53

DOWN Sec. 2.4.1 EQ. 6 $D+ 0.75*0.7*E +0.75L +0.75Lr = D + 0.525*Q_E + 0.525*(0.2\ S_{DS}\ D) + 0.75L + 0.75Lr$

	Vetical Force at Wall End					
STORY	D	QE	$0.2S_{DS}D$	L	Lr	Sum
	+D/2	+0.525*QE*h/w	+(0.525)*0.2S_{DS}D/2	+0.75L/2	+0.75Lr/2	
		+V*h/w				
3rd	2.790	2.523	0.293	0.000	1.733	7.339
Δ 2nd	4.110	3.921	0.432	4.680	0.000	
Σ 2nd	6.900	6.445	0.725	4.680	1.733	20.482
Δ 1st	4.110	4.791	0.432	4.680	0.000	
Σ 1st	11.010	11.235	1.156	9.360	1.733	34.494

OVERTURNING USING ASCE 7-05 Em LOAD ** STRENGTH COMBINATIONS**

GRAVITY LOADS					HORIZONTAL SEISMIC		STORY	SDS
DEAD LOAD		LIVE LOAD			INCREMENTAL	TOTAL	HEIGHT	1.00
Dr	1.54	Lr	1.54	QE			h	**WALL**
Df	2.5	Lf	4.16	3rd	3.204	3.204	12	**WIDTH**
Dw3	0.96			2nd	2.771	5.976	10	w
Dw12	0.72			1st	1.325	7.300	10	8
				Ω_0	3			

UPLIFT Sec. 12.4..3.2 EQ. 7 $(0.9-0.2S_{DS})*D - \Omega_0*Q_E$

	Vetical Force at Wall End					
STORY	**D**	**Q_E**	**$0.2S_{DS}D$**	**L**	**Lr**	**Sum**
	+0.9*D/2	$-3*Q_E*h/w$	$-0.2S_{DS}D/2$	NA	NA	
				NA	NA	
3rd	1.125	-14.419	-0.250	0.000	0.000	-13.54
Δ 2nd	1.449	-22.408	-0.322	0.000	0.000	
Σ 2nd	2.574	-36.827	-0.572	0.000	0.000	-34.83
Δ 1st	1.449	-27.375	-0.322	0.000	0.000	
Σ 1st	4.023	-64.202	-0.894	0.000	0.000	-61.07

DOWN Sec. 12.4..3.2 EQ. 5 $(1.2+0.2SDS)*D+ \Omega_0*Q_E + 0.5*L$

	Vetical Force at Wall End					
STORY	**D**	**Q_E**	**$0.2S_{DS}D$**	**L**	**Lr**	**Sum**
	+1.2*D/2	+3*QE*h/w	$+0.2S_{DS}D/2$	+0.5*L/2	NA	
				(1)	NA	
3rd	1.500	14.419	0.250	0.000	0.000	16.17
Δ 2nd	1.932	22.408	0.322	1.040	0.000	
Σ 2nd	3.432	36.827	0.572	1.040	0.000	41.87
Δ 1st	1.932	27.375	0.322	1.040	0.000	
Σ 1st	5.364	64.202	0.894	2.080	0.000	72.54

1) Note 1 to ASCE 7 Section 12.4.3.2

OVERTURNING USING ASCE 7-05 Em LOAD ** STRENGTH COMBINATIONS**

GRAVITY LOADS w/ beam reaction DEAD LOAD		LIVE LOAD			HORIZONTAL SEISMIC INCREMENTAL	TOTAL	STORY HEIGHT	SDS 1.00
Dr	4.62	Lr	4.62	QE			h	WALL
Df	7.5	Lf	12.504	3rd	3.204	3.204	12	WIDTH
Dw3	0.96			2nd	2.771	5.976	10	w
Dw12	0.72			1st	1.325	7.300	10	8
				Ω_0	3			

UPLIFT Sec. 12.4..3.2 EQ. 7 $(0.9-0.2S_{DS})*D - \Omega_0*Q_E$

	Vetical Force at Wall End					
STORY	D	Q_E	$0.2S_{DS}D$	L	Lr	Sum
	+0.9*D/2	$-3*Q_E*h/w$	$-0.2S_{DS}D/2$	NA	NA	
				NA	NA	
3rd	2.511	-14.419	-0.558	0.000	0.000	-12.47
Δ 2nd	3.699	-22.408	-0.822	0.000	0.000	
Σ 2nd	6.210	-36.827	-1.380	0.000	0.000	-32.00
Δ 1st	3.699	-27.375	-0.822	0.000	0.000	
Σ 1st	9.909	-64.202	-2.202	0.000	0.000	-56.50

DOWN Sec. 12.4..3.2 EQ. 5 $(1.2+0.2SDS)*D+ \Omega_0*Q_E + 0.5*L$

	Vetical Force at Wall End					
STORY	D	Q_E	$0.2S_{DS}D$	L	Lr	Sum
	+1.2*D/2	+3*QE*h/w	$+0.2S_{DS}D/2$	+0.5*L/2	NA	
				(1)	NA	
3rd	3.348	14.419	0.558	0.000	0.000	18.32
Δ 2nd	4.932	22.408	0.822	3.126	0.000	
Σ 2nd	8.280	36.827	1.380	3.126	0.000	49.61
Δ 1st	4.932	27.375	0.822	3.126	0.000	
Σ 1st	13.212	64.202	2.202	6.252	0.000	85.87

1) Note 1 to ASCE 7 Section 12.4.3.2

OVERTURNING USING ASCE 7-05 Em LOAD **ASD COMBINATIONS**

GRAVITY LOADS				HORIZONTAL SEISMIC			STORY	SDS
DEAD LOAD		LIVE LOAD			INCREMENTAL	TOTAL	HEIGHT	1.00
Dr	1.54	Lr	1.54	QE			h	**WALL**
Df	2.5	Lf	4.16	3rd	3.204	3.204	12	**WIDTH**
Dw3	0.96			2nd	2.771	5.976	10	w
Dw1,2	0.72			1st	1.325	7.300	10	8
				Ω_0	3			

UPLIFT Sec. 12.4..3.2 EQ. 8 $(0.6-0.14S_{DS})*D - 0.7*\Omega_0*Q_E$

		Vetical Force at Wall End					
	STORY	**D**	**Q_E**	**$0.14S_{DS}D$**	**L**	**Lr**	**Sum**
		+0.6*D/2	-0.7*3*Q_E*h/w	-0.14S_{DS}D/2	NA	NA	
					NA	NA	
	3rd	0.750	-10.093	-0.175	0.000	0.000	-9.52
	Δ 2nd	0.966	-15.686	-0.225	0.000	0.000	
	Σ 2nd	1.716	-25.779	-0.400	0.000	0.000	-24.46
	Δ 1st	0.966	-19.163	-0.225	0.000	0.000	
	Σ 1st	2.682	-44.942	-0.626	0.000	0.000	-42.89

DOWN Sec. 12.4..3.2 EQ. 5 $(1.0+0.14S_{DS})*D+ 0.7*\Omega_0*Q_E$

		Vetical Force at Wall End					
	STORY	**D**	**Q_E**	**$0.14S_{DS}D$**	**L**	**Lr**	**Sum**
		+1.0*D/2	+0.7*3*QE*h/w	+0.14S_{DS}D/2	NA	NA	
						NA	
	3rd	1.250	10.093	0.175	0.000	0.000	11.52
	Δ 2nd	1.610	15.686	0.225	0.000	0.000	
	Σ 2nd	2.860	25.779	0.400	0.000	0.000	29.04
	Δ 1st	1.610	19.163	0.225	0.000	0.000	
	Σ 1st	4.470	44.942	0.626	0.000	0.000	50.04

DOWN Sec. 12.4..3.2 EQ. 6 $(1.0+0.105S_{DS})*D+ 0.525*\Omega_0*Q_E + 0.75*L + 0.75*Lr$

		Vetical Force at Wall End					
	STORY	**D**	**Q_E**	**$0.105*S_{DS}D$**	**L**	**Lr**	**Sum**
		+1.0*D/2	+0.525*3*QE*h/w	+0.105*S_{DS}D/2	+0.75*L/2	+0.75*Lr/2	
	3rd	1.250	7.570	0.131	0.000	0.578	9.53
	Δ 2nd	1.610	11.764	0.169	1.560	0.000	
	Σ 2nd	2.860	19.334	0.300	1.560	0.578	24.63
	Δ 1st	1.610	14.372	0.169	1.560	0.000	
	Σ 1st	4.470	33.706	0.469	3.120	0.578	42.34

OVERTURNING USING ASCE 7-05 Em LOAD **ASD COMBINATIONS**

GRAVITY LOADS w/ beam reaction				HORIZONTAL SEISMIC			STORY	SDS
DEAD LOAD		LIVE LOAD			INCREMENTAL	TOTAL	HEIGHT	1.00
Dr	4.62	Lr	4.62	QE			h	**WALL**
Df	7.5	Lf	12.48	3rd	3.204	3.204	12	**WIDTH**
Dw3	0.96			2nd	2.771	5.976	10	w
Dw1,2	0.72			1st	1.325	7.300	10	8
				Ω_0	3			

UPLIFT Sec. 12.4..3.2 EQ. 8 $(0.6-0.14S_{DS})*D - 0.7*\Omega_0*Q_E$

		Vetical Force at Wall End					
	STORY	**D**	**Q_E**	**$0.14S_{DS}D$**	**L**	**Lr**	**Sum**
		+0.6*D/2	-0.7*3*Q_E*h/w	-0.14S_{DS}D/2	NA	NA	
					NA	NA	
	3rd	1.674	-10.093	-0.391	0.000	0.000	-8.81
	Δ 2nd	2.466	-15.686	-0.575	0.000	0.000	
	Σ 2nd	4.140	-25.779	-0.966	0.000	0.000	-22.60
	Δ 1st	2.466	-19.163	-0.575	0.000	0.000	
	Σ 1st	6.606	-44.942	-1.541	0.000	0.000	-39.88

DOWN Sec. 12.4..3.2 EQ. 5 $(1.0+0.14S_{DS})*D+ 0.7*\Omega_0*Q_E$

		Vetical Force at Wall End					
	STORY	**D**	**Q_E**	**$0.14S_{DS}D$**	**L**	**Lr**	**Sum**
		+1.0*D/2	+0.7*3*QE*h/w	+0.14S_{DS}D/2	NA	NA	
						NA	
	3rd	2.790	10.093	0.391	0.000	0.000	13.27
	Δ 2nd	4.110	15.686	0.575	0.000	0.000	
	Σ 2nd	6.900	25.779	0.966	0.000	0.000	33.64
	Δ 1st	4.110	19.163	0.575	0.000	0.000	
	Σ 1st	11.010	44.942	1.541	0.000	0.000	57.49

DOWN Sec. 12.4..3.2 EQ. 6 $(1.0+0.105S_{DS})*D+ 0.525*\Omega_0*Q_E + 0.75*L + 0.75*Lr$

		Vetical Force at Wall End					
	STORY	**D**	**Q_E**	**$0.105*S_{DS}D$**	**L**	**Lr**	**Sum**
		+1.0*D/2	+0.525*3*QE*h/w	+0.105*S_{DS}D/2	+0.75*L/2	+0.75*Lr/2	
	3rd	2.790	7.570	0.293	0.000	1.733	12.39
	Δ 2nd	4.110	11.764	0.432	4.680	0.000	
	Σ 2nd	6.900	19.334	0.725	4.680	1.733	33.37
	Δ 1st	4.110	14.372	0.432	4.680	0.000	
	Σ 1st	11.010	33.706	1.156	9.360	1.733	56.96

APPENDIX B

to

Report on Key Seismic Design Parameters Steel and Wood Light-Frame Shear Walls

11 August 2005

Prepared For:
AISI Committee on Framing Standards
And
Steel Framing Alliance
Washington, DC

Analytical Study Description

A model multi-family residential building has been analyzed as a part of this report in order to study the seismic performance of a light-frame building with wood structural panel shear walls. The building is made up an individual dwelling unit that is stacked to two and three stories. Stacks are arranged into clusters to form buildings. A unit plan, cluster plan, and building elevations follow.

Engineered structural design of the shear walls for a two and three story stack of units has been prepared in accordance with ASCE Minimum Design Loads for Buildings and other Structures, 2005 Edition with Supplement 1 (ASCE 7-05), AISI Standard for Cold Formed Steel Framing—Lateral Design, 2004 Edition (AISI Lateral Standard), and AF&PA 2005 Edition Supplement Special Design Provisions for Wind and Seismic (AF&PA Wind and Seismic). Assembly weights, building weights, seismic forces, and shear wall design are included in this appendix following the building drawings.

Purpose

The primary purpose of this analysis it to quantify and compare seismic demand on a building in order to judge the need for seismic design using overstrength factors, and in order to judge the estimated relative performance of steel and wood light-frame shear wall buildings.

Nonlinear Analysis

Nonlinear analysis of the example building used the Seismic Analysis of Woodframe Structures (SAWS) program (Folz and Filiatrault, 2003). This program was developed as part of the CUREE-Caltech Woodframe Program in order to provide nonlinear analysis tool for researchers and designers. Limited validation studies of SAWS program results against shake table results from the CUREE-Caltech Woodframe Project were conducted by Folz and Filiatrault. Analysis models included both designated bracing and finish materials. SAWS analytically predicted forces and deflections compared favorably with shake table results, and clearly differentiated analysis results with and without finish materials.

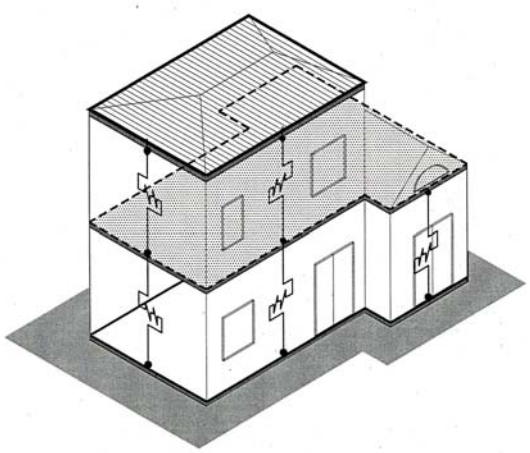


Figure B-1. Analysis model. (From FEMA 232 Home Builder's Guide to Seismic Resistant Construction, 2005)

The SAWS analysis program uses rigid diaphragms to represent floor and roof diaphragms. Walls are modeled as nonlinear springs with hysteretic parameters developed specifically to describe the behavior of wood-frame bracing systems. A simplified representation of the rigid diaphragms and wall springs for the model building are illustrated.

Hysteretic parameters have been developed from component testing data to describe load-deflection Behavior of shear walls and finishes. The parameters are derived from available component testing data. Figure B-2 illustrates the meaning of the parameters, and a tabulated summary of parameter values is provided. Also provided are several illustrations of the hysteretic behavior resulting from defined parameters. Although developed for wood light-frame construction, the parameters and analysis method are equally suited to describing the hysteretic behavior of steel light-frame components.

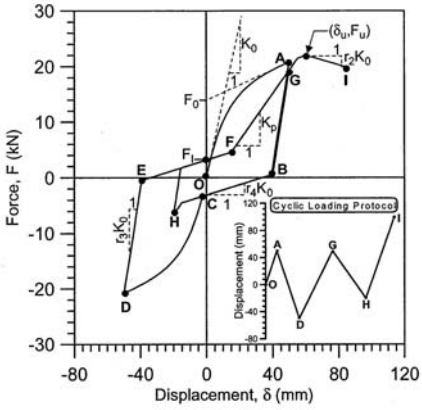


Figure B-2. Hysteretic Parameters for model (from Folz and Filiatrault, 2002).

For each of the sheathing and finish materials the hysteretic parameters were determined for a four-foot length. Because widely varying wall lengths are used in the building, the parameters have been scaled for varying lengths in the analysis input. Hysteretic parameters that are currently available from laboratory testing of wall components vary based on wall boundary conditions, test set-up and test protocol. Parameters chosen for this analysis attempted to capture both upper and lower bounds of gypsum wallboard hysteretic behavior.

Earthquake demand is represented using the larger horizontal acceleration record from Canoga Park for the 1994 Northridge, California earthquake. This record was chosen because it corresponds well with the code design spectra over a range of building periods. The peak acceleration has been scaled by dividing an S_{DS} value of 1.0 by 2.5, resulting in a peak ground acceleration 0.40g for SDC D. For comparison, the recorded ground motion has a peak acceleration of 0.42g and was scaled to 0.50g to represent Zone 4 anticipated ground motions in the CUREE shake table testing. The ground motion scaling used for this analysis represents the demand used as a basis for code design. The demand from the maximum considered earthquake ground motion would be approximately 50% greater.

Analysis Variations

Variations in the analysis model include steel light-frame and wood light-frame results, two and thre story unit stacks, upper and lower bound gypsum wallboard strength and stiffness properties, and an evaluation of the implications of under-designed shear wall boundary members (modeled as a 10% reduction in capacity and a 20% reduction in drift at peak capacity). The study with under-designed shear wall boundary members was originally envisioned as providing information on shear wall performance with and without overstrength design, however the analysis results do not support this comparison. Analysis was only run in the transverse direction since the long party walls will make longitudinal behavior more conservative than transverse.

Analysis Results

The primary results taken from the analysis are peak story drifts for each wall line at each story level. The peak drifts provide information on the deformation demand, and the likely force demand placed on shear walls in varying locations through the building. Peak drifts for the different analysis runs are included in tabular form.

Performance Implications

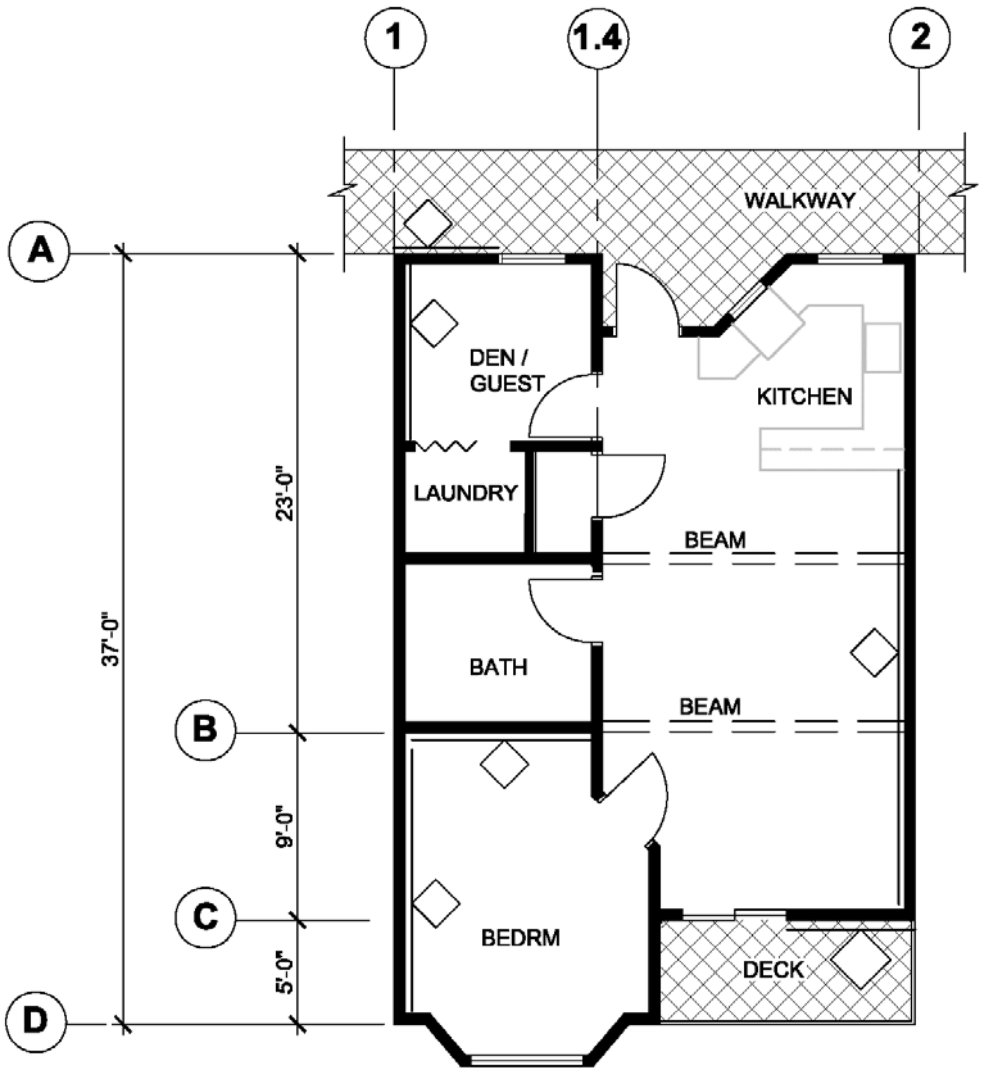
A suggested in Part IV of this report, story drift can provide an indication of the level of force and deformation demand placed on shear wall components. Peak drifts in the range of 0.005h are nearly elastic, while drifts in excess of 0.02h can be anticipated to create significant nonlinear demands. Where nonlinear demand is expected, design of shear wall boundary members and collectors for overstrength is an approach to ensuring that energy dissipation is primarily focused in the sheathing to framing

Limitations

When reviewing and using the results of this analysis, it needs to be kept in mind that:

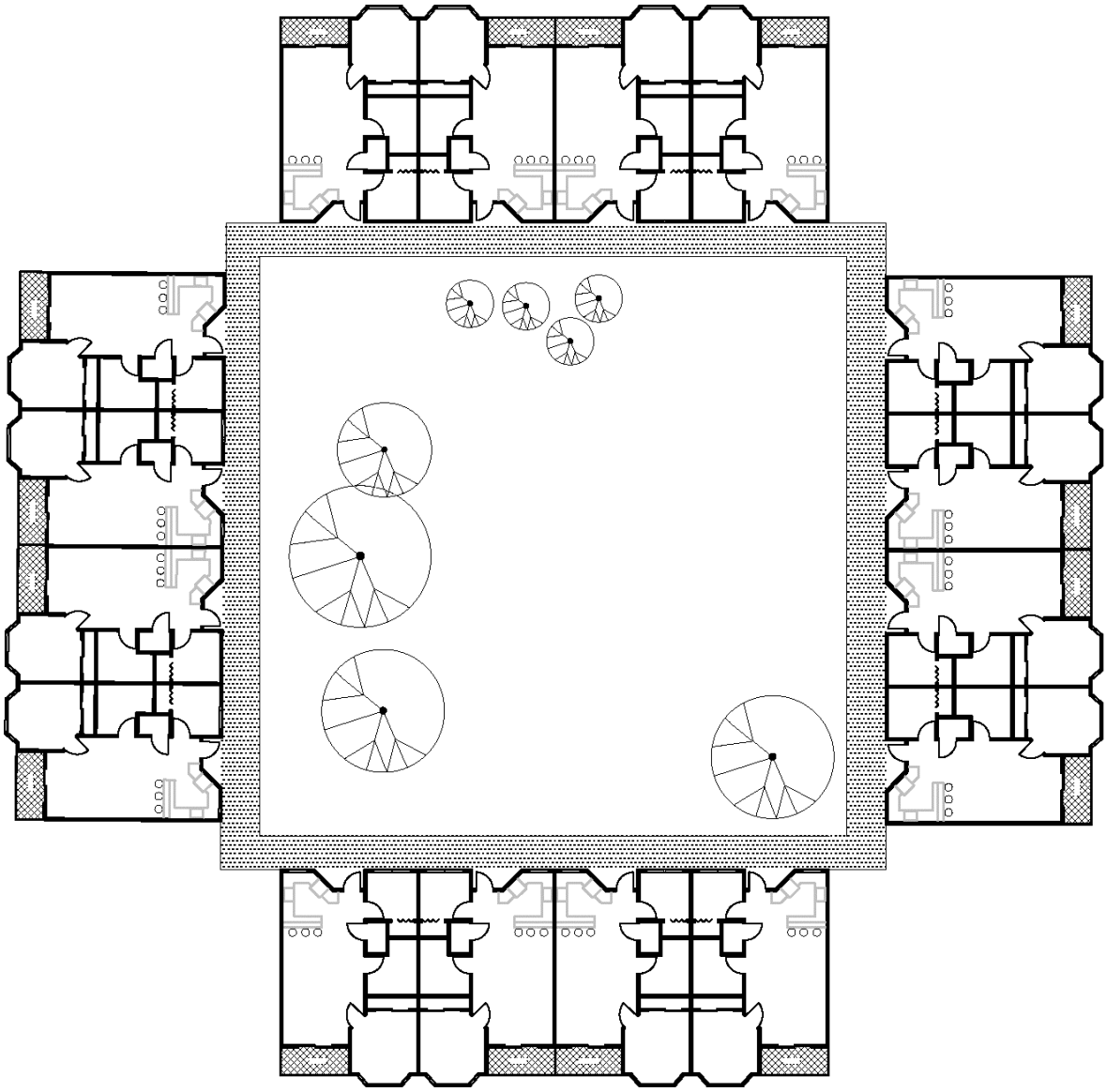
1) Considerable judgment was used in arriving at the hysteretic parameters and building models used in the analysis,
2) Only a single ground motion has been used for this comparison; other ground motions could have significantly different results, and
3) Only a single building has been used for this analysis; other buildings could have significantly different results.

This analysis servers as one illustration of the analysis methodology and results for the particular building model and ground motion. Conclusions should not be drawn without further study.

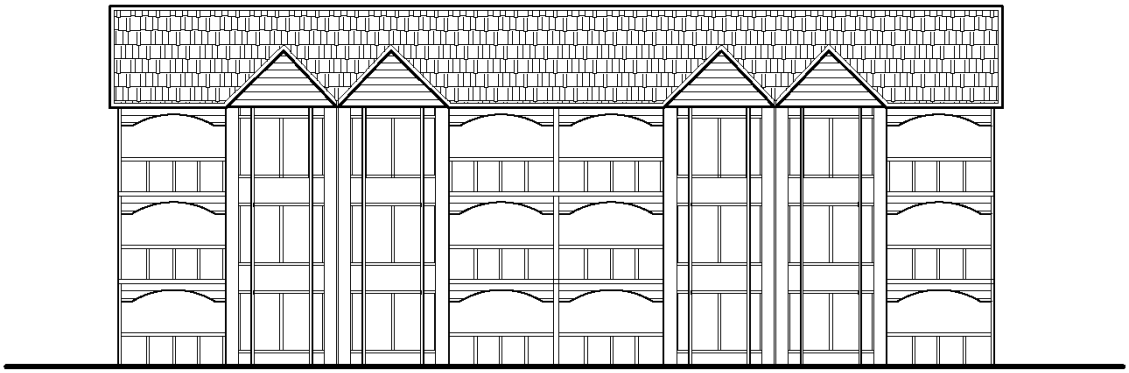


UNIT PLAN
NOT TO SCALE

fasteners.

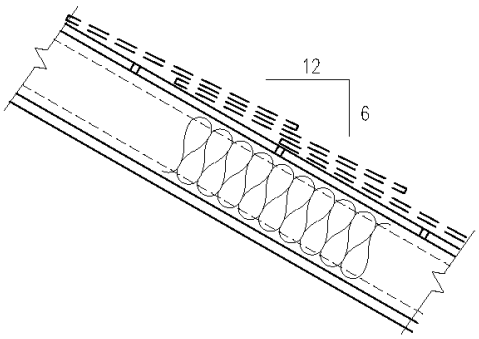


CLUSTER PLAN
NOT TO SCALE



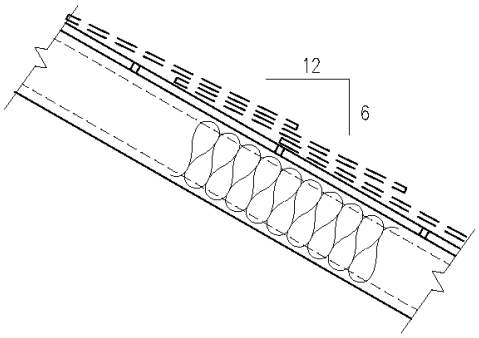
ELEVATION
3 STORY RESIDENTIAL
NOT TO SCALE

ROOF WITH CATHEDRAL CEILING



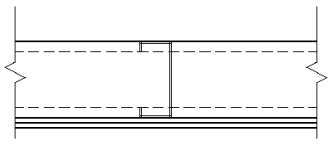
	PSF
COMP SHINGLE	3.5
REROOF	3.0
5/8 OR 19/32" WOOD STRUCTURAL PANEL SHEATHING	1.9
R30 BATT INSULATION	1.4
600C162, 33 MIL	0.6
BLK @ 8'	0.6
5/8" GYP.	3.1
MISC	0.3
TOTAL	14.4
ADJUST FOR SLOPE X 1.12	16.1
USE	17

TRUSS ROOF WITHOUT CEILING



	PSF
COMP SHINGLE	3.5
REROOF	3.0
5/8 OR 19/32" WOOD STRUCTURAL PANEL SHEATHING	1.9
R30 BATT INSULATION	1.4
TRUSS @ 24	0.7
BLK @ 8'	0.6
MISC	0.4
TOTAL	12.1
ADJUST FOR SLOPE X1.12	13.6
USE	14.0

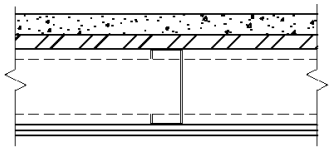
CEILING AT TRUSSES



	PSF
TRUSS @ 24	0.7
BLKG	0.6
2 X 5/8" GYPSUM WALLBOARD	6.3
MISC	0.4
TOTAL	8.0
USE	8

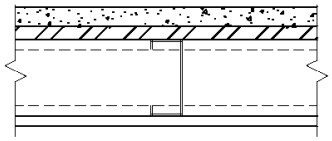
7/6/05

RESIDENCE FLOOR



	PSF
FINISH	1.0
1 1/2" LT. WT. CONC.	15.0
¾" OR 23/32" COMBINED SUB–FLOOR & UNDERLAYMENT	2.3
JOIST @ 16"	2.0
BLKG & BEAMS	1.0
5/8" GYPBD	3.1
MISC/ MEP/ HUNG SOFFIT	1.6
TOTAL	26
USE	26

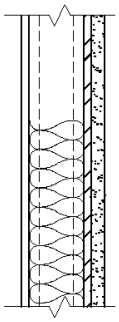
DECK AND COMMON AREA FLOOR



	PSF
1 1/2" LT. WT. CONC.	15.0
¾" OR 23/32" COMBINED SUB–FLOOR & UNDERLAYMENT	2.3
JOIST @ 16"	2.0
BLKG & BEAMS	1.0
STUCCO	10.0
MISC	1.3
TOTAL	33
USE	33

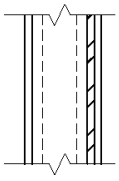
7/6/05

EXTERIOR WALL WITH WSP BRACING



	PSF
7/8" 3–COAT STUCCO	10.0
1/2" OR 15/32" WOOD STRUCTURAL PANEL SHEATHING	1.5
350S162, 33 STUDS@16"	0.7
INSUL	0.8
TOP & BOTT PLATES	0.3
5/8" GYPBD	3.1
MISC	0.9
TOTAL	17.0
USE	17

INTERIOR WALL



	PSF
350S162, 33 STUDS@16"	0.7
1/2" OR 15/32" WOOD STRUCTURAL PANEL SHEATHING	1.5
TOP & BOTT PLATES	0.3
2 X 5/8" GYPBD	6.2
MISC	0.9
TOTAL	9.5
USE W/ PLWD	10
USE W/O PLWD	8

7/6/05

BUILDING WEIGHT SUMMARY									
Stucco Wall Finish									
End Unit									
Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #	
Sloped Ceiling	typ				390	17.0	6,630	6,630	**Roof+Ceil**
Flat Roof/Ceil	bath,kitch,				673	22.0	14,806	14,806	21,436
Walls 2-R	Line 1	Total	37	9	333			5,301	
	Ext	window/door			30	5.0	150		
		typ exterior			303	17.0	5,151		
	Line 2	Total	32	9	288			2,304	
	Int Party	open & door			0	2.0	0		
		interior party			288	8.0	2,304		
	Lines 1-2	Total	37	9	333			2,214	
	Int Partition	open & door			75	2.0	150		
		typ int			258	8.0	2,064		
	Line A	Total	27	7	189			2,625	
	Ext	open & door			49	5.0	245		
		typ exterior			140	17.0	2,380		
	Line C	Total	32	9	288			3,936	
	Ext	open & door			80	5.0	400		
		typ exterior			208	17.0	3,536		
									Walls 2-R
	Lines A-C	interior	40	9	360			2,448	18,828
					72	2.0	144		
					288	8.0	2,304		
2nd FL	Deck				60	33.0	1980	1980	
	Unit				860	26.0	22360	22360	**2nd FL**
	Walkway				146	33.0	4818	4818	29158
Walls 1-2	Line 1	Total	37	9	333			5,301	
	Ext	window/door			30	5.0	150		
		typ exterior			303	17.0	5,151		
	Line 2	Total	32	9	288			2,304	
	Party	open & door			0	2.0	0		
		typ interior			288	8.0	2,304		
	Lines 1-2	Total	37	9	333			2,214	
	Partition	open & door			75	2.0	150		
		typ interior			258	8.0	2,064		
	Line A	Total	27	9	243			3,543	
	Exterior	open & door			49	5.0	245		
		typ exterior			194	17.0	3,298		
	Line C	Total	32	9	288			3,936	
	Exterior	window/door			80	5.0	400		
		typ interior			208	17.0	3,536		
	Lines A-C	Total	40	9	360			2,448	
	Partition	window/door			72	2.0	144		**Walls 1-2**
		typ exterior			288	8.0	2,304		19,746

Sum of Weight for Seismic Forces (lb)		
Roof/Ceiling		
Roof + Ceiling		21436
1/2 walls 2-R		9414
Sum		**30850**
Second Floor		
Second Floor		29158
1/2 walls 2-R		9414
1/2 walls 1-2		9873
Sum		**48445**
At Grade		
1/2 walls 1-2		9873
Sum		**9873**
Sum 2nd & Roof		**79295**
Sum at Grade		**89168**

Primary & Diaphragm Vertical Distribution - Strength Level Forces

Level	W (K)	h (ft)	Wxh	Wxh/Sum	Str V (k)	Sum V (k)	SumV/Sum w	Fpx
Roof	30.85	24.5	756	0.634	7.74	7.74	0.251	7.74
Second	48.45	9.0	436	0.366	4.46	12.20	0.154	7.45
Sum	79.30		1192	1.0				

2003 IBC			
Assume Site Class D			
SS	1.5	Max for regular str. 9.5.5.2.1	
Fa	1		
SMS	1.5		
SDS	1		
Rho	1		
Eh	$\rho*S_{DS}/R$		
	0.154	Strength Level	
	0.108	ASD Level= 0.7*Str	
S1	0.6		
Fv	1.5		
SM1	0.9		
SD1	0.6		
T	0.28	SEC	
Rho	1		ASCE 7-05
R	6.5		Table 12.2-1
Eh	$\rho*S_{D1}/(R*T)$		
	0.330	but not more than 0.165	

BUILDING WEIGHT SUMMARY									
Stucco Wall Finish									
End Unit									
Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #	
Sloped Ceiling	typ				390	17.0	6,630	6,630	**Roof+Ceil**
Flat Roof/Ceil	bath,kitch,				673	22.0	14,806	14,806	21,436
Walls 3-R	Line 1	Total	37	9	333			5,301	
	Ext	window/door			30	5.0	150		
		typ exterior			303	17.0	5,151		
	Line 2	Total	32	9	288			2,304	
	Int Party	open & door			0	2.0	0		
		interior party			288	8.0	2,304		
	Lines 1-2	Total	37	9	333			2,214	
	Int Partition	open & door			75	2.0	150		
		typ int			258	8.0	2,064		
	Line A	Total	27	7	189			2,625	
	Ext	open & door			49	5.0	245		
		typ exterior			140	17.0	2,380		
	Line C	Total	32	9	288			3,936	
	Ext	open & door			80	5.0	400		
		typ exterior			208	17.0	3,536		
									Walls 3-R
	Lines A-C	interior	40	9	360			2,448	18,828
					72	2.0	144		
					288	8.0	2,304		
2nd or 3rd Fl	Deck				60	33.0	1980	1980	
	Unit				860	26.0	22360	22360	**2nd, 3rd Fl**
	Walkway				146	33.0	4818	4818	29158
Walls 1-2	Line 1	Total	37	9	333			5,301	
	Ext	window/door			30	5.0	150		
		typ exterior			303	17.0	5,151		
	Line 2	Total	32	9	288			2,304	
	Party	open & door			0	2.0	0		
		typ interior			288	8.0	2,304		
	Lines 1-2	Total	37	9	333			2,214	
	Partition	open & door			75	2.0	150		
		typ interior			258	8.0	2,064		
	Line A	Total	27	9	243			3,543	
	Exterior	open & door			49	5.0	245		
		typ exterior			194	17.0	3,298		
	Line C	Total	32	9	288			3,936	
	Exterior	window/door			80	5.0	400		
		typ interior			208	17.0	3,536		
	Lines A-C	Total	40	9	360			2,448	
	Partition	window/door			72	2.0	144		**Walls 1-2, 2-3**
		typ exterior			288	8.0	2,304		19,746

Sum of Weight for Seismic Forces (lb)		
Roof/Ceiling		
Roof + Ceiling		21436
1/2 walls 2-R		9414
Sum		**30850**
Third Floor		
Third Floor		29158
1/2 walls 3-R		9414
1/2 walls 2-3		9873
Sum		**48445**
Second Floor		
Second Floor		29158
1/2 walls 2-3		9873
1/2 walls 1-2		9873
Sum		**48904**
At Grade		
1/2 walls 1-2		9873
Sum		**9873**
Sum 2nd, 3rd & Roof		**128199**
Sum at Grade		**138072**

Primary & Diaphragm Vertical Distribution - Strength Level Forces

Level	W (K)	h (ft)	Wxh	Wxh/Sum	Str V (k)	Sum V (k)	SumV/Sum w	Fpx
Roof	30.85	34.5	1064	0.439	8.66	8.66	0.281	8.66
Third	48.45	19.0	920	0.380	7.49	16.14	0.204	9.86
Second	48.90	9.0	440	0.182	3.58	19.72	0.154	7.52
Sum	128.20		2425	1.0				

2003 IBC			
Assume Site Class D			
SS	1.5	Cap per Sec. 9.5.5.2.1	
Fa	1		
SMS	1.5		
SDS	1		
Rho	1		
Eh	ρ^*S_{DS}/R		
	0.154	Strength Level	
	0.108	ASD Level= 0.7*Str	
S1	0.6		
Fv	1.5		
SM1	0.9		
SD1	0.6		
T	0.28	SEC	
Rho	1		ASCE 7-05
R	6.5		Table 12.2-1
Eh	$\rho^*S_{D1}/(R^*T)$		
	0.330	but not more than 0.165	

Shear Wall Selection - Two Story Building

Story	Line	Percent Story Shear	Shear Force (k)	Wall Length (ft)	Unit Shear ASD (plf)	Required Nominal Strength (plf)	Shear Wall				
							Sheathing	Stud Thickness (mils)	Fastener Size (Screw Number)	Fastener Spacing (in)	Assigned Nominal Strength (plf)
2	**A**	39	3.02	5	604	1509	7/16 OSB	43	8	3	1545
V =	**B**	37	2.86	9.5	301	754		33		4	915
7.74	**C**	24	1.86	6	310	774		33		4	915
	1	50	3.87	20	194	484		33		6	700
	2	50	3.87	20	194	484		33		6	700
1	**A**	39	4.76	9.5	501	1252		43		3	1545
V =	**B**	37	4.51	9.5	475	1188		43		4	1235
12.2	**C**	24	2.93	6	488	1220		43		4	1235
	1	50	6.10	20	305	763		33		4	915
	2	50	6.10	20	305	763		33		4	915

Shear Wall Selection - Three Story Building

Story	Line	Percent Story Shear	Shear Force (k)	Wall Length (ft)	Unit Shear ASD (plf)	Required Nominal Shear (plf)	Shear Wall				
							Sheathing	Stud Thickness (mils)	Fastener Size	Fastener Spacing (in)	Assigned Nominal Strength (plf)
3	**A**	39	3.38	5	675	1689	7/16 OSB	43	8	2	2060
V =	**B**	37	3.20	9.5	337	843		33		4	915
8.66	**C**	24	2.08	6	346	866		33		4	915
	1	50	4.33	20	217	541		33		6	700
	2	50	4.33	20	217	541		33		6	700
2	**A**	39	6.29	9.5	663	1656		43		2	2060
V =	**B**	37	5.97	9.5	629	1572		43		2	2060
16.14	**C**	24	3.87	6	646	1614		43		2	2060
	1	50	8.07	20	404	1009		43		3	1235
	2	50	8.07	20	404	1009		43		3	1235
1	**A**	39	7.69	9.5	810	2024		43		2	2060
V =	**B**	37	7.30	9.5	768	1920		43		2	2060
19.72	**C**	24	4.73	6	789	1972		43		2	2060
	1	50	9.86	20	493	1233		43		3	1235
	2	50	9.86	20	493	1233		43		3	1235

Wood Shear Wall Selection - Two Story Building

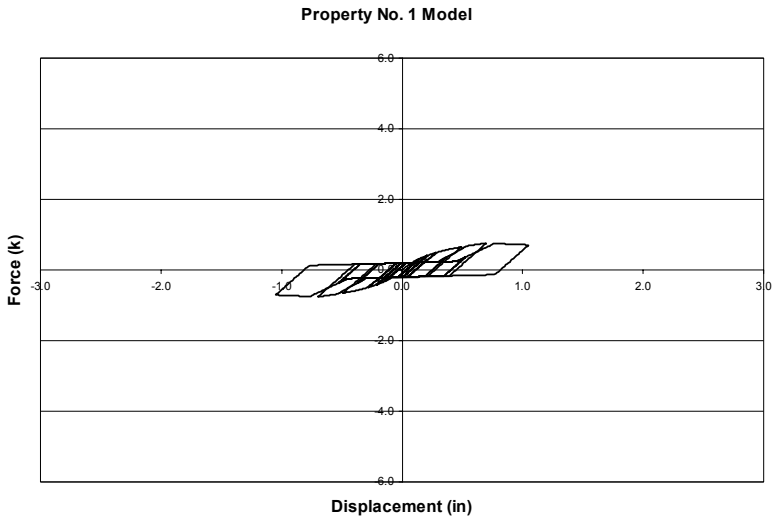
Story	Line	Percent Story Shear	Shear Force (k)	Wall Length (ft)	Unit Shear ASD (plf)	Required Nominal Strength (plf)	Shear Wall				
							Sheathing	Stud	Fastener Size (Nail Penny Weight)	Fastener Spacing (in)	Assigned Nominal Strength (plf)
2	**A**	39	3.02	5	604	1207	3/8 OSB	NA	8d	2	1280
V =	**B**	37	2.86	9.5	301	603				4	760
7.74	**C**	24	1.86	6	310	619				4	760
	1	50	3.87	20	194	387				6	520
	2	50	3.87	20	194	387				6	520
1	**A**	39	4.76	9.5	501	1002				2	1280
V =	**B**	37	4.51	9.5	475	950				3	980
12.2	**C**	24	2.93	6	488	976				3	980
	1	50	6.10	20	305	610				4	760
	2	50	6.10	20	305	610				4	760

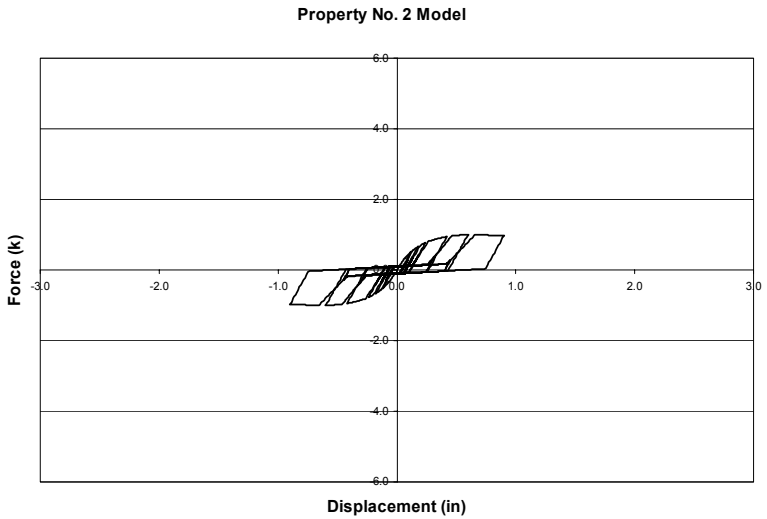
Wood Shear Wall Selection - Three Story Building

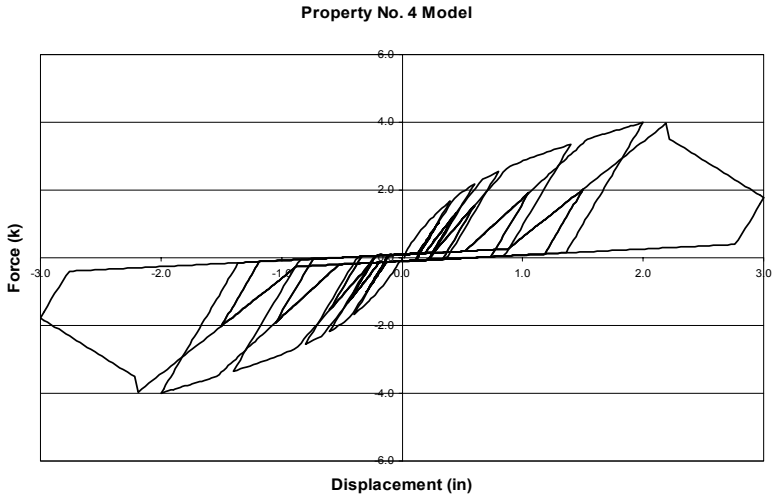
Story	Line	Percent Story Shear	Shear Force (k)	Wall Length (ft)	Unit Shear ASD (plf)	Required Nominal Shear (plf)	Shear Wall				
							Sheathing	Stud	Fastener Size	Fastener Spacing (in)	Assigned Nominal Strength (plf)
3	**A**	39	3.38	5	675	1351	15/32 STR I	NA	10d	2	1740
V =	**B**	37	3.20	9.5	337	675	3/8 OSB		8d	4	760
8.66	**C**	24	2.08	6	346	693	3/8 OSB		8d	4	760
	1	50	4.33	20	217	433	3/8 OSB		8d	6	520
	2	50	4.33	20	217	433	3/8 OSB		8d	6	520
2	**A**	39	6.29	9.5	663	1325	15/32 STR I		10d	3	1330
V =	**B**	37	5.97	9.5	629	1257	3/8 OSB		8d	2	1280
16.14	**C**	24	3.87	6	646	1291	3/8 OSB		8d	2	1280
	1	50	8.07	20	404	807	3/8 OSB		8d	3	980
	2	50	8.07	20	404	807	3/8 OSB		8d	3	980
1	**A**	39	7.69	9.5	810	1619	15/32 STR I		10d	2	1740
V =	**B**	37	7.30	9.5	768	1536	15/32 STR I		10d	2	1740
19.72	**C**	24	4.73	6	789	1578	15/32 STR I		10d	2	1740
	1	50	9.86	20	493	986	3/8 OSB		8d	3	980
	2	50	9.86	20	493	986	3/8 OSB		8d	3	980

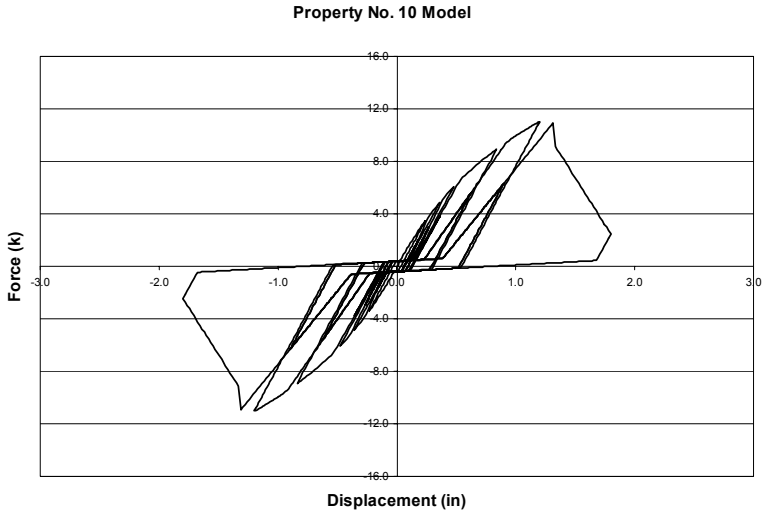
Hysteretic Parameters Used For Non-linear Dynamic Analysis

Property No.	Use	Test Basis	Fo (k)	Fl (k)	Δ_u (in)	So (Ko) (k/in)	r1	r2	r3	r4	alpha	beta	Δ_{CUREE} (in)	Δ_{CUREE} (in)	Fu (k)
Basic Model Properties[1]															
STEEL															
1	gypsum wallboard lower bound	Toothman Test 23	0.50	0.20	0.70	3.0	0.130	-0.050	1.000	0.030	0.40	1.10			0.74
2	gypsum wallboard upper bound	McMullin Test 11	1.00	0.10	0.60	6.0	0.010	-0.020	1.000	0.030	0.40	1.10			1.04
3	stucco	Pardoen Group 15	2.80	0.20	2.20	12.3	0.011	-0.400	1.000	0.030	0.40	1.10			3.10
4	OSB 33 8 6	LGSRG-3-96 AISI-OSB2	2.00	0.10	2.00	6.0	0.167	-0.370	1.000	0.030	0.40	1.10			4.00
5	OSB 33 8 4	LGSRG-3-96 AISI-OSB3	4.00	0.10	2.20	4.4	0.100	-1.100	1.000	0.030	0.40	1.10			5.20
6	OSB 43 8 6	interpolated	3.00	0.20	1.80	9.5	0.076	-0.850	1.000	0.030	0.40	1.10			4.30
7	OSB 43 8 4	interpolated	4.40	0.20	1.60	13.0	0.096	-0.850	1.000	0.030	0.40	1.10			6.40
8	OSB 43 8 3	interpolated	5.50	0.20	1.40	17.5	0.102	-0.850	1.000	0.030	0.40	1.10			8.00
9	OSB 43 8 2	LGSRG-06-02 Test 1	7.20	0.20	1.20	20.8	0.128	-0.850	1.000	0.030	0.40	1.10			10.40
10	OSB 68 10 2	LGSRG-06-02 Test 4	8.00	0.40	1.20	16.8	0.198	-0.850	1.000	0.030	0.40	1.10			12.80
11	w/o OSB 33 8 6		1.80		1.60		0.150								
12	w/o OSB 33 8 4		3.60		1.76		0.090								
13	w/o OSB 43 8 6		2.70		1.44		0.068								
14	w/o OSB 43 8 4		3.96		1.28		0.086								
15	w/o OSB 43 8 3		4.95		1.12		0.092								
16	w/o OSB 43 8 2		6.48		0.96		0.115								
17	w/o OSB 68 10 2		7.20		0.96		0.178								
WOOD															
18	3/8 OSB 8d@6	Pardoen Test 6	2.20	0.50	2.00	6.0	0.070	-0.200	1.000	0.030	0.40	1.10			3.00
19	3/8 OSB 8d@4	Interpolated	3.20	0.50	2.00	6.0	0.100	-0.200	1.000	0.030	0.40	1.10			4.40
20	3/8 OSB 8d@3	Interpolated	4.20	0.50	2.00	6.0	0.130	-0.200	1.000	0.030	0.40	1.10			5.80
21	3/8 OSB 8d@2	Interpolated	5.20	0.50	2.00	6.0	0.170	-0.200	1.000	0.030	0.40	1.10			7.20
22	15/32 STR I 10d@6	CoLA Group 3	3.00	0.50	2.00	8.0	0.012	-0.500	1.000	0.030	0.40	1.10			3.20
23	15/32 STR I 10d@4	CoLA Group 4	3.50	0.50	2.00	8.0	0.044	-0.500	1.000	0.030	0.40	1.10			4.20
24	15/32 STR I 10d@3	Interpolated	4.80	0.50	2.00	8.0	0.069	-0.500	1.000	0.030	0.40	1.10			5.90
25	15/32 STR I 10d@2	CoLA Group 9	6.00	0.50	2.00	8.0	0.093	-0.500	1.000	0.030	0.40	1.10			7.50


Property No. 1 Model
Force (k)
Displacement (in)
6.0
4.0
2.0
-2.0
-4.0
-6.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0


Property No. 2 Model
Force (k)
Displacement (in)
6.0
4.0
2.0
-2.0
-4.0
-6.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0


Property No. 4 Model
Force (k)
Displacement (in)
6.0
4.0
2.0
-2.0
-4.0
-6.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0


Property No. 10 Model
Force (k)
Displacement (in)
16.0
12.0
8.0
4.0
-4.0
-8.0
-12.0
-16.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0

Peak Shear Wall Drifts - Inches

Seismic Parameter Analysis Summary of Story Drifts																
wo2lbx Steel 2-Story Lower Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line B**	**Line B**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 2**	**Line 2**					
	Location	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**					
	Min	-0.281	-0.321	-0.407	-0.312	-0.513	-0.376	-0.111	-0.079	-0.109	-0.066					
	Max	0.359	0.319	0.532	0.351	0.645	0.393	0.140	0.081	0.098	0.060					
wo2ubx Steel 2-Story Upper Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line B**	**Line B**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 2**	**Line 2**					
	Location	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**					
	Min	-0.240	-0.275	-0.353	-0.300	-0.466	-0.371	-0.124	-0.118	-0.084	-0.069					
	Max	0.270	0.231	0.330	0.307	0.420	0.381	0.138	0.114	0.076	0.066					
woo2lbx Steel 2-Story Lower Bound Gyp Undersize Boundary		**Line A**	**Line A**	**Line B**	**Line B**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 2**	**Line 2**					
	Location	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**					
	Min	-0.281	-0.329	-0.406	-0.315	-0.507	-0.368	-0.108	-0.076	-0.110	-0.064					
	Max	0.360	0.328	0.526	0.361	0.635	0.401	0.128	0.074	0.099	0.055					
wood2lbx Wood 2-Story Lower Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line B**	**Line B**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 2**	**Line 2**					
	Location	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**					
	Min	-0.409	-0.280	-0.485	-0.288	-0.534	-0.293	-0.050	-0.040	-0.054	-0.029					
	Max	0.606	0.289	0.704	0.283	0.768	0.279	0.060	0.036	0.048	0.042					
wood2ubx Wood 2-Story Upper Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line B**	**Line B**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 2**	**Line 2**					
	Location	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**	**1st-2nd**	**2nd-Roof**					
	Min	-0.370	-0.249	-0.423	-0.274	-0.463	-0.296	-0.051	-0.036	-0.036	-0.022					
	Max	0.415	0.283	0.456	0.294	0.482	0.301	0.053	0.028	0.039	0.025					
wo3lbx Steel 3-Story Lower Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line A**	**Line B**	**Line B**	**Line B**	**Line C**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 1**	**Line 2**	**Line 2**	**Line 2**
	Location	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**
	Min	-0.466	-0.393	-0.368	-0.521	-0.470	-0.503	-0.558	-0.520	-0.590	-0.070	-0.067	-0.099	-0.073	-0.064	-0.063
	Max	0.694	0.439	0.507	0.786	0.492	0.600	0.847	0.528	0.662	0.074	0.056	0.069	0.055	0.071	0.074
wo3ubx Steel 3-Story Upper Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line A**	**Line B**	**Line B**	**Line B**	**Line C**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 1**	**Line 2**	**Line 2**	**Line 2**
	Location	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**
	Min	-0.436	-0.512	-0.485	-0.511	-0.643	-0.653	-0.566	-0.729	-0.766	-0.059	-0.080	-0.129	-0.060	-0.060	-0.078
	Max	0.657	0.393	0.380	0.743	0.517	0.451	0.812	0.598	0.509	0.071	0.078	0.089	0.052	0.077	0.113
woo2lbx Steel 3-Story Lower Bound Gyp Undersize Boundary		**Line A**	**Line A**	**Line A**	**Line B**	**Line B**	**Line B**	**Line C**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 1**	**Line 2**	**Line 2**	**Line 2**
	Location	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**
	Min	-0.462	-0.345	-0.335	-0.531	-0.422	-0.452	-0.596	-0.488	-0.531	-0.062	-0.073	-0.097	-0.082	-0.053	-0.072
	Max	0.863	0.415	0.401	1.009	0.507	0.533	1.104	0.571	0.620	0.088	0.055	0.075	0.061	0.082	0.082
wood3lbx Wood 3-Story Lower Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line A**	**Line B**	**Line B**	**Line B**	**Line C**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 1**	**Line 2**	**Line 2**	**Line 2**
	Location	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**
	Min	-0.563	-0.588	-0.437	-0.657	-0.691	-0.374	-0.718	-0.760	-0.425	-0.141	-0.120	-0.063	-0.167	-0.116	-0.070
	Max	1.366	0.710	0.271	1.707	0.898	0.402	1.942	1.020	0.492	0.226	0.127	0.082	0.103	0.091	0.063
wo3ubx Wood 3-Story Upper Bound Gyp ASD Boundary		**Line A**	**Line A**	**Line A**	**Line B**	**Line B**	**Line B**	**Line C**	**Line C**	**Line C**	**Line 1**	**Line 1**	**Line 1**	**Line 2**	**Line 2**	**Line 2**
	Location	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**	**1st-2nd**	**2nd-3rd**	**3rd-4th**
	Min	-0.619	-0.611	-0.443	-0.802	-0.834	-0.411	-0.934	-0.999	-0.391	-0.142	-0.169	-0.049	-0.144	-0.132	-0.050
	Max	1.267	0.775	0.332	1.534	1.037	0.388	1.746	1.213	0.425	0.224	0.177	0.085	0.094	0.107	0.045

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