AN ABSTRACT OF THE DISSERTATION OF

William J. Kirkham for the degree of Doctor of Philosophy in Wood Science and Civil Engineering presented on August 21, 2013.

Title: Examination of Lateral Stiffness and Strength of Pitched Residential Roof Diaphragms with Implications for Seismic Design

Abstract approved:

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There are about 80 million single-family dwellings (SFD) in the United States, predominantly of wood-frame construction. Of these, 68% are owner occupied. A home is typically the largest single investment of a family, and is often not covered by earthquake insurance, even where it is available. Of all SFD in America, 50% were built before 1974, and 76% built before 1990. Most wood frame SFD (WFSFD) were built to prescriptive code provisions before seismic requirements were introduced.

This dissertation examines a broad field of seismic research applicable to WFSFD seismic analysis and the design, and to the retrofit of existing WFSFD. It summarizes the "state-of-the-art" of seismic experimentation and seismic evaluation, and provides observations and recommendations for future research. The study showed that performance based design (PBD) needs to develop a consensus on performance

objectives for WDSFD with respect to damage and repair costs, including new design techniques which balance life-safety with the effects of damage to building finish materials. There is also a strong need to develop a better understanding of the effects of WDSFD components, attachment, the lateral force resisting system and how loads are distributed to these elements within the structure.

To address concerns with limited pitched light-frame roof diaphragm experiments, ten full size (3.7 x 4.9 m) plywood roof diaphragms with metal plate connected (MPC) common and hip wood trusses or joists, typical of WFSFD construction were tested according to ASTM E455. Specimens included three gable roof slopes of 33, 67 and 100%, a hip roof of 33% slope, and a flat roof. Gable and hip roofs experiments examined the effect of eave sheathing and gypsum ceilings on the bottom chord. Results showed eave plywood had negligible effect on diaphragm apparent stiffness; gable roofs had apparent stiffnesses that were about 50% that of the flat roofs; and gypsum provided more than 1/3 of the total roof apparent stiffness for gable roofs at slopes of less than 33%. There was no effect of pitch on ultimate roof strength in any configuration and all exhibited approximately the same ultimate shear strength. Failure modes of roofs included nail withdrawal, nail tear-through, metal plate tear-out on trusses and chord tensile failure.

This dissertation also examines code provisions applicable to WFSFD seismic design, compares rigid and flexible diaphragm analyses for different geometries of “L” shaped WFSFD applying stiffness reductions due to differing roof geometry and pitch. These analyses are applied to historic earthquake damage reports and compared with a practical rigid, semi-rigid or flexible diaphragm plate FEM analysis method. This study determined that most WFSFD should be designed using an envelope method due to a mix of diaphragm types and the effects of roof pitch and geometry on the stiffness. Cases may occur where determination of semi-rigid or flexible diaphragm behavior is difficult because the code prescribed analysis is contradictory or fallacious. This suggests that use of semi-rigid finite element model (FEM) or a manual envelope method is prudent. The use of RP, FP or SP FEM methods can be simple and practical methods for analyzing WFSFD with a reasonable level of detail and effort.

EXAMINATION OF LATERAL STIFFNESS AND STRENGTH OF PITCHED RESIDENTIAL ROOF DIAPHRAGMS WITH IMPLICATIONS FOR SEISMIC DESIGN

by

William J. Kirkham

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I understand that my dissertation will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my dissertation to any reader upon request.

William J. Kirkham, Author

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CONTRIBUTION OF AUTHORS

Dr. Rakesh Gupta and Dr. Thomas H. Miller were involved in all aspects of the work leading to this dissertation including advising on data collection, testing, analysis and help in writing the manuscripts.

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Chapter 1. General Introduction

Most of the building structures in the United States and Canada are single-family dwellings (SFD). Like other structures, they vary in size, configuration, age and condition. These factors affect their use as habitable dwellings and also their performance in natural disasters. Predecessors of the International Building Code (IBC) (International Code Council, 2006a) and the International Residential Code (IRC) (International Code Council, 2006b) have addressed seismic issues in past editions. However, according to the American Housing Survey of the US Census Bureau (2001), most wood-frame single-family dwellings (WFSFD) were constructed before these seismic provisions were introduced during the 1970s and 1980s. More than half of the existing housing inventory was constructed before 1970. Seventy-five percent of the inventory was constructed before 1985. People spend more than one- third of their lives in these structures, usually asleep and not prepared to react to a potential disaster. The seismic performance of WFSFD is of vital importance.

Much design information on wood-frame construction is based on educated opinion or limited research. This paper is concerned only with conventional construction materials and methods that are used within the United States and Canada. There are methods of construction that are common in Africa, Asia, Australia, Europe and South America which are not used in the United States and Canada. Without discussion of the relative merits of these systems, this study has concentrated on the traditional methods used in the United States and Canada. Unique systems have been and are being developed in the United States and Canada which involve special materials or component mechanics. Some of these systems are proposed, but not in regular use. Most of these systems are considered to be experimental and are excluded from this review.

The objectives of this thesis are to:

1. Review the current state-of-the-art with respect to WFSFD design and research for conventional materials and methods of construction,

2. Review those methods that are not conventional, but may be readily adopted for use in SFD,

3. Locate research areas where potential for additional research or improvement exist;

4. Provide conclusions and recommendations for future research in seismic behavior of WFSFD,

5. Determine whether roof pitch has any effect on roof diaphragm apparent stiffness or strength,

6. Determine whether hip roofs have the same strength and apparent stiffness as gable roofs of the same pitch,

7. Determine whether roof diaphragm strength or apparent stiffness is increased by the application of gypsum ceiling, and how differing roof pitches affect this apparent stiffness?

8. Determine how effective roof eave sheathing is when compared with the remainder of the sheathing,

9. Examine the effects of roof pitch on lateral force resisting system (LFRS) design of WFSFD,

10. Examine existing code provisions with respect to LFRS design in WFSFD to determine areas of concern where strict application of the provisions could be misleading, unconservative or ambiguous,

11. Compare the Tributary Area Method (flexible diaphragm analysis), rigid diaphragm analysis by hand, rigid plate FEM, and flexible plate FEM to test results and selected seismic damage reports,

12. Develop a simple, practical method to evaluate diaphragms of differing stiffness and to consider torsional effects for WFSFD.

ORGANIZATION

The results of this study are presented in three manuscripts. Supporting data and tables for these manuscripts are presented in a series of supporting appendices. The first manuscript (Chapter 2) is "State-of-the-Art: Seismic Behavior of Wood-Frame

Residential Structures." The manuscript is intended to cover most readily available papers published in major U.S. journals and at major conferences in the area of seismic modeling, testing and evaluation of WFSFD. The "state-of-the-art" of seismic experimentation and seismic evaluation is discussed, and observations and recommendations for future research are provided. The second manuscript (Chapter 3) is "Effects of Roof Pitch and Gypsum Ceilings on the Behavior of Wood Roof Diaphragms." It presents the results of experiments on ten full size (3.7 x 4.9 m) plywood roof diaphragms, constructed using metal plate connected (MPC) common and hip wood trusses or joists, typical of WFSFD construction. The specimens included three gable roof slopes of 33, 67 and 100%, a hip roof of 33% slope, and a flat roof, with a horizontal bottom chord. These roofs were constructed and tested in duplicate to make a total of ten roofs. The third manuscript (Chapter 4) is "Practical Analysis Method for Partial Diaphragm Rigidity and Torsion in Wood Frame Single Family Dwellings." It describes the results from an analytical study of the effects of pitch and roof geometry on the design of the LFRS of WFSFD, and demonstrates a practical method to design WFSFD. The methods are compared to observed earthquake damage from the 1994 Northridge Earthquake on WFSFD. The appended information is intended to supplement the manuscripts.

State-of-the-Art: Seismic Behavior of Wood-Frame Residential Structures

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Chapter 2. State-of-the-Art: Seismic Behavior of Wood-Frame Residential Structures

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Abstract

There are about 80 million single-family dwellings (SFD) in the United States, predominantly of wood-frame construction. Of these, 68% are owner occupied. A home is typically the largest single investment of a family, and is often not covered by earthquake insurance, even where it is available. Of all the houses in America, 50% were built before 1974, and 76% built before 1990. Most wood-frame SFD (WFSFD) were built to prescriptive code provisions before seismic requirements were introduced. After the introduction of seismic design requirements, the importance of examining structures as an assembly of connected elements became more common. Much of the seismic design information on SFD construction is based on educated opinion or limited research. This review examines research that can be applied to WFSFD seismic analysis and the design and retrofit of existing WFSFD. The review is intended to cover most readily available papers published in major U.S. journals and at major conferences in the area of seismic modeling, testing and evaluation. We review the "state-of-the-art" of seismic experimentation and seismic evaluation, and provide our observations and recommendations for future research.

CE Database subject headings: Shearwalls; Diaphragms; Roofs; Wood Structures; Wood; Bibliographies; Seismic.

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INTRODUCTION

Most of the building structures in the United States and Canada are single-family, residential dwellings (Ni et al. 2010) . Like other structures, these buildings vary in size, configuration, age and condition. These factors affect their use as habitable dwellings and also their performance in natural disasters. Predecessors of the International Building Code (IBC) (International Code Council, 2011a) and the International Residential Code (IRC) (International Code Council, 2011b) have addressed seismic issues in past editions. However, according to the American Housing Survey of the US Census Bureau (2011), most single-family dwellings (SFD) were constructed before these seismic provisions were introduced during the 1970s and 1980s. More than half of the existing housing inventory was constructed before 1974. Seventy-five percent of the inventory was constructed before 1990. About 70% of the housing inventory are individual dwellings. People spend more than 1/3 of their lives in these structures, usually asleep and not prepared to react to a potential disaster. The seismic performance of wood-frame, single-family dwellings (WFSFD) is of vital importance to many of us.

The Association of Bay Area Governments commissioned a study (Perkins 1996) which indicates that reasonably expected major earthquakes of approximately magnitude 7 could result in over 150,000 uninhabitable housing units and 360,000 people made homeless as a result. The 1994 Northridge Earthquake Buildings Case Studies Project (Holmes et. al 1996) examined a two-story house with moderate damage and concluded, "hundreds of thousands of existing houses similar to this case study are located in areas that can expect…similar or greater levels of damage." During the 1994 Northridge Earthquake, a 6.7 magnitude event, 20 lives were lost in wood platform framed buildings according to Rainer and Karacabeyli (2000). Though the loss of life has been limited in WFSFD, Holmes et. al (1996) also concluded, "Life Safety Performance as a minimum code requirement does not meet the expectations of those investing in housing (e.g., owners and lenders)." Therefore, improvements in performance (reduction in damage), in addition to improved life-safety, were primary motivations for conducting this study.

Extensive literature reviews were conducted by Carney (1975) and Peterson (1983) on wood diaphragm testing and design and by van de Lindt (2004) on shearwall testing, modeling and reliability analysis. These collections will not be repeated here. This review will include more recent research in those areas and extend those reviews to cover whole building testing, finite element analysis of these structures, research on post-frame roof diaphragms and earthquake damage analysis and estimation methods. It is intended to cover most readily-available papers published in major U.S. journals and presented at major conferences. This review will also discuss the state-of-the-art in these areas, the general progress of research to date and offer an opinion on where additional research is needed.

This paper is limited to conventional construction materials and methods that are used within the United States and Canada. Conventional materials include sawn dimensional lumber, structural sheathing and metal plate connected wood trusses, which are assembled with nails, screws, adhesive and proprietary sheet metal connectors. Typical systems in these areas use modular construction with typical floor-to-floor heights generally of 2.4 m (8 ft.), walls made of studs 38x89 mm (nominal 2x4 in.) or 38x140 mm (nominal 2x6 in.), floors of joists 38 mm (nominal 2 in.) width, all with a spacing of 400 mm (16 in.) or 600 mm (24 in.) and a roof composed of plate-connected engineered trusses or rafters 38 mm (nominal 2 in.) in width, spaced 600 mm (24 in.). There are methods of construction that are common in Africa, Asia, Australia, Europe and South America that are not frequently used in the United States and Canada. Without discussion of the relative merits of these systems, we have concentrated on these traditional methods used in the United States and Canada.

Unique systems have been and are being developed in the United States and Canada which involve special materials or component mechanics, such as visco-elastomeric damping materials in shearwalls, installing plywood in the center of the shearwall plane (mid-ply walls), seismic dampers and similar non-conventional systems. Some of these systems are proposed, but not in regular use at present. Most of these systems remain experimental and are excluded from this review.

The present state of seismic research is presented in the following sections. The research is summarized in tabular form for brevity. The focus of the presentation is recent research, and the objectives in writing this paper are to:

1. Review the current state-of-the-art with respect to WFSFD design and research for conventional materials and methods of construction;

2. Locate research areas where potential for additional research or improvement exists;

3. Compare, contrast and synthesize conclusions between different areas of research;

4. Provide conclusions and recommendations for future research in the seismic behavior of woodframe WFSFD.

PROGRESSION OF RESEARCH

Structural systems of various materials were developed using concepts of beams, columns, shearwalls and diaphragms. Though most materials use these concepts, not all systems are truly composed of individual components. Systems such as concrete or masonry often have continuity through joints and are cast on-site instead of being assembled from pieces shipped to the construction site. Conversely, steel, wood, precast concrete and cold-formed steel are assembled from individual components fabricated in a factory and assembled at the construction site.

Although this general progression has been followed in all materials, wood construction has unique characteristics that do not affect the other construction materials. Concerns about the applicability of scaling factors to wood, a natural product with a cellular makeup, have led to efforts to test full-scale models. Though wood members can be trimmed to a scaled size, the fundamental fiber or cell size does not change, so scaling of some wood test models may not be valid (van de Lindt 2007). Wood is anisotropic due to this cellular composition, and thus it has different strength and stiffness properties in each orthogonal direction (Breyer et al. 2007). Wood horizontal diaphragms have been traditionally considered “flexible” rather than “rigid” (Breyer et al. 2007). The IBC (International Code Council, 2011a) defines a flexible horizontal diaphragm as one which has more than twice as much mid-span deflection as the supporting story drift, and also includes an assumption that non-cantilevered horizontal diaphragms with no more than 38 mm (1.5 in.) of concrete are flexible. An ASCE 7 (2010) provision indicates that “.,,wood structural panels are permitted to be idealized as flexible if any of the following conditions exist…”. There is a growing list of peer-reviewed journal papers indicating that wood diaphragms often behave rigidly, rather than flexibly (Breyer et al. 2007). The authors are now seeing research that questions the assumption by which so many WFSFD have been designed (Christovasilis and Filiatrault, 2010, Philips et al. 1993, Skaggs and Martin 2004) . These distinctions lead to concerns that research into other materials and methods of construction or design may not fully apply to wood construction.

Quality control in the design and construction of WFSFD is sometimes difficult to achieve. Traditionally, WFSFD can be designed by unlicensed individuals unless the WFSFD is of significant size or complexity. If a structural engineer (SE) is involved, it is common for the SE to design only specific components (door or window headers, for example) and mark their design items on the architect's or designer's plans. This makes it hard for the construction team to know which component is engineered and which component is based on the prescriptive code. Field changes are occasionally made by the construction team without consultation with the designer. The relative ease with which wood members can be reshaped makes field changes more likely than with steel, for example. Basic elements (studs, plywood, OSB or oriented-strand-board) are assembled into walls, floors and roofs, and then stacked to assemble the structure. Thus, an SE may not be involved in the design of a specific WFSFD, and even if an SE is involved, the SE may not have designed the lateral force resisting system (LFRS) or the structure as a whole. Occasionally significant components are omitted, either by faulty design or construction. Falk and Itani (1988) and Graf and Seligson (2011) recommend increased quality control from design through final inspection, engineered design of all new WFSFD, engineering evaluation of older structures and certain mandated upgrades. A survey of architects and engineers in California indicated that significant omissions of key seismic resisting elements were missing in more than 40% of the buildings surveyed (Schierle 1996).

As late as the 1970s, WFSFD were considered to be very safe in earthquakes (Li and Ellingwood 2007, Skaggs and Martin 2004). Traditional WFSFD built before the 1970s were often regular in shape, usually one story in height, with a roof structure that was continuous throughout the structure. Though these simple dwellings suffered some types of damage during earthquakes, there was little loss of life. Changes in architectural style resulted in WFSFD of more recent construction, that have multiple stories, segmented roofs at different levels, and few long runs of shearwalls. These more recent dwellings suffered more damage, which became noticeable in the 1971 San Fernando earthquake (Moment Magnitude (MM) 6.6), the 1989 Loma Prieta earthquake (MM 6.9), and very notably, the 1994 Northridge earthquake (MM 6.7) (Graf and Seligson 2011). Minimum requirements for seismic connections between components began in the 1980s versions of the Uniform Building Code (UBC) (Breyer et al., 2007).

Research into WFSFD began with individual elements and progressed to horizontal and vertical diaphragms and finally to assembled structures. The general progression of research is shown in Fig.1. (Delineations between types of design or analysis are approximate.)

The damage and loss of life in wood-frame structures during the 1994 Northridge earthquake led to several major wood-frame research projects. These "Megaprojects" are discussed individually in the following sections:

CUREE-Caltech Woodframe Project

The Consortium of Universities for Research in Earthquake Engineering (CUREE) worked with the California Institute of Technology (Caltech) to study earthquake hazard mitigation in woodframe structures (CUREE 2002). This project was announced in 1998 as a $12.1 million, three-year study, funded by the Federal Emergency Management

Agency (FEMA) and the California Governor's Office of Emergency Services. This project became known as the CUREE-Caltech Wood-frame Project, and was charged with advancing the state-of-the-art in wood-frame analysis and design for seismic hazards. Under this project approximately 30 reports were produced, comprised of five elements: testing and analysis; field investigation; proposed revisions to the building codes; economic analysis and education and outreach. This was a coordinated program involving many universities, researchers and research efforts. Summaries of the results of those relating to this paper appear in Tables 2.1, 2.5 and 2.8, where noted. Discussion of specific portions of the project will appear in the following sections: Shearwall Testing and Analysis, Wood-Frame Dwelling Testing and Earthquake Damage Surveys.

The CUREE-Caltech project was among the first to dynamically test full-scale dwellings and to perform analysis of those results by different methods for comparison. Though it produced answers to many of the questions of the day, it also provided direction for further research in areas that it could not answer within the project timeframe (Cobeen 2004a&b). It further developed and standardized dynamic testing methods intended to better evaluate wood structure performance.

NEESWood Project

The NEESWood project (Network for Earthquake Engineering Simulation - Wood) began in 2005 and was funded by the National Science Foundation (NSF) with a $1.2 million grant, as a multi-year project to study how wood-frame structures respond to seismic forces. NEESWood continued the work begun by the CUREE-Caltech project by performing and analyzing a series of experiments based on the CUREE prototype buildings (van de Lindt 2006a&b). (These prototype buildings were designed to provide a basis for experimentation and analysis that all the CUREE-Caltech researchers could use.) NEESWood experiments included shake table testing of two-story townhouses and culminated with shake table testing of a 6-story wood building for a 7.5 magnitude earthquake. Papers based on these tests appear in Table 2.5 and are discussed under Wood-Frame Dwelling Testing below. The performance of the 6-story building has indicated that large residential buildings can be successfully designed to withstand expected seismic activity in any region. Additional analyses of the test results are being performed and further papers will be appearing, so the results of this project are not yet complete. Though much larger than typical houses, the experiments with this structure help to evaluate some of the wood structural systems and elements that are also used in houses.

SHEARWALL TESTING AND ANALYSIS

Conventional shearwalls are constructed of 38x89 mm (nominal 2x4 in.) or 38x140 mm (nominal 2x6 in.) studs with a structural sheathing consisting of plywood, oriented-strand board (OSB), Portland cement plaster or other approved material. Research efforts on the static and dynamic properties of conventional structural panel shearwalls are numerous. Table 2.1 provides a chronological list with a focus on the most recent ten years of research (for brevity). Table 2.1 also includes some notable previous studies that investigate brittle finishes or non-wood materials. The most common non-wood materials are gypsum wallboard panels or Portland cement plaster on lath or expanded metal mesh. The reader is also referred to an excellent review of shearwall research by van de Lindt (2004).

Early studies tended to be static loading tests based on ASTM E-72 (American Society for Testing and Materials 2005) and more recently ASTM E-564 (American Society for Testing and Materials 2006). The studies that followed 1990 often used cyclic loading protocols proposed by the Structural Engineers Association of Southern California (SEAOSC) and the CUREE protocols developed for the CUREE-Caltech Woodframe Project. Table 2.2 describes the characteristics of seismic loading protocols known as ASTM E-546 (American Society for Testing and Materials 2006), ISO-16670 (ISO 1998), sequential phased displacement (SPD) (SEA 1996, Dinehart and Shenton Ill 1998), FCC-Forintek (Karacabeyli and Ceccotti 1991) and CUREE-CalTech/CUREE-CalTech Near Fault (Krawinkler et al. 2001). Figures 2.2a through 2.2f referenced in Table 2.2 show the shape of each of these loading protocols.

Cyclic testing has shown that concerns about the nonlinear performance of brittle or non-ductile materials (mainly gypsum and cement plaster) are warranted (Falk and Itani 1987, Hart et al. 2008, Seaders 2004&2009a). Various studies indicate permissible elastic drift limits for brittle materials, but limited research has been conducted to determine whether these materials can be kept within elastic limits for WFSFD design, or whether more attention to fastener or connection ductility may lead to improved methods of construction with brittle materials or in walls with openings (Merrick 1999, Uang and Gatto 2003, Rosowsky and Kim 2004a&b). Further, use of ductile framing or elastic adhesives to support brittle materials may allow more effective use of their strength without pushing these materials into the non-linear range.

HORIZONTAL WOOD DIAPHRAGM RESEARCH

Research on residential diaphragms can be divided into two groups; floor (flat) diaphragms and roof diaphragms that usually have a slope or pitch (which may also be flat). Horizontal diaphragm studies are listed chronologically with the specific research focus in Table 2.3. Design practice has not typically differentiated between flat and pitched diaphragms (Breyer et al. 2007). Many excellent studies have been published on horizontal floor or roof diaphragms, however, there is very limited research on pitched roof diaphragms (Johnson and Burrows 1956, Tissel and Rose 1993) or roof diaphragms that include gypsum board ceilings (Walker and Gonano 1984, Alsmarker 1991). Significant numbers of these studies involve analytical models, rather than laboratory experiments.

Wood horizontal diaphragms have been traditionally considered “flexible” rather than “rigid” (ASCE 2010) . Studies by Phillips (1990); Phillips et al. (1993) and Tarabia & Itani (1997) indicate that the assumption of a flexible diaphragm may be unconservative.

Thirty papers were found investigating different aspects of gypsum shearwall design, but only three considering the effect or contribution of the gypsum ceiling in the horizontal diaphragm design. The IBC (2011a) Table 2306.3(3) contains 17 lines of gypsum board, gypsum lath and plaster or Portland cement plaster shearwall design values, using only staples, but no diaphragm design values for gypsum products used on a ceiling. Though shear capacities of walls having both plywood and gypsum cannot be summed, the IBC permits the designer to use twice the lesser shear capacity (usually the gypsum), which treats the shearwall as if gypsum existed on both sides.

The restriction on combining gypsum and wood panel sheathing strengths is based on the understanding that the stiffer material will provide most of the lateral resistance. This is true with respect to wood shearwalls. However, with roofs that are framed with dimension lumber or metal-plate-connected trusses (MPCT), the top and bottom truss chords (or joists and rafters) are distinct and separate components, so it is unclear if some additional capacity or efficiency may be obtained through a combination of the ceiling and roof sheathing. Table 2.3 shows two papers on pitched roof diaphragms (Johnson and Burrows 1956; Tissell and Rose 1993) and only one paper on MPCT (Tissell and Rose 1993). Also, the greatest pitch tested was 4:12, which is less than in many current roofs. The majority of the wood roof diaphragm research involves roof pitches of 3:12 or less, often with plywood overlaid on T&G roof decking several inches thick and trusses of 2x, 3x or 4x members bolted at joints. The roof diaphragm stiffness or strength for differing pitches of light 2x MPC truss configurations typically used today (6:12 or higher pitches) remain untested or unreported. There may be significant opportunities to improve the performance of MPCT with gypsum ceiling and structural panel diaphragms.

FINITE ELEMENT MODELING

Static testing of wood dwellings occurred primarily before 1990, with tests performed by Yokel et al. (1973), Yancy and Somes (1973), Tuomi and McCutcheon (1978) and Boughton and Reardon (1982). Dynamic testing of full-scale models of residential buildings was rare until the early 1990's, due to several problems. The expense of constructing a complete dwelling is great. There is also a limited number of shake tables capable of testing a full-sized model. As an alternative method of analysis, researchers have used finite element (FE) models to test their analytical understanding of material and connection behavior, with model complexity ranging from simple static, linear-elastic models to complex 3-D nonlinear models analyzed with seismic excitation data. These analyses are listed chronologically including the specific focus in Table 2.4.

Yancey and Somes (1973) indicate that research is needed on torsional behavior, post-ultimate load behavior and simplified, practical analytical models. They stated that "the available studies are either too complicated and time consuming or too simplified that their accuracy is questionable."

Recent research has included structural reliability or fragility analysis combined with probabilistic seismic hazard models to determine damage risk (Li et al. 2010, Kim and

Rosowsky 2005a&b, Li 2005, Li and Ellingwood 2007). These methods require thousands of model evaluations to produce reasonable results. Typical studies use a few tested structures or components and then perform the required analysis calibrated to the physical tests.

Much effort has been spent developing various models without reaching a consensus on the methods and elements to be used. For example, there is a general consensus in the mechanics of modeling concrete and steel with various connections and fixity. Commonly used software does not require that the designer implement an element from scratch. With WFSFD, there is no consensus on the methods used to model connections, shearwalls or diaphragms. Many studies have been performed, independently of the others, and there are dozens of finite element model approaches for wood structures. However, it is difficult to compare the accuracy of models of structures using different elements and techniques. Research comparing these elements may assist in determining which would be most useful to the practitioner.

WOOD FRAME DWELLING TESTING

A limited number of full-scale experiments has been performed on WFSFD, as summarized chronologically in Table 2.5. In static tests, loads or displacements are applied to a dwelling at specific locations to test deflections of diaphragms or shearwalls (Yokel et al. 1973, Yancy and Somes 1973, Tuomi and McCutcheon 1978 and Boughton and Reardon 1982). Dynamic tests can be performed using computer-controlled hydraulic rams attached to the structure or by securing the structure on shake tables capable of generating earthquake level accelerations. The first full-scale shake table test was reported by Fischer et al. (2000), with most of the subsequent studies being performed on ever larger shake tables.

A small residential structure was designed and tested on a shake table at the University of California at San Diego (UCSD) as a portion of the CUREE-Woodframe Project (Fischer et al., 2000). The structure was heavily instrumented and many configurations for wall construction were examined. One objective for the project was to obtain as much data as possible on component and system deformations for potential study by other researchers.

Recent tests from the NEESWood project by Christovasilis et al. (2006); van de Lindt et al. (2006a,b); Pang et al. (2007) and Filiatrault et al. (2007), that continues some of the work of the CUREE-Woodframe project, investigated the performance of a complex townhouse on two coupled shake tables.

The most recent project focuses on prediction, testing and evaluation of a 6-story wood-frame building tested on a shake table in Japan. A key component of the investigation was to verify the applicability of performance-based design (PBD) for wood structures. See van de Lindt et al. (2010, 2011, 2012); Pang et al. (2010), Pei and van de Lindt (2011).

Pavaere et al. (2003) performed experiments on a full-scale L shaped house using static and dynamic loading applied to the structure with a hydraulic ram. Displacements were measured at key locations and uplift forces were measured at locations where anchor bolts would typically be installed into a foundation.

In some cases, a specific structure was tested, but it was either a small, research sized building or an individual, unique dwelling. Researchers used the testing protocol that they believed most important or most practical at the time. These tests cannot be easily compared due to these issues. From these limited tests, there has not been enough consistent, comparable data to permit evaluation of the significance of building geometric factors on the behavior of the structure. Recent projects have both performed experimental research and performed analysis, so have resulted in models and techniques that are correlated. In historic cases where researchers performed well-documented testing, it was often difficult for subsequent researchers, not involved in the specific experiment series, to produce accurate models without making many assumptions. Recently, we have seen projects such as the NEESWood Project, which involve many researchers simultaneously working on different aspects of the research. This helped improve communication among the researchers and is a favorable trend that should result in more useful results.

Early research on shearwalls, diaphragms and other components (straps, tie-downs, etc.) was primarily interested in determining yielding behavior, rather than system deformation or deformation-based damage. Whole structure testing on shake tables is beginning to yield useful information on deformation and system effects. Full-scale testing has not, however, resulted in substantially more practical information on the design of individual WFSFD components, such as shearwalls or diaphragms. Testing of WFSFD has been of limited use thus far in design in part because there are many variations in geometry and materials that have prevented development of accurate, general purpose design methods.

POST-FRAME BUILDINGS

Unique to post-frame design is the roof diaphragm shear reduction factor. No such reduction is used in WFSFD, thus a discussion of post-frame research is merited and a brief discussion is included here. Post-frame buildings use the moment connection capacities of timber connections and the flexural capacities of columns with a fixed base to provide the lateral force resisting system for these structures Gebremedhin et al. (1986). These buildings can be heavy timber resort lodges or SFD, but may also include many agricultural buildings. Typical construction of an agricultural post-frame building consists of corrugated metal siding and roofing over a timber framework.

Much of the agricultural post-frame design research consists of analytical studies rather than experimental programs (see chronological list with conclusions in Table 2.7). Generally, post-frame buildings have pitched roofs rather than flat, horizontal diaphragms. In Gebremedhin et al. (1986), an equation is used to calculate a reduction in the stiffness of horizontal diaphragms for pitches other than strictly horizontal. This is unique to post-frame construction and is not a part of typical design practice in WFSFD. Post-frame testing programs specifically examining roof diaphragm stiffness are summarized with their principal conclusions, chronologically in Table 2.6. Experiments generally used heavy trusses (due to the size of the structures) with corrugated sheet metal roofs, so it’s not clear that these experiments are relevant to WFSFD.

EARTHQUAKE DAMAGE SURVEYS

Surveys of damage from major earthquakes in the United States include those shown in Table 2.8. Table 2.8 shows the survey reports or papers chronologically with seismic event and conclusions. Typical surveys review either substantial amounts of data at a limited level or a few specific cases in depth. Many studies (10 out of 13 in the table) are based on structures damaged in California. State laws that protect both the owners’ privacy and the copyrights of the architect and engineer also limit California building surveys. Signed releases must be obtained from all these parties to gain access to the plans. So, in many cases, these are not available. Further, when an organization is charged with conducting the study, the work is generally targeted towards the final report and its conclusions, rather than concentrating on extensive details that would be useful to subsequent researchers. For example, plans were rare in the early studies, but more frequent in later studies. Elevations showing the sizes of openings in interior and exterior walls are non-existent.

The 1971 San Fernando earthquake study contains valuable information on the state of seismic design as well as the results of field surveys of damaged buildings by Morgan and Bockemohle (1973); Pinkham (1973) and Steinbrugge and Schader (1973). These studies provide the examination and opinions of the researchers, but lack plans and details sufficient for further analysis. Many of their recommendations have been implemented in the appropriate building codes.

The 1989 Loma Prieta earthquake was the largest event in California since the 1906 San Francisco earthquake. In a survey of damage, EQE Engineering (1989) noted that: " . . . [wood-frame] buildings have generally performed well in past earthquakes. . . ," except, "older (especially pre-1940s) homes, because these lack positive connections to their foundations or have raised floors supported by relatively weak cripple walls," and "some of the more irregularly shaped newer homes that lack clear load paths due to complex geometry or are built without enough wall area to resist the seismic forces." Additional serious problems included multi-story apartment buildings with garages on the first floor. The survey data did not include plans or details for the WFSFD examined.

The Applied Technology Council (ATC) (Poland and Scawthorn, 2000) produced a study of 500 buildings located within 0.3 km (1000 ft.) of the 1994 Northridge Earthquake fault. Though this study is quite detailed, plans were excluded in the distributed electronic database. Therefore, the database is not useful for analyzing the design of those structures. It can only be used on a gross scale to compare building damage by type or location, for example.

The Earthquake Engineering Research Institute (EERI) published a report detailing damage to different types of structures and facilities by Holmes and Somers (1996). It details some of the types of damage seen in various WFSFD during the 1994 Northridge Earthquake. Holmes et al. (1996) include evaluation of two WFSFD damaged in the Northridge earthquake. The first report describes damage to a two- story WFSFD constructed in 1958 and located within 0.8 km (1/2 mile) of a strong motion seismograph. Damage to this structure was non-structural and the WFSFD was considered suitable for immediate occupancy. Notably, the cost of repairing the damage to the structure was actually so great that it was considered a total loss. The second WFSFD was a single-story home built in 1911 and seismically retrofitted three months before the earthquake. This WFSFD experienced minimal damage in comparison to WFSFD of similar construction in the immediate neighborhood. The report included plans and details, but no elevations or schedules that would show window and door opening sizes, therefore further analysis would depend on significant assumptions about the construction.

In a general survey, Crandell and Kochkin (2003) reviewed the history of wood-frame WFSFD construction and related current design concerns to engineering practice. Engineering design uses the seismic provisions of the IBC (International Code Council, 2011a). The IRC (International Code Council, 2011b) is a prescriptive code, and is based on traditional methods of construction. Though related to engineering and construction practice, this code is not necessarily easily linked to engineering principles and calculations. The authors identified the following differences between engineering design practice and conventional prescriptive construction methods:

1. Lateral Force Resistance (Shearwalls and Diaphragms), including perforated shearwalls and rigid diaphragm behavior;

2. Connection Design, discussing cross-grain tension and toe-nailing;

3. System Effects, where loads are redistributed in the system, increasing its redundancy;

4. Safety Margins and Performance Objectives, addressing the absence of a commonly understood level of performance for WFSFDs;

5. Design Loads: differences between engineering loads and prescriptive design standards.

Subsequent work on the IRC (International Code Council, 2011b) has attempted to address these issues (Crandell 2007, Crandell and Martin 2009).

Schierle (2003) provides engineering surveys of damaged residential and commercial buildings affected by the Northridge earthquake. Floor plans and elevations including categorization of the damage were included, along with the engineer's written evaluation. Since there were no elevations included, some assumptions need to be made regarding exact heights of windows and doors if this study is used for further analysis.

Damage surveys have shown the types of damage that have been problematic in WFSFD. However, without significantly more detail in the surveys, it will be difficult to use these structures for further analysis. It's important to include more information in the future because these structures are of typical construction and have gone through major natural events, characteristics not necessarily true of WFSFD constructed for laboratory research. Further refinement in methods and data collection will await the next major U.S. event.

Authors of some damage surveys suggest that correctly following building codes and engineered plans would mitigate or reduce seismic damage. It is certainly true that omitting one or two fasteners on each diaphragm will reduce its capacity. Further, the building codes that were once booklets that contractors could easily carry have become large tomes that are difficult to interpret. Within the damage surveys shown in Table 2.8, most indicate a number of design problems with the structures (example: cripple wall bracing: Falk and Soltis 1988; EQE Engineering 1989; NAHB 1994) but also one or two quality control items on each structure (example: no anchor bolts: Falk and Soltis 1988). Part of the problem is that without the original plans for the WFSFD, it's difficult to know if a hold-down or anchor bolt is missing because it's not on the plans, or because it was omitted during construction. So, differentiating between design errors and construction defects has been difficult for these surveys. Nevertheless, the quality of WFSFD construction is generally not as good as commercial construction and more quality control would help.

Damage surveys also frequently conclude that seismic strengthening efforts are effective. Much attention is usually paid to ensuring that new construction adheres to the current code, whereas upgrading older construction is considered “elective.” (Holmes et al. 1996).

DAMAGE ESTIMATION METHODS

There are a number of different damage estimation methods and strategies that have been developed by different researchers and organizations. Table 2.9 summarizes these methods and their specific purposes. These strategies are largely based on the accepted traditional basis for design, life-safety. Buildings constructed to the code requirements in the United States are intended, “…to minimize the hazard to life and improve occupancy capability of essential facitities after a design level event or occurance.” (International Code Council, 2011a) Under these strategies, a building will most likely suffer significant damage to the structural system and need to be significantly repaired or replaced due to the economics of repair.

In recent seismic events, some wood-framed WFSFD which were judged habitable were nevertheless considered total losses by the insurance companies (Holmes et al 1996). The damage was non-structural, limited to cracking of walls and finishes. The cost of repairing building finishes was too great relative to the value of the WFSFD. None of the existing damage estimation methods can accurately predict the level or cost of damage because the methods are directed towards evaluating and obtaining the life-safety standard.

Lucksiri et al. (2012) adapt the basic philosophy of rapid visual screening to the unique characteristics of WFSFD, emphasizing plan geometry, and validate the method by a comparison of 480 representative models.

Generally, damage estimation methods seem to be well developed at present. These methods were mainly developed after the Northridge earthquake. Additional opportunities for research in this area will require further comparison to concurrent experiments or await the next major event in the U.S. Since this is most likely to be in California, amendments to state laws allowing access to building plans by researchers would be very useful. Additionally, involving an analysis component by researchers whose primary focus is in wood construction would help to expose information gaps and omissions. Similar to the experiences of dynamic structural testing, it would be useful to perform detailed or FEA analysis simultaneously with damage investigation, so that useful comparisons with existing design methods can be obtained.

RESEARCH CHALLENGES AND FUTURE DIRECTIONS

Knowledge needs to be created to ensure that WFSFD can be designed and built to resist seismic loads to the level expected by building owners, civil authorities and society expectations. By producing research that improves accurate modeling of different WFSFD configurations, designers will understand what components are truly required and what level of performance can be expected. For example, designers currently design roof structures based on data developed in the 1950s, when the cost of lumber was relatively low due to sale of inexpensive Federal timber in the National Forests and old growth lumber was readily available. Presently, the decision on whether these configurations are cost effective remains with the architect, engineer and the owner, not with the researcher. Consequently, cost of construction is rarely a reported factor in WFSFD research.

Improvements in the following areas are crucial to improving seismic performance of WFSFD. Research has not addressed many areas in seismic behavior of WFSFD.

Innovative Methods. Conventional construction methods were developed to be cost-effective and easily installed. For example, the use of short or 'pony' walls to span vertically from a short concrete foundation to the first level of a house built on sloping terrain. But these methods have been difficult to analyze and research has shown some of them to be ineffective. Thus, there is a significant need to develop new and innovative construction materials, connections, fasteners and techniques to overcome the limitations of wood, such as increasing system ductility.

Brittle Finishes. Present research has concluded that brittle materials are of limited value in providing seismic resistance. There has been limited research to improve seismic performance of brittle materials, such as gypsum wallboard, including the effects of openings, nor to improve ductility in the construction of shearwalls designed with brittle materials. Brittle finishes may have stiffness and strength that can be exploited if ductile methods of connection can be developed. Use of elastomeric sheets, resilient channels or ductile fasteners could be routes to achieve this. Examination of using all of the gypsum walls in an SFD may result in elastic (non-damaging) performance. There is also substantial opportunity to study the behavior of MPCT (metal-plate-connected trusses) in WFSFD lateral force resisting systems (LFRS), as wells as combinations of gypsum board ceilings with structural wood sheathing on the MPCT and on flat roofs.

Horizontal/Pitched Diaphragms. Abundant data exist on rectangular horizontal or low pitch gable roofs particularly with a heavy timber supporting framework. Different configurations (L, T and U shapes, for example) need to be tested, as do roofs of differing pitches and hip roofs. It needs to be determined whether a shear stiffness or strength reduction factor similar to post-frame design is applicable to WFSFD. OSB and structural insulated panels (SIP) roofs should also be tested to verify whether existing data are applicable to their design. Assumptions of flexible diaphragm behavior continue to persist in the building codes in spite of research indicating that the assumption is not valid for all structures; therefore additional research is needed to show that the assessment of diaphragm flexibility needs to be made in each case by the designer. If pitch results in a stiffness reduction factor, some rigid roofs could be flexible or semi-rigid or rigid at different pitches. Research should address how a stiffness reduction factor, if any, affects design of SFD horizontal diaphragms.

Finite Element Methods. Researchers have contributed much effort in finite element modeling of wood structures, but have not yet developed consensus methods and elements that should be used. PBD may result in better designs for buildings than the present code based methods. However, to date, different researchers have used different methods of analysis and design. As a result, it is difficult to compare the accuracy of models of structures using PBD, different FE elements and techniques. Research comparing these methods may assist in determining which would be most useful to the practitioner. For many practitioners, PBD methods will need to be codified to result in widespread use. But at present, few of these methods have been adopted or provided in the commercially available finite element software, limiting use by design practitioners. Synthesizing the existing research and disseminating this research to the designers is the greatest challenge here.

Whole Structure Testing. Historic tests of WFSFD have been of limited use because it has been so difficult to completely quantify the structure so as to allow an independent researcher to refine their analysis methods. To date, there have not been enough consistent, comparable data to permit evaluation of the significance of building geometric factors on the behavior of the structure. Whole house testing had primarily measured damage to the WFSFD components, rather than determining whether a limiting behavior has been reached by the WFSFD as a whole. Therefore, such testing is not easily correlated with the testing of individual components. Research on shearwalls, diaphragms and other components is usually based on yielding performance as a method of determining whether life-safety goals are being met. Such tests generally do not measure the amount or type of damage at various loading intervals. Substantial recent progress in testing large structures has been made. Understanding and integrating the measured results into present analysis methods remains the major challenge.

Damage Estimation Methods: Damage estimation methods seem to be well developed at present, and are mainly products after the Northridge earthquake. Additional opportunities for research in this area will require further comparison to concurrent experiments (such as application to a shake table structure before testing) or await the next significant earthquake in the U.S.

Damage Surveys. More complete reports of damaged WFSFD are needed. Open access to California plans and documents on WFSFD for research would assist this effort greatly. (California is not the only state affected by earthquakes, but earthquakes are common and the laws restricting release of the original plans affect researcher's access to data that might improve design.) There is a challenge to define the required document sufficiently to permit detailed analysis while protecting the designer from the risk of losing their intellectual property within the plans. It's important to include more information in the future because these structures are of typical construction and have gone through major natural events, characteristics not necessarily true of WFSFD constructed for laboratory research. Further refinement in methods of documenting the existing structure and communicating that data to future researchers is needed so that the present or future PBD models can be applied to real structures with real damage. It would be helpful to test some structures or portions of structures using the different testing protocols developed to date, to determine which protocol(s) best simulate(s) actual seismic stresses, deflection and damage. Application of FEA to sample damaged structures before demolition, would allow more accurate modeling to be performed.

<u>Collaboration.</u> Research continues along paths that seem most likely to improve design and evaluation of WFSFD. The following trends seem very positive: full-scale shake table tests of large structures; comparison of tested structure performance with results from finite element design programs, both for strength and prediction of deformation of components; and multi-researcher projects where test results have been analyzed, and finite element models produced by researchers either from the same institution or operating under the same grant, thus ensuring access to sufficient structural detail to permit accurate modeling. There is a strong need to develop a better understanding of the effects of WDSFD components, attachment, LFRS and how loads are distributed to these

elements within the structure. A consensus needs to be developed on performance objectives for WDSFD with respect to damage and repair costs, including new design techniques which balance life-safety with the effects of damage to building finish materials. Finite element analyses of seismically damaged WFSFD would lead to a better understanding of component performance and allow evaluation of seismic testing protocols.

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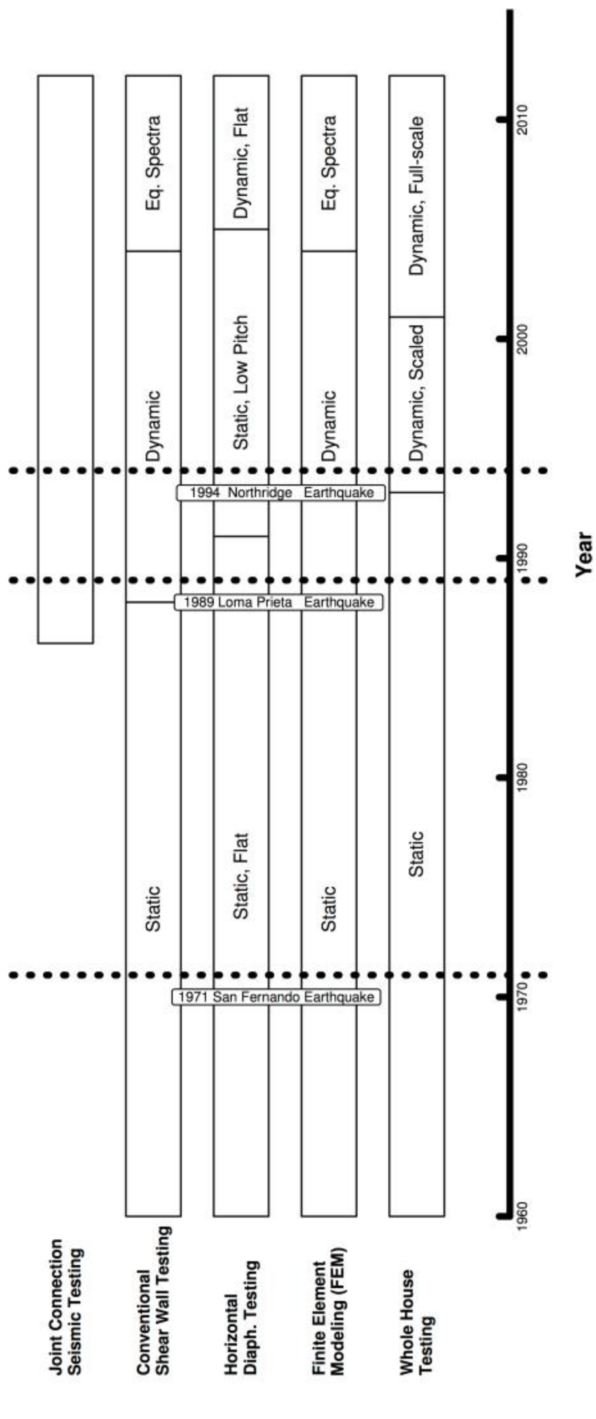
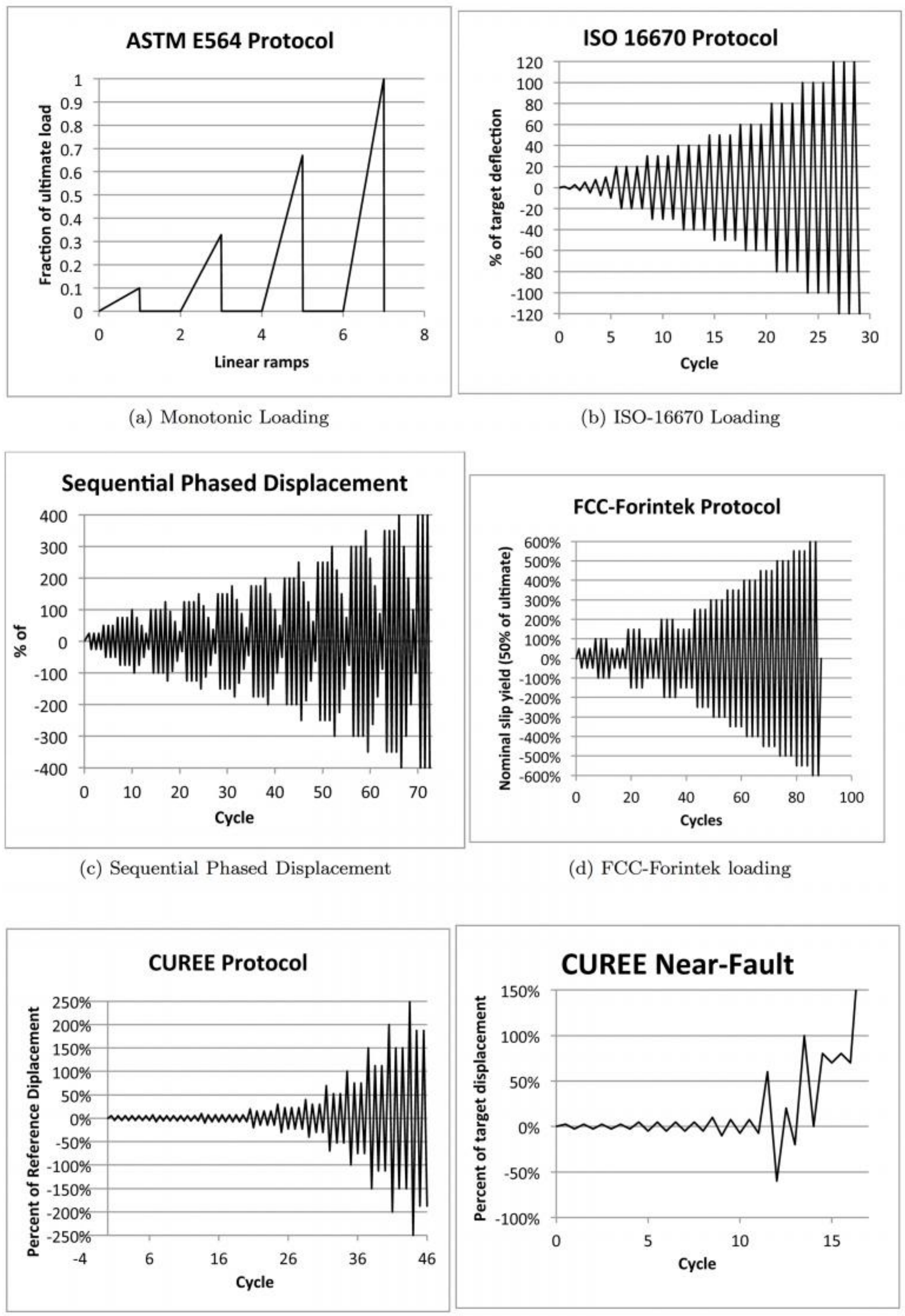


Figure 2.1. Progression of Wood-frame Dwelling Research and Methods.



(a) Monotonic Loading

(b) ISO-16670 Loading

(c) Sequential Phased Displacement

(d) FCC-Forintek loading

(e) CUREE loading

(f) CUREE near-fault loading

Figure 2.2. Common Shearwall Testing Protocols.

Table 2.1. Conventional Wood Shearwall (SW) Testing and Analysis

Reference	Method/Loading	Focus of Research
Oliva & Wolfe (1988); Oliva (1990)	ASTM E564. monotonic, static cycles @ 1 Hz, dynamic @ 5 Hz.	Tested 59 gypsum SW for racking resistance. 2.4 m (8 ft.) long walls confirmed codes, but longer walls, and horizontal sheets were better. Gluing increased stiffness and strength but resulted in more abrupt transition to plastic behavior.
Thurston & King (1994)	Racking resistance	Ten SW, varying wall returns, openings & materials w/o hold-downs.
Seible et al. (1999)	Analytical study	CUREE workshop on testing, analysis & design.
Karacabeyli et al. (1999b)	Static & dynamic	Compares static & dynamic SW test results.
Merrick (1999)	cyclic, non-increasing	7 tests of plywood, OSB, gypsum wallboard SW to evaluate energy dissipation.
Salenikovich & Dolan (2000, 2003a,b)	Monotonic, cyclic @.25 Hz (ISO 1998)	Investigates the strength of anchored SW, 2.4 m (8 ft.) tall, 4:1, 2:1, 1:1, 2:3 aspect ratios.
NAHB (2001)	Static	Strength and deflection of SW w/ corners and openings.
Gatto & Uang (2002, 2003); Uang & Gatto (2003)	Dynamic & cyclic	Standard construction 2.4x2.4 m (8x8 ft.) woodframe shearwalls were tested using: monotonic, CUREE-Caltech standard (CUREE), CUREE-Caltech near-fault, sequential phased displacement (SPD), & International Standards Organization test protocols
McMullin & Merrick (2002)	Cyclic	6 shearwalls of grade CD plywood, OSB & gypsum wallboard, includes tests of different types of drywall screws.
Kim (2003); Rosowsky & Kim (2004a,b); Kim & Rosowsky (2005a,b)	Reliability analysis	Develops fragility curves for various SW materials.
Langlois et al. (2004)	Static, cyclic	Applied monotonic (ASTM E564) & cyclic (CUREE) testing protocols to SW.
Ni & Karacabeyli (2004)	Analytical study	Presents equations for evaluating deflection of unblocked SW & horizontal diaphragms.
van de Lindt et al. (2004); van de Lindt & Rosowsky (2004)	Reliability analysis	Tested 12 SW designed & evaluated for reliability w/ASCE 16.
Seaders et al. (2004); Seaders et al. (2004); Seaders et al. (2009a); Seaders et al. (2009b)	Monotonic (ASTM E564), cyclic & earthquake loads	Two sets of tests of 8 partially & 2 fully anchored 2.4x2.4 m (8x8 ft.) shearwalls w/ 38x89 mm (nominal 2x4 in.) Douglas-fir studs at 610 mm (24 in) o.c. 2 OSB w/8d nails & GWB.
van de Lindt (2004)	Literature review	Details 31 SW tests, modeling & reliability analysis.
Williamson & Yeh (2004)	SPD (SEAOSC, FME=3 cm)	SW w/openings (“portal frames”).
Dean & Shenton III (2005)	ASTM E564 modified to exceed design allowable before the final half-cycle	Ten 2.4x2.4 m (8x8 ft.) SW w/11 mm (7/16 in.) OSB & applied vertical load.

Table 2.1 cont'd. Conventional Wood Shearwall (SW) Testing and Analysis

Reference	Method/Loading	Focus of Research
Lebeda et al. (2005)	Static, cyclic	13 2.4x2.4 m (8x8 ft.) SW w/ misplaced hold-downs. (CUREE)
White (2005); White et al. (2009, 2010)	Earthquake records.	Tested 34 identical 2.4x2.4 m (8x8 ft.) walls of 38x89 mm (nominal 2x4 in.) kiln-dried DF. Studs were spaced at 610 mm (24 in.) o.c. Half partially anchored, half fully anchored.
Johnston et al. (2006)	Cyclic	Compares effects of vertical load & hold-down placement.
Seaders et al. (2004); White (2005); van de Lindt & Gupta (2006); White et al. (2009)	3 SAC response spectra	2.4x2.4 m (8x8 ft.) SW w/11.1 mm (7/16 in.) OSB & 12.5 mm (1/2 in.) gypsum panels.
Leichti et al. (2006)	CURRE	Tested SW with different nail strengths.
Mi et al. (2004, 2006)	Monotonic and ASTM E2126	Eight 4.9x4.9m (16x16 ft.) SW w/12.5mm (1/2 in.) plywood.
Winkel (2006); Winkel & Smith (2010)	Static	14 tests of shearwalls with combined racking, uplift and bending loads.
Yasumura et al. (2006)	1940 El Centro	Two-story 3x3x6m (9x9x18 ft.) 7.5mm (5/16 in.) plywood w/openings.
McMullin & Merrick (2007)	Monotonic & CUREE-CalTech	11 tests. Discusses seismic damage thresholds for gypsum wallboard.
Ni & Karacabeyli (2007)	ISO 16670, ASTM 2126	16 SW w/ diagonal or transverse horizontal lumber sheathing and gypsum sheathing varying hold-downs, vertical load, & width of sheathing.
van de Lindt (2008)	shake table tests	24 shake table tests of SW, some w/gypsum, some w/ corner walls.
Hart et al. (2008)	Cyclic, varying by author	Discusses 195 drywall & stucco sheathing tests done by APA, Merrick, City of Los Angeles and McMullen & Pardoen for CUREE.
McMullin & Merrick (2008)	Cyclic CUREE-Caltech	17 tests w/ screws & nails w & w/o window openings.
Sinha (2007); Sinha & Gupta (2009)	Monotonically (ASTM E564)	Tested 16 standard 2.4x2.4 m (8x8 ft.) walls, 11 were sheathed with OSB on one side & GWB on the other, & 5 walls were tested without GWB. Digital image correlation was used for data acquisition & analysis which is a full-field, noncontact technique for measurement of displacements and strains.
Zisi (2009)	Monotonic & cyclic w/increasing amplitude.	Tested brick veneer on wood-framed walls w/ OSB and gypsum.
Ni et al (2010)	Monotonic & cyclic (ISO 16670)	Tested 20 configurations of 1.22, 2.44 or 4.88 m long SW with 9.5 mm OSB or 12.7 mm GWB, some 4.88 m SW with a 2,44 opening, some 2.44 m walls with 1.22 or 0.61 m perpendicular bracing walls.
Goodall & Gupta (2011); Goodall (2010)	Monotonically (ASTM E564)	Tested 14 shearwalls, 2 of each of 7 different designs. Six walls had 1105x610 mm window openings, eight did not. All walls were 2.4x2.4 m (8x8 ft.) & built from 38x89 mm (nominal 2x4 in.) DF studs at 610 mm o.c. Tests stopped at deflections of 4.0, 8.0, 12.0, 16.0, 20.0, 24.4, 48.8 & 73.2 mm (5/32, 5/16. 5/8, 3/4, 1, 2 in.) to record damage.

Table 2.1 cont'd. Conventional Wood Shearwall (SW) Testing and Analysis

NOTE:

DF – Douglas-Fir

FEM – Finite Element Model

FME – First Major Event, defined as an event sufficient to bring the structure to the yield point

GWB – Gypsum Wall board

OSB – Oriented Strand Bard

SAC – A joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering

SEAOSC – Structural Engineers Association of Southern California

SPD – Sequential Phased Displacement

SW – Shearwall

Table 2.2. Definition of Shearwall Testing Methodologies.

Protocol Type	Standard or Common Name	Classification	Description	Figure	Reference
Static	Monotonic (ASTM E564)	Linear increasing	Ramps load to incremental limits	Fig. 2.2a	American Society for & testing materials (2006)
Cyclic	ISO-16670 2003	Full cycle reversing	Initial increasing sequence, then 3 cycles at each increasing displacement level	Fig. 2.2b	ISO (1998)
	Sequential phased displacement (SPD)		Sequential phased displacement (SPD)	Fig. 2.2c	SEA (1996) Dinehart & Shenton (1998)
	FCC-Forintek		Similar to SPD	Fig. 2.2d	Karacabeyli & Ceccotti (1996)
	CUREE		CUREE-CalTech	Fig. 2.2e	Krawinkler et al. (2001)
	NF		CUREE-CalTech Near Fault	Fig. 2.2f	Krawinkler et al. (2001)
Dynamic	Usually a set of scaled historic seismic records				

NOTE:

SPD – Sequential Phased Displacement

NF – Near field

Table 2.3. Horizontal Wood Diaphragm Testing and Analysis.

Reference	Methods	Focus of Research
Countryman (1952)	Static, Dynamic	Plywood diaphragms, mostly blocked. Tested 6 quarter-scale models, 1.5x3.0 m (5x10 ft.) & 4 full-scale models 3.7 m or 6.1x12.2 m (12 or 20x40 ft.). Dynamic load was static load increased by 1/3 & cycled 5 times.
Countryman & Colbenson (1954)	Static, Dynamic	Plywood diaphragms, about half blocked. Tested 15 full-scale models 7.3x7.3 m (24x24 ft.). Dynamic load was static load increased by 1/3 & cycled 6 times, then single cycles of increasing amplitude to failure.
Johnson (1955)	Static	Tested 3.7x18.3m (12x60 ft.) plywood diaphragm.
Johnson & Burrows (1956)	Static	Gable roofs were tested & found stronger than flat diaphragms w/ no boundary reinforcing & weaker than flat diaphragms w/ boundary reinforcing.
Tissell (1967)	Static	Tested 18 diaphragms 4.9x14.63 m (16x48 ft.).
Carney (1971)	Analytical study	Presents development of the general theory of folded plates as it applies to plywood roof diaphragms.
Johnson (1971)	-	Tested 6.1x18.3 m (20x60 ft.) roof section sheathed w/ plywood overlaid on decking.
Falk et al. (1984)	SOTA	Reviews literature on low-rise wood diaphragms. Concludes more research is needed on roofs and dynamic behavior.
Walker & Gonano (1984)	-	Tested gypsum and asbestos cement ceiling panels. Determined independent panels should be modeled for shear, solidly connected panels for flexure.
Falk & Itani (1988)	Analytical study	Compares deflection model to previous flat diaphragm tests.
Kamiya (1988)	Pseudo-dynamic tests	Simple hysteretic loop model.
Mahaney & Kehoe (1988)	Analytical study	Traditional tributary area methods & rigid diaphragm methods may be unconservative. Presents a generalized linear shear stiffness method for plywood diaphragms to distribute the shear between lateral resisting elements.
Falk et al. (1989)	SOTA	Reviews literature on low-rise wood diaphragm modeling.
Falk & Itani (1989)	Analytical study	Compares finite-element model to previous flat diaphragm tests.

Table 2.3 cont'd. Horizontal Wood Diaphragm Testing and Analysis

Reference	Methods	Focus of Research
Alsmarker (1991)	Static load tests of 3 flat diaphragm panels. Load parallel to ridge/eaves.	Gypsum fasteners ultimately fail at far above allowable load. Design for elastic fastener failure.
Tissell & Rose (1993)	Static load tests of 5 low pitch trussed diaphragms	Used 8 mm (5/16 in.) plywood on 50x50 mm (2x2 in.) trusses 406 mm (16 in.) o.c. Maximum slope was 2:12. No ceiling material was used in the tests. Includes some MPCT.
Foliente (1994, 1995)	Analytical study	Hysteresis model includes nonlinearity, strength and stiffness degradation, pinching & historical loading.
Tarabia & Itani (1997)	Analytical study	Nonlinear 3-D FEM. Concludes diaphragm rigidity is a significant factor in determining loads on building elements.
Yancey et al. (1998)	SOTA	NIST review of the state of residential design research.
Carradine et al. (2004b); Dolan et al. (2003)	Monotonically increasing, cyclic CUREE	Presents research on the deflection of horizontal diaphragms.
Collins et al. (2005)	Analytical study	Develops 3-D finite element model for a house.
Bott (2005)	Dynamic-elastic load test of 6 flat diaphragm panels. Load perp. to ridge/eaves.	Shear stiffness increased by: Foam adhesive/blocking 259%, blocking 135%, foam adhesive 89%; relative to unblocked diaphragm
Min & Li (2012)	Analytical study	FEM of nine flat horizontal diaphragms.

Table 2.4. Finite Element and Analytic Models of Wood-Frame Dwellings.

Reference	Analysis Method	Focus of Research
Falk & Itani (1988)	Two-dimensional nonlinear FEM	Nonlinear elements model the connections between the fasteners, sheathing & framing members.
Kamiya (1988)	Simple hysteretic loop model	Pseudo-dynamic tests
Kataoka & Asano (1988)	Nonlinear stiffness model	Compared model w/ tests for a two story Japanese post & beam structure.
Mahaney & Kehoe (1988)	Linear	Traditional tributary area methods & rigid diaphragm methods may be unconservative. Presents a generalized shear stiffness method for plywood diaphragms to distribute the shear between lateral resisting elements.
Moss & Carr (1988)	-	New Zealand building code. Tested timber portal frames & excluded shearwalls. Seismic response.
Kasal & Leichti (1992)	Program "ANSYS" nonlinear FEM	Wood stud wall with openings.
Foliente (1994, 1995)	Nonlinear	Hysteresis model includes nonlinearity, strength & stiffness degradation, pinching & historical loading.
Kasal et al. (1994)	Nonlinear	Model of one-story house tested by Phillips [1990]
Kasal et al. (1999)	Nonlinear	Hybrid dynamic model including hysteretic & stochastic methods.
He et al. (2001)	Program "Lightframe3D" nonlinear FEM	Presents FEM which includes individual nail connections.
Masaki & Kenji (2002); Kenji et al. (2002)	Nonlinear FEM	Dynamic model of Japanese house demonstrates 45% increase in loads due to eccentricity.
Lam et al. (2002)	Nonlinear FEM	Dynamic model of individual nail connections in the diaphragm system. Verified w/ a simple box structure.
Symans et al. (2004)	Nonlinear FEM	Modeled behavior of a house using viscous dampers.
Collins et al. (2005)	Program "ANSYS" nonlinear FEM	Modeled hysteretic behavior of a house.
Li (2005)	Program "CASHEW" nonlinear FEM	Used to develop fragility information for light frame shearwalls
Winkel (2006)	Nonlinear FEM	FEM using uncoupled spring model for sheathing-framing and framing-framing nail connections is compared to test data.
Xu (2006)	Program "ABAQUS" nonlinear FEM	General hysteretic model, BWBN, was modified for nailed joints, embedded in ABAQUS & compared with the test data.
Li & Ellingwood (2007)	Programs CASHEW/OpenSees	Models of three typical shearwall types demonstrate applicability of this technique to general WFSFD structures. Concludes that this method can predict WFSFD response and assist in evaluating retrofit methods.

Table 2.4 cont'd. Finite Element and Analytic Models of Wood-Frame Dwellings.

Reference	Analysis Method	Focus of Research
Blasetti et al. (2008)	Program "ANSYS" nonlinear FEM	Modeled hysteretic behavior of shearwalls.
Osteraas et al. (2008)	Nonlinear FEM	Uses programs "SAWS" & "SAPWood" w/ laboratory test data (COLA, CUREE-CalTech, CUREE-EDA) compared w/ documented damage of two buildings due to the Northridge Earthquake.
Pei & van de Lindt (2009)	Program SAPWood	Model using Bayesian predictive distribution fragilities to simulate damage and repair cost. Applied to one story ranch and two-story houses, concluding the method provides reasonable results.
Pang et al. (2009)	Programs CASHEW/SAWS	Fragility analysis of 6 buildings of 2 foundation types with OSB and gypsum sheathing in Central US. Concludes that 1 story WFSFD have good life safety response but can have significant financial loss, 2 story WFSFD may need additional nailing and hold-downs.
Black et al. (2010)	Programs SAPWood/Matlab	Emperical seismic loss model applied to a 2-story, WFSFD. Concludes loss analysis can help evaluate loss, help define performance objectives and guide objective WFSFD design.
Christovasilis & Filiatrault (2010); Christovasilis (2011)	Nonlinear FEM	A 2D FEM with rigid floors including explicit connection elements.
Li et al. (2010)	Programs CASHEW/SAWS	Compares collapse probabilities of WFSFD in Western US with Central & Eastern US and concludes existing ASCE 7 seismic maps do not result in uniform risk.
Pei & van de Lindt (2010)	Programs SAPWood/Nail Pattern	Develop fragility curves based on differing possible construction quality and relating the damage to economic loss. Concludes that retrofits are of limited use in either large or small earthquakes and construction quality has major impacts.
Yin & Li (2010)	Programs CASHEW/SAWS	Examines collapse risk due to uncertainties in ground motion and in shearwall resistance in a Monte Carlo simulation to a 1 story. Concludes these uncertainties result in significant variation in outcome.
Goda et al. (2011)	Progran SAWS	Examined 1415 houses in Richmond, BC using seismic hazard model of Geological Survey of Canada. Estimates sensitivity of analysis to differing assumptions of hazard models, spatial correlation model, uncertainty in ultimate seismic capacity and spectral shape.
Pei & van de Lindt (2011)	Nonlinear FEM	FEM including hysteretic & anchorage behavior is compared to shake-table tests of a 6-story apartment building.

NOTE:

FEM – Finite element model

Table 2.5. Wood-Frame Dwelling Testing.

Reference	Loading	Focus of Research
Yokel et al. (1973)	Concentrated, static & cyclic	Two story house before occupancy. Gypsum wall sheathing, trusses w/ plywood roof sheathing. Measured damping, natural frequency & drift.
Yancey & Somes (1973)	Static, cyclic	Two story HUD "Operation Breakthrough" modular unit. Gypsum wall sheathing, trusses w/ plywood roof sheathing.
Tuomi & McCutcheon (1978)	Static racking at various stages of construction	Component interaction study. One-story. Plywood wall and roof sheathing. Trusses with gypsum on bottom chord.
Boughton & Reardon (1982)	Static	1940s USAF building converted to house. Applied loads to portions of the house to determine system load distribution.
Sugiyama et al. (1988)	Static at specific locations	Tested stiffness and deformation of Japanese house under static loading.
Phillips (1990); Phillips et al. (1993)	Cyclic, ASTM E72	Single story rectangular house. Study indicates roof behaves as rigid diaphragm.
King & Deam (1998)	Dynamic testing	New Zealand code. Evaluated the post-elastic performance of wall panel, used to develop a 'dependable lateral load resistance rating.'
Kasal et al. (1999)	3-D FEM non-linear	Uses statistical properties of building components in FEM to distribute seismic forces to the lateral resisting elements. Then uses SDOF shear model to calculate displacements.
Fischer et al. (2000)	Dynamic uniaxial shake table	CUREe-Caltech two-story single-family woodframe house was tested at UC San Diego. It was 4.9x6.1 m (16x20 ft.), 38x89 mm (nominal 2x4 in.) with OSB & oriented such that shaking occurred along the short dimension of the structure. Tested at 10 different phases of construction.
"	"	Four types of shake table tests were performed for quasi-static inplane floor diaphragm tests, frequency evaluation tests, damping evaluation tests, & seismic tests, at up to five levels of increasing in amplitude.
Foliente et al. (2000); Foliente et al. (1998) Paevere & Foliente (2002) Phillips et al. (1993) Paevere et al. (2003)	Wind loading, static, dynamic and destructive	Tested single story L-shaped house containing required structural elements, with interior finishes. Concluded tributary area method was least accurate, & FEM gave most accurate results.
Kohara & Miyazawa (1998); Miyazawa & Kohara (1998)	Dynamic	Tested 2 story Japanese house.
Ohashi et al. (1998)	Dynamic	Tested 5.4x3.6x2.9m (17x12x8 ft.) tall model house.
Kharrazi (2001)	Shake table & field tests	Vibration & damping tests on shake tables & houses in the field.
Folz & Filiatrault (2001)	Cyclic SDOF FEM	CUREE Development of CASHEW model of displacement & energy dissipation in wood shearwalls.

Table 2.5 cont'd. Wood-Frame Dwelling Testing.

Reference	Loading	Focus of Research
Filiatrault et al. (2002)	Dynamic Uniaxial shake table	CUREE UCSD house. Rectangular, 2-story. Different configurations of sheathing, finish & mass distribution.
Malesza et al. (2004)	Static	Applied static load to house center w/ cables & measured floor diaphragm deflection. FEM 1.45-2.54 times measured deflections, rigid diaphragm model 1.84-4.92 times measured deflections.
van de Lindt & Liu (2006)	Uniaxial shake table	Six tests of a one story house with: (1) the exterior wood shearwalls w/ only oriented strand board (OSB) & no non-structural finishes, (2) the exterior wood shearwalls w/ OSB & drywall, and (3) the exterior wood shearwalls w/ OSB & drywall & a non-structural partition wall.
van de Lindt (2007)	Uniaxial shake table	Tested full-scale & half-scale house models. Determined that scaling was not reliable in wood-frame structures.
Filiatrault et al. (2008)	Dynamic Uniaxial shake table	CUREE 2 story townhouse. Part of NEESWood project.
Xilin Lu et al. (2006)	3-D shake table	Tested 2 story wood-frame structure with I joists, OSB.
van de Lindt et al. (2008, 2007); van de Lindt & Liu (2006)	Uniaxial shake table	Simple 1 story box model, 24 tests of 4 specimens with 6 ground motions.
van de Lindt et al. (2010, 2012); Pang et al. (2010); Pei & van de Lindt (2011)	Shake Table	5 tests of 6 story light framed apartment building. Examines damage, drift & performance of largest full-size structure to date. Part of NEESWood project.
Kang et al. (2009)	ISO 16670 (cyclic, increasing)	Tests of 9 full-scale 1-2 story, light framed structures.
Christovasilis (2011); Filiatrault et al. (2007, 2010, 2008)	Triaxial shake tables	Full-scale, two-story, light-frame wood townhouse building tested at MCEER on two triaxial shake tables. Part of NEESWood project.
van de Lindt et al (2011)	Shake Table	Report of testing a 6 story wood building on a 1 story steel frame. Concludes structure performed well in testing, a first story SMF is a viable option to add commercial space at ground level, and DDD produced better performance than would have been expected under current IBC requirements. Part of NEESWood project.

NOTE:

HUD – U.S. Department of Housing and Urban Development

FEM – Finite Element Model

DDD – Direct Displacement Design, a method of performance based design

Table 2.5 cont'd. Wood-Frame Dwelling Testing.

MCEER – Multidisciplinary Center for Earthquake Engineering Research, University of Buffalo, NY

SDOF – Single Degree of Freedom

USAF – U.S. Air Force

SMF – Special Moment Frame, a type of steel structure

Table 2.6. Post Frame Diaphragm Testing.

Reference	Experiment	Conclusions
Hoagland & Bundy (1983)	Corrugated aluminum & steel attached w/ screws	Developed strength & stiffness values.
Gebremedhin & Irish (1984)	-	Aluminum & steel clad, timber framed, screw fastened diaphragms were tested as deep beams. Variables include direction of ribs, size of supporting grid, diaphragm width to length ratio, fastener spacing, and effect of an 'opening'.
Gebremedhin & Bartsch (1988)	Corrugated aluminum and steel panels w/urethane foam inserts	Strength and stiffness increased 3-7 times w/ foam. Failure was sudden when foam sheared.
Anderson & Bundy (1990)	Corrugated steel with openings	Plane truss analog under or over predicts stiffness by ~10%. Number and type of fasteners have significant effect.
McFadden & Bundy (1991)	Compares cantilever and two-bay diaphragm tests	Both tests gave similar values if the corners of the cantilevered test panel were reinforced.
Bohnhoff et al. (1991)	25 steel diaphragm w/rigid foam between steel and framing	Addition of insulation layer reduces stiffness & strength. Deformation of screws controlled failure mode.
Woeste & Townsend (1991)	19 cantilevered panels	Cantilevered tests need framing stiffeners & out-of-plane restraint to be consistent.
Gebremedhin et al. (1992)	Full- scale post frame building w/static loading	Endwall stiffness highly significant.
Bohnhoff (1992a)	Analytic study	Demonstrates method of calculating frame stiffness & eave loads.
Gebremedhin & Price (1999)	Full- scale post frame building tests	Data show that the roof diaphragm halves act as a unit rather than two independent parts.

Table 2.7. Post Frame Design Methods.

Reference	Experiment	Conclusions
Gebremedhin & Woeste (1986)	Analytic study	Using diaphragm stiffness to redistribute loads resulted in smaller post sizes.
Gebremedhin et al. (1986)	Analytic study	Demonstrates design using diaphragm stiffness to optimize member sizes.
Gebremedhin (1988)	Analytic study	Describes the methods used in "METCLAD" design program.
Gebremedhin et al. (1989)	Analytic study	Describes Met-X-PERT program design methods.
Anderson & Bundy (1990)	Corrugated steel with openings	Plane truss analog under or over predicts stiffness by ~10%. Number & type of fasteners has significant effect.
Bender et al. (1991)	Analytic study	Shows rigid diaphragm analysis results are similar to elaborate ASAE EP484.1 flexible analysis method.
Bohnhoff (1992b)	Analytic study	Demonstrates method of calculating frame stiffness and eave loads.
Niu & Gebremedhin (1997)	Analytic study	Demonstrates method of analyzing post-frame structure in a 3-D model
Carradine et al. (2000)	Analytic study	Demonstrates application of Post Frame Design Methods to timber framed dwelling.
Carradine et al. (2004a)	Analytic study	Demonstrates application of Post Frame Design Methods to timber framed dwelling with SIP panels.

NOTE:

SIP – Structural Insulated Panel, a sandwich panel made from OSB glued to an insulating core.

Table 2.8: Previous Earthquake Damage Surveys.

Reference	Earthquake	Dates	Plans	Conclusions
Kochkin & Crandell (2004)	New Madrid	1811-1812	No	Studied damage to historic homes. No current const. methods.
Berg (1973)	Great Alaska	1964	-	Most damage to dwellings due to earth movement or tsunami.
Morgan & Bockemohle (1973); Pinkham (1973); Steinbrugge & Schader (1973)	San Fernando	1971	-	Contains detailed surveys of damaged wood-frame buildings.
Falk & Soltis (1988)	California	1980s	No	Reviews wood-frame building performance in Ca. earthquakes in 1980s.
EQE Engineering (1989)	Loma Prieta	1989	No	Damaged homes generally pre-1940s w/cripple walls, modern irregular homes, apartments with soft 1st story.
Holmes & Somers (1996)	Northridge	1994	No	Concludes, "the...earthquake should dispel the myth that wood construction is largely immune to earthquake shaking."
Holmes et al. (1996)	Northridge	1994	Yes	Includes plans and photos of 2 damaged houses with analysis.
Comerio (1997)	Loma Prieta	1989	No	Estimated that approximately 12,000 housing units were severely damaged or destroyed, and 30-35,000 incurred some minor damage.
Thywissen & Boatwright (1998)	Northridge	1994	No	Examined database of ATC-20 surveyed structures. Concluded homes were mostly resistant.
Yancey et al. (1998)	Great Alaska, San Fernando, Loma Prieta	various	No	Summarizes recent literature on damage surveys specifically related to house engineering.
Poland & Scawthorn (2000)	Northridge	1994	No	ATC-38 study of 500 buildings <1000' from fault.
NAHB Research Center, Inc. (1994)	Northridge	1994	-	Concludes most single-family dwellings had no structural damage to the roof or walls, but that approximately 50% suffered some damage to the interior or exterior finishes, 7% suffered moderate or high damage to the finishes.
Schierle (2003)	Northridge	1994	Yes	Includes plans & elevations of 4 damaged houses with analysis. (CUREE)

Table 2.9. Current Damage Estimation Methods.

Reference	Name of Method	Purpose
ASCE (2006)	ASCE/SEI 41-06 Prestandard & Commentary for the Seismic Rehabilitation of Buildings.	Employs linear/nonlinear, static/dynamic analyses. Objective is to avoid individual component failure.
FEMA (2002)	FEMA-154 Rapid Visual Screening of Buildings for Seismic Hazards.	Provides a method for a quick visual survey of general structures. Only 9 possible ratings for any dwelling regardless of age, material or complexity of construction. Identifies structures to receive a more detailed analysis.
ASCE (2003)	ASCE 31 Handbook for the Seismic Evaluation of Buildings. (Previously FEMA 310.)	Uses calculations based on simple methods and assumptions. Does not consider system effects of redistribution of forces. Intended to check common potential component deficiencies that might contribute to collapse.
International Code Council (2011a)	2012 International Building Code	These codes provides the basis for engineering design of new structures, prescriptive design of residential and modifications to existing structures. Sometimes these are used to evaluate seismic conformance of existing structures.
van de Lindt (2005)	Reliability Model for Drift Performance.	Damage-based seismic reliability model for light-frame wood structures subject to earthquake load.
Baxter (2004); Baxter et al. (2007)	n/a	Compares different screening, evaluation, rehabilitation and design provisions for wood- framed structures.
Lucksiri et al. (2012)	Rapid Visual Screening of Wood-Frame Dwellings with Plan Irregularity	Approach to screening for seismic hazards in wood houses with plan irregularity is developed. Plan shape, number of stories, plan area, cutoffs in area, and wall openings are investigated.

Effects of Roof Pitch and Gypsum Ceilings on the Behavior of Wood Roof Diaphragms.

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Chapter 3. Effects of Roof Pitch and Gypsum Ceilings on the Behavior of Wood Roof Diaphragms.

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Abstract

Ten full size (3.7 x 4.9 m) plywood roof diaphragms were constructed using metal plate connected (MPC) common and hip wood trusses or joists, typical of single-family dwelling (SFD) construction. The specimens included three gable roof slopes of 33, 67 and 100%, a hip roof of 33% slope, and a flat roof, with a horizontal bottom chord. These roofs were constructed and tested in duplicate to make the total of ten roofs. Gable and hip roofs were tested with plywood sheathing applied to the eaves, with plywood sheathing removed from the eaves, and with a gypsum ceiling attached to the bottom chord of the trusses. Roofs were tested following the ASTM E455 standard procedures and analysis. Results showed eave plywood had negligible effect on diaphragm apparent stiffness; pitch affected gable roof apparent stiffness significantly but did not affect gable roof strength; hip roofs had almost the same apparent stiffness as flat roofs, and had the same strength as flat roofs; gable roofs had apparent stiffnesses which were about 50% that of the flat roofs; and gypsum provided more than 1/3 of the total roof apparent stiffness at slopes of less than 33%. There was no effect of pitch on roof strength in any configuration; all roofs exhibited approximately the same shear strength. Failure modes of roofs included nail withdrawal, nail tear-through, metal plate tear-out on trusses and chord tensile failure.

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INTRODUCTION

Wood-frame structures make up about 90% of the low-rise multifamily or single-family-dwellings (SFD) in North America (Ni et al. 2010). Most SFD have sloped or pitched roofs, yet there has been limited study of pitched wood roof diaphragms in the United States to date, and few wood roof diaphragm tests performed for sheathing attached to metal plate connected (MPC) wood trusses. Gypsum sheathing has been studied for use in shearwalls and design values are provided in various references, but few studies have included gypsum ceilings on MPC trusses as a part of a pitched wood roof diaphragm and there are no design values for gypsum horizontal diaphragms in the present U.S. code documents. Wood diaphragms with non-planar sheathing (such as gable or hip roofs) have only been studied on relatively low slope roofs, less than 33% slope, but current preferences in SFD design commonly uses slopes greater than 33%.

This paper will refer to roof slope as a percent or fraction of vertical rise for each unit of horizontal travel. Pitches express the slope in units of rise (height) per unit of run (horizontal travel). A slope of 0.33 (or 33%) might be expressed as a ratio, 33:100 (4:12), using SI or (customary US) units.

This study compares the apparent stiffnesses of 33%, 67% and 100% (4:12, 8:12 and 12:12) pitched gable roofs and 33% (4:12) pitched hip roofs with that of flat roofs (0% or 0:12) which have been traditionally tested.

A substantial review of the roof diaphragm experimental literature was performed by Kirkham et al. (2013) in an examination of the "state of the art" in seismic design and testing of SFD. The research relating to wood horizontal diaphragms will be briefly summarized here.

Substantial experimentation on wood diaphragms occurred during a period from 1950 to the early 1970s. These experiments were primarily to test different systems using plywood as sheathing. Most of the tests used large, flat diaphragms, loaded horizontally as a simply supported beam to determine maximum loads and the corresponding deflections. These studies were done by the Douglas Fir Plywood Association (Countryman, 1952; Countryman and Colbenson, 1954), at Oregon State University (Johnson, 1955a, b, c; Johnson and Burrows, 1956; Johnson, 1956, 1968, 1971, 1972, 1974, 1979) and by the APA (Tissell, 1967; Tissell and Rose, 1993; Tissell and Elliott, 2000, 2004). Countryman (1952) notes that their study was the first and only study of plywood roof diaphragms known to them, so it is unlikely that there was any research on plywood diaphragms before that year.

Concerns about the effectiveness or contribution of gypsum ceiling panels led to tests by Alsmarker (1991) and Walker and Gonano (1984), both occurring outside the United States. Their results do not appear to have been considered in the US building codes.

Before 1988, the experiment programs tended to use static loading, while more recent testing has involved some dynamic loading (Kamiya and Itani, 1998; Bott, 2005).

Overall, few experimental programs have examined the effect of different roof geometries (hip vs. gable, for example), roof pitch, or the use of light-framed MPC for seismic resistance. Most of the testing programs have used static or linearly increasing loading protocols.

Between 1983 and 1995, substantial research into post-frame construction was performed. The experiments with pitched, corrugated steel roofs on heavy wood trusses led to development of a strength reduction factor based on roof pitch (Gebremedhin et al. 1986). Steeper roofs were determined to be less rigid and have lower lateral load resisting capacity than roofs constructed at a shallower pitch.

RESEARCH GOALS

There has been a shift in the goals of research related to wood roof diaphragms over the recent decades. In the initial experiments conducted in the period from 1950 to 1990, tests were performed on specific building components to determine reasonable design strengths or the "allowable loads" for that component. Factors of safety (FS) were applied to ultimate failure loads to determine reasonable allowable design loads. This is consistent with the building code goal of life-safety. In later experiments, tests of whole houses became the focus but results were difficult to express as an allowable load. Results tended to be expressed as allowable story drift ratios.

Recent experiments testing full-scale SFD have concentrated on damage to non-structural finishes. Shake tables used in these experiments provide a platform which can be programmed to simulate earthquake motions. The rapidly varying accelerations which shake tables provide make force evaluation difficult. In shake table testing, it is rare to see a report which indicates design loads for components. The connection between life-safety based on strength or allowable stresses and damage to non-structural finishes based on applied ground accelerations is unclear. It is not easy to apply these data to present design methods. The change in focus is partially due to an increased interest in performance-based design (PBD) as well as the emphasis of the insurance industry on reducing losses. Many SFD in recent earthquakes have been considered total losses by the insurer even though the structure was considered safely habitable by the city building inspectors.

An important goal of this study was to better understand the performance of roof structures, with respect to diaphragm stiffness. Building deflections have a significant effect on the performance of non-structural finishes. A flexible diaphragm may result in higher damage than more rigid diaphragms.

These were the major objectives of this study:

1. Determine whether roof pitch had any effect on roof diaphragm apparent stiffness or strength,

2. Determine whether hip roofs had the same strength and apparent stiffness as gable roofs of the same pitch,

3. Determine whether roof diaphragm strength or apparent stiffness was increased by the application of gypsum ceiling, and how do differing roof pitches affect this apparent stiffness?

4. Determine how effective roof eave sheathing was when compared with the remainder of the sheathing,

MATERIALS AND METHODS

ASTM E455 describes the testing of full roof diaphragms, either tested as a simply supported beam or as a cantilevered beam fully fixed at one end.

Data Collection

Data from the sensors were sampled once per second by a PC compatible computer running Labview 8.6. Data were collected from 11 channels during the course of each experiment.

An LCD display next to the computer, by the hydraulic controls, showed the raw load-deflection curve for the specimen that was being tested. This real-time feedback allowed the operator to determine when elastic tests had reached the limit of the elastic region, so a test could be terminated before significant damage occurred to the specimen. During the inelastic tests, observing the load-deflection curve gave the operator a method of determining when localized and overall failures were occurring in the test, and provided some warning when the test was reaching maximum values.

Test Specimens

Five different full-size (3.7 x 4.9 m) plywood roof diaphragms were constructed in duplicate from new materials. The configurations included three gable roof slopes of 33, 67 and 100%, a hip roof of 33% slope and a flat roof as a reference. The gable and hip roofs were constructed using metal plate connected (MPC) common wood trusses with 38 x 89 mm members, typical of single-family dwelling (SFD) construction. The common wood trusses were queen-post or fan trusses for the gable roofs and hip roofs, with a step-down truss and jack trusses to complete the hip roof. The flat roofs were constructed using 38 x 140 mm joists to act as references and for comparison to previously reported experiments by others. The bottom of the chords or joists lie in the same plane, so there is no effect of pitch on a gypsum ceiling if one is provided.

Diaphragm sheathing was 12 mm thick type CD Exposure 1, species group 4, APA Rated 32/16 nailed with 8d machine nails, 60 x 2.9 mm, 102 mm O.C. at the edges and 208 mm at intermediate supports. "Bird blocking" was cut from 38 x 140 mm material and nailed between the trusses at the eaves with 8d machine nails, 60 x 2.9 mm. There was no other blocking in the diaphragm. The sheathing was not nailed to the eave or ridge blocking. The nailing was performed according to Table 2304.9.1 of the 2012 International Building Code (International Code Council, 2011), items 10, 11, 13 and 31 with footnote n, using Stanley/Bostich and Senco machine nailers. Machine nails are smaller in diameter than common nails and some adjustment needs to be made for their use in these diaphragms. Typical 8d common nails, (3.33 mm dia.) would have been used at 152 mm O.C. Footnote n reduces the spacing of 2.9 mm dia. nails to 102 mm o.c., which is basically three nails per 305 mm instead of two nails per 305 mm. In ICC ESR-1539 (ICC Evaluation Service, 2011), Table 10, 8d common nails 3.33 mm dia. at 152 mm o.c. have an allowable shear of 3.87 N/mm and 8d machine nails (2.9 mm dia.) at 152 mm o.c. have an allowable shear of 4.01 N/mm a difference of slightly less than 4%. The statement of equivalence also occurs in Table 27 (ICC Evaluation Service, 2011). Therefore, the reduced nail spacing of the machine nails is comparable to the common nails specified in the 2006 International Building Code.

Trusses were manufactured locally and designed by a licensed professional engineer. The top and bottom chords of all trusses were 38 x 89 mm DF-L #1. Trusses were fabricated with tails that were 407 mm long, measured horizontally. Trusses were connected to the double top plates with Simpson Strong-Tie H1 hurricane clips. The H1 clips were hand nailed with Simpson 10d short nails. Blocking was cut to length and fit between the trusses, and machine nailed with 10d (3.3 mm dia.) nails per IBC. Measurements for moisture content were made with a capacitive moisture meter for all sheets of plywood, gypsum, wood members and the trusses. Moisture content of the wood materials measured between 5-10% for all tests during this project.

Eaves were added to the basic roof structure by nailing sheets of plywood with a width sufficient to cover the distance between the top plates and blocking and the mid-point of the fascia boards. Nailing was of the same size and spacing as the basic roof structure sheathing. Fascia boards were 38 x 140 mm material and nailed to each truss end with (2) 16d (3.4 mm dia.) nails. Sheathing was cut to fit the eave extensions. Eave sheathing was nailed to the trusses only with the same nails and nail pattern as the principle sheathing.

Gypsum 12 mm thick was attached to the underside of the wood trusses with 32 mm type W bugle head drywall screws spaced 305 mm o.c. The edges of the sheets which could bear on other gypsum sheets (interior edges) were installed snug tight, but a gap of up to 12 mm was permitted at the top plates on the perimeter.

See Figure 3.1 for a graphic explanation of all the roof configurations and Figures 3.2a and 3.2b showing the testing equipment setup.

These experiments examine the system from the double top plates of a typical SFD, to the pitched plywood diaphragm. This is more representative of the actual construction of a SFD than the large, flat roof diaphragms previously examined.

Test Procedures

Test procedures were based on ASTM E455 (ASTM 2011). There were three experiment series for each constructed roof:

Elastic with eaves - Eave plywood and facia boards were attached to the trusses. The elastic test series (with eaves and without eaves) were repeatedly loaded to a deflection of approximately 30 mm (1-1/4 in.), and then the load was removed. After allowing the roof structure to relax, the elastic test was repeated 3-6 times to obtain consistent performance. This ensured that elastic behavior was observed for both series. The maximum loads during the elastic tests are not relevant to the performance of the system, and do not indicate the strengths of the system. These loads only indicate the maximum loads that were applied while remaining in the elastic range. In order to measure roof apparent stiffness in these elastic tests, a sufficient series of data points was needed to permit calculation of the apparent stiffness or slope of the experiment trace. Each elastic test trace ends at approximately the beginning of the reduction in apparent stiffness of the inelastic tests, plus, little if any curvature is observable at the upper limits of the elastic tests.

Elastic without eaves - Eave plywood and facia boards were removed from the trusses. Elastic tests were performed as in the preceding section.

Inelastic with gypsum ceiling - After the gypsum ceiling was installed, load was applied until the roof had clearly failed. This was typically when some portion of the structure ruptured or until the load-deflection curve had peaked and was declining. There were two inelastic tests performed for each configuration of roof. These tests were performed once for each configuration, because the result of the test is a seriously damaged roof.

Data Analysis

The equations in ASTM E455 (ASTM, 2011) were used to adjust the data to determine the apparent stiffness, G_a, which is adjusted for the diaphragm dimensions to obtain a unit shear value that can be used for design, the ultimate shear strength, S_u, and the adjusted displacement. The apparent stiffness, G_a , is useful where it is necessary to make comparisons to building codes or standards, when values are needed to demonstrate design principles. The adjusted displacement removes the effect of minor changes in position of the roof structure that occurs during the test.

TEST RESULTS

Wood roof systems involve the interaction of a number of members or components acting in multiple planes. There are so many connections and components in a roof system that it is very difficult to track all the forces. The stiffness and consequent deformations of members and their connections affect the portion of the applied load that is distributed to any member.

The framework of joists or trusses, plates, blocking and braces support the roof sheathing. The framework design typically does not resist moments in any of the connections, but instead, the shear resistance of the sheathing when attached to the framework, provides the lateral resistance of the assembled roof. The idealized test framework is assumed not to deflect in a manner that would reduce the sheathing apparent stiffness.

The early roof diaphragm experiments evaluated the stiffness of the plywood as nailed to a substantial supporting framework and blocking. In some cases, these roofs were constructed by nailing plywood to tongue and groove decking. The blocking and framework did not resist moments, but were sufficiently stiff to ensure that weak-axis deformation of the framework members was not a factor in the experiments.

This paper examines the complete roof system including the double top plates of a typical SFD as well as the pitched plywood diaphragm and supporting structure. This is more representative of the actual construction of a SFD than the large, flat roof diaphragms in historic references. The apparent stiffness values from this paper directly show the apparent stiffness of the complete roof structure in a horizontal plane and rather than the plywood sheathing stiffness alone.

All elastic tests for a 33% pitch gable roof and all of the inelastic tests for flat roofs are plotted in Figure 3.3. Figure 3.3a shows a primarily linear response over the range of deformation shown, and comparison of Figure 3.3a with Figure 3.3b shows that deflections of 30 mm or less are within the elastic range for the tested structure.

In Figure 3.3a, the apparent stiffness is shown by the slope of each test. The average slope of the roof with and without eave plywood can be calculated, and the average increase in apparent stiffness from the eave plywood is 6.67%. Examining the slope of tests with and without the addition of gypsum ceiling showed that the gypsum board increased the apparent stiffness 25% on average.

In this brief example, it can be seen that eave plywood adds some stiffness to the structure, but the added value is small. Gypsum added significant stiffness even in the configuration which was most advantageous to the plywood sheathing. Additional and more detailed calculations are in the following sections of this paper.

Overall, the inelastic tests show an increased apparent stiffness varying from 2.51 to 36.6%, due to the addition of the gypsum ceiling. These tests also demonstrate that the elastic tests were performed within the elastic range of the roof system. Figure 3.3a shows only the elastic tests at a different scale for better examination.

For each system, the slopes of all the elastic load-deflection plots for the roof with or without eaves appear similar. Some have shifted right slightly due to the test framework adjusting as the test series proceeded. Though eave plywood provides a few percent increase in apparent stiffness of the roof, it is clearly limited in usefulness as shown in Figure 3.3a. There is more variation (13.7% COV) due to individual roof construction than the 13.6% increase due to the addition of eave plywood (Table 3.2).

For three of the gable roof tests, one for each different pitch, an error in coding of the data acquisition system caused the higher loads to be omitted in some of the inelastic tests. This was caused by an incorrect scaling factor that provided high resolution for individual load points, but resulted in the amplifiers saturating (or limiting out) before the roof actually reached maximum load. Nevertheless, the initial values are similar and provide information about the elastic phase of the experiment. Further, the data do show the effect of the gypsum ceiling used in all the inelastic tests. The only data lost were the maximum load and deflection on those three test duplicates.

In subsequent sections, it should be noted the empirical equations are only from the size and set-up tested here in, other types and connection details will likely have a different formula. Different experimental layouts, materials and constraints will likely produce different results. Correlation equations that follow should be applied only where the conditions are similar and using good engineering judgment.

Comparing Elastic Stiffnesses with Differing Roof Pitches.

To examine the effect of differing roof pitches on the elastic apparent stiffness of the roof diaphragms, see Figure 3.4. It is reasonable to expect differences due to geometric considerations. A flat roof has joists that are solid members that may support both the gypsum ceiling and the roof sheathing. In a flat roof, the sheathing lies all in a plane, parallel to the applied shear load from the structure, and being in one plane together, the individual sheets of sheathing will bear on each other during diaphragm shear across the complete surface. A pitched roof comprised of MPC common trusses has a top chord that is fastened to the roof sheathing, and a horizontal lower chord that is optionally attached to a gypsum ceiling. The flat roof joists have some mechanical restraint on the limits of their weak-axis deflection, because the joists are attached to both sheathing surfaces. Trusses have top and bottom chords which are seldom attached to the same sheathing or surface material. The joist experiences forces on the top and bottom from the different surfaces, but the truss has little ability to transfer weak-axis forces between the top and bottom chord. In addition, the sheathing of a pitched roof lies in different planes, due to the pitch. The planes of the sheathing are not parallel to the applied shear loads from the structure, and the two planes are free to move independently, and do not transfer shear forces by impinging on each other as in a flat roof diaphragm.

As shown in Figure 3.4, hip and flat roof configurations have similar apparent stiffnesses, which are greater than the apparent stiffnesses of gable roof systems, in general, for similar pitches. Also, gable roof apparent stiffness appears to be as low as half of the apparent stiffness of flat or hip roofs. Hip roofs use approximately the same quantity of sheathing material as flat roofs. The primary difference between hip and gable roofs is the pitched sheathing at the diaphragm ends. It seems likely that having this pitched sheathing acts to restrain any torsion in the trusses, causing hip roofs to have similar stiffness properties to flat roofs. (Refer also to the section discussing "torsion on gable trusses.")

Gable roof diaphragms show increasing apparent stiffness with increasing pitch from 33% to 100%. This seems counterintuitive. The effect of increasing pitch is to move the shear resisting the plywood diaphragm web increasingly out-of-plane with respect to the applied force. This appears to be partially counteracted by other effects. As plywood is nailed at "100 mm on-center," and the roof pitch increases, the distance between nails decreases when projected onto the horizontal plane. That is, a 100 mm nail spacing measured parallel to the plywood surface on a 100% (12:12) pitch roof results in a nail spacing of 71 mm apart when measured on the projected plane beneath. 100 mm/71 mm = 1.41, or a 40% increase in nails along each top truss chord. Further, though the projected area of the plywood on the 100% (12:12) pitch roof is no different than the projected area of plywood on the 0% (0:12) pitch roof, there is more plywood used in construction of the 100% pitch roof and the projected thickness of 17 mm is also 1.41 or

40% greater than that of the flat roof at 12 mm. This increased projected thickness increases with pitch. Thus, the apparent stiffness increases with increasing pitch on the gable roofs. There may be other effects of geometry that are important here, but it is sufficient for this paper to show that the loss of efficiency in resisting applied shear can be counteracted to some extent by geometrical factors that also result from the increased pitch.

The analysis of the elastic test data for gable roof apparent stiffness, both with and without eave plywood, as a function of pitch indicates that a linear equation fit to the data is about as good a predictor of apparent stiffness as any higher order curve. This correlation applies only for roof pitches between 33% (4:12) and 100% (12:12):

$$G_a = 109.9x + 180.5 \tag{3.1}$$

where

Ga is the expected apparent stiffness in N/mm, and

x is the pitch as a ratio of rise over a horizontal distance (ex. 4:12 pitch would be x = 4/12 = 0.33).

This can also be expressed as a pitch reduction factor, but dividing Equation (1) by the average apparent stiffness of the flat roofs:

$$\Delta_{G_a} = \frac{G_{a-gable}}{G_{a-flat}} = \frac{109.9x + 180.5}{523.1} = 0.21x + 0.35 \tag{3.2}$$

So a gable roof with a pitch of 0.33 would be 0.419 times as stiff as a flat roof of the same size. Again, this correlation applies only for roof pitches between 33% (4:12) and 100% (12:12).

Comparing Elastic Stiffnesses with and without Gypsum Ceilings.

Elastic tests without eaves or gypsum can be compared to the elastic range tests of the roofs with gypsum and without eaves. Though these tests with gypsum were inelastic tests, the elastic behavior remains a portion of the inelastic tests at low levels of deflection. Therefore, the elastic range can be extracted from the load-deflection curve for use in this comparison as shown in Table 3.1.

All roofs showed an increase in apparent stiffness when a gypsum ceiling was installed on the bottom truss chord. The least increase is for flat roofs, averaging 2.5%. This is not surprising because the plane of the plywood and the plane of the gypsum are parallel. If the top sheathing lies within the plane where the force is applied and the resistance is required, the top wood composite sheathing (plywood in this case) should have higher stiffness than the lower gypsum sheathing. If the top sheathing occurs in the plane at a pitch to that where the force is applied and the resistance is required, the sheathing (gypsum drywall in this case) should have the higher stiffness.

Hip roofs were about 3.6% less stiff than flat roofs tested without gypsum. When gypsum was added, only a negligible improvement occurred with the flat roof (2.51%). When gypsum was added to the hip roof, the apparent stiffness increased 21.4% compared to the hip roof without gypsum. This significant increase in apparent stiffness of the hip roofs tested with gypsum resulted in the hip roof with gypsum being about 12.3% stiffer than the flat roofs tested with gypsum.

For gable roofs only, increased apparent stiffness from adding gypsum in individual tests is about 13.0% to 59.4%, averaging 32%. The least increase is for the highest pitched gable roofs.

Increasing gable roof pitch continues to result in increased horizontal diaphragm apparent stiffness. Analyzing the apparent stiffness values versus pitch indicates that a linear equation fit to the data is about as good a predictor of apparent stiffness as any higher order curve. This correlation applies only for roof pitches between 33% (4:12) and 100% (12:12):

$$G_a = 44.01x + 284.3 \tag{3.3}$$

where

Ga is the expected apparent stiffness in N/mm, and

x is the pitch as a ratio of rise over a horizontal distance (ex. 4:12 pitch would be x = 4/12= 0.33).

Gable roof systems are about half as stiff as flat or hip roof systems, and gable roof systems increase in apparent stiffness with increasing pitch within the range of 33% to 100% pitch. See Figure 3.5 for a graphic comparison of the effects of adding gypsum.

Gable roofs showed increases in apparent stiffness of 27-37% with the addition of a gypsum ceiling, with the lowest pitch showing the highest increase. The higher increase in apparent stiffness at low pitch is not the result of any change in gypsum configuration or application. The gypsum ceiling is identical in all gable tests in all aspects. The reason for the higher increase in apparent stiffness of the gypsum ceiling is due to the lower relative (effective) stiffness of the plywood sheathing due to its differing pitch. There can be no real increase in gypsum ceiling stiffness because all ceilings are identical in construction therefore the contribution of the gypsum to the diaphragm apparent stiffness is the same in all configurations. It is only the reduced stiffness of the plywood that makes the gypsum contribution to the overall apparent stiffness appear higher.

Analyzing the increase in apparent stiffness values for gable roofs versus pitch with the addition of gypsum indicates that a linear equation fit well to the data. This correlation applies only for roof pitches between 33% (4:12) and 100% (12:12):

$$\Delta_{\%} = -0.131x + 0.423 \tag{3.4}$$

where

Δ% is the percentage increase in apparent stiffness with the addition of gypsum as a function of pitch, and

x is the pitch as a ratio of rise over a horizontal distance (ex. 4:12 pitch would be x = 4/12 = 0.33).

At gable roof pitches below 0.33 or 33%, about 38% or more of the elastic roof apparent stiffness is due to the added gypsum ceiling.

Elastic roof behavior is observed below the design strength of 23.9 kN, corresponding to deflections which are about 30 mm for flat and hip roof configurations.

Diaphragm drift can be calculated as follows:

$$\frac{30mm}{3658mm} = 0.82\% \tag{3.5}$$

Gypsum ceilings can be expected to perform well with minimal damage at these drift levels. Therefore, consideration of gypsum ceiling stiffness could be important to understanding the actual performance of SFD that remain in the elastic range.

Gypsum increases the apparent stiffness of gable roofs by an average of 32% and hip roofs by 21%. The increase in apparent stiffness for flat roofs is negligible. Gable roofs with gypsum show increasing apparent stiffness with increasing pitch.

Effect of Additional Plywood on Eaves.

Table 3.2 shows the results of the tests of diaphragm apparent stiffness with and without the eave plywood. Averaging all data produced a net increase of 13.6%, but excluding likely outliers the average improvement was only 2.2%. Therefore the contribution of eave plywood to the strength of a roof diaphragm should be disregarded.

Torsion on Gable Trusses.

During the course of the experiments at high loads, it became apparent that there was substantial deformation to the end gable truss top chord. The loading caused the gable truss top chord to assume an “S” shape (for the three pitches of gable roofs), with the gable truss heels and peak appearing at approximately the original, unloaded conditions

(Figure 3.6). This behavior is observed on both ends of the roof, thus it is likely that each truss in the structure shows a similar deformation. This is believed to be due to the effect of a couple developing between the plywood sheets and the diaphragm chord, to resist the deflection of the plywood sheathing. This behavior was also noted by Johnson and Burrows (1956) without explanation. Diaphragm shear deformations result in double curvature bending of the top truss chords. The stiffness of the system is due to the ability of the individual components and connections to resist deformation caused by the shearing force. Thus the weak axis bending of the gable trusses significantly reduces the system apparent stiffness resulting in the performance shown in Figure 3.4.

Common trusses with pitched top chords and horizontal bottom chords have a smaller weak-axis moment of inertia than flat roofs, therefore the truss will bend more in weak axis bending during roof shear than a flat roof joist. Flat roof joists can be attached to gypsum and plywood on both the top and bottom of each joist, which restrains joist and reduces weak axis bending.

This behavior was not observed on the hip roofs during these experiments. It is likely that this torsional behavior in the gable roof trusses is partially responsible for the lower system apparent stiffness in the gable systems.

Ultimate Roof Strength

A goal of this experimental program was to verify whether a strength reduction factor is needed for the shear capacity of gable and hip roofs of various pitches. Roofs previously tested by other researchers had relatively low pitches, so the effect of pitch could not be verified for certain. By testing roofs of up to 100% (12:12) pitch, the post-frame strength reduction equation indicates a 50% reduction should be applied (see Table 3.3). This should be sufficient reduction that it would be obvious in these test results if this reduction equation is applicable to SFD construction. It was planned to load each roof to ultimate failure and record the results. Unfortunately, calibration problems adversely affected roof specimens 1 to 3 and 5, resulting in no good data for the 33% (4:12) gable inelastic strength tests, and with only one test rather than two for the remaining gable roofs. Results presented here are the best data that was available, but appears to be sufficient to resolve this question.

Maximum shear strength was determined from data records and the value of Su was calculated as described in ASTM E455 (ASTM 2011), based on the horizontal projection of the pitched roof diaphragm and is shown in Table 3.3. In order to compare these experiments with the values shown in the Special Design Provisions for Wind and Seismic with Commentary (SDPWS) (AF&PA 2005), some additional calculations are required and shown at the bottom of the table. The nominal shear capacity without any resistance or safety factor, for 8d nails in 9 mm (3/8 inch) or thicker plywood, loaded perpendicular to the long axis, is 7.01 kN/m (480 plf in Table A4.2B, AF&PA 2005).

AF&PA does not provide design values for gypsum ceilings, so the effect of the gypsum must be estimated from Table 4.3B in AF&PA (2005). For shearwalls with 1/2” gypsum wallboard attached with #6 screws, 200 mm (8 inches) on-center on the edges and 300 mm (12 inches) on-center in the field of the panel is 120 plf. For plywood, Table 4.2A in AF&PA (2005) with horizontal diaphragms, 8d nails, 15/32” thickness, 2” framing, nails 6” on-center has a shear value of 480 plf. Table 4.3A in (AF&PA (2005), for shearwalls, identical conditions, has a shear value of 520 plf. Thus, for a gypsum ceiling, ((480 plf)/(520 plf))(120 plf gypsum shearwall) = 111 plf. In Table 3.3, this value is converted to SI as 1.62 kN/m. (Note that this violates paragraph 4.3.3.2.2 which prohibits summing shear capacities of dissimilar materials for seismic design but permits it for wind design.)

All tests lie within +/- 12% of the Su mean. Based on the results of the postframe design experiments, it might be expected that there would be up to a 50% loss of strength on the steepest roof pitch that was tested, as shown in the rightmost column of Table 3.2. But these data show that the steepest gable roof was the strongest gable roof tested. There is no indication that roof pitch adversely affects the strength of roofs constructed of plywood sheathing and MPC trusses. Tests showed average strength values within 1% of AF&PA (2005) tabular values. Though roof stiffness (and therefore deflection) is affected by pitch, roof strength appears uniform for all pitches tested.

In wood construction, gable roofs are not as stiff as flat roofs, because the upper truss chord can significantly displace relatively and independently from the bottom truss chord as shown in Figure 3.6. The joists supporting a flat roof take on both of the roles of the top and bottom chords.

CONCLUSIONS

The following conclusions can be drawn based on the testing of pitched wood roof diaphragms:

1) Gable roof systems have lower apparent stiffnesses than flat or hip roof systems. Gable roof apparent stiffness can be as low as half the apparent stiffness of flat or hip roof systems, and gable roof systems increase in apparent stiffness with increasing pitch within the range of 33% to 100% pitch.

2) Eave plywood resulted in a net increase of 13.6%, but if outliers were excluded, the average improvement was only 2.2%. Therefore the contribution of eave plywood to the strength of a roof diaphragm should be disregarded.

3) Hip and flat roof configurations have similar apparent stiffness.

4) Diaphragm shear deformations result in double curvature bending of the top truss chords, significantly reducing diaphragm apparent stiffness.

5) Gypsum increases the apparent stiffness of gable roofs by an average of 32% and hip roofs by 21%. The increase in apparent stiffness for flat roofs in negligible. Gable roofs with gypsum show increasing apparent stiffness with increasing pitch.

6) Common trusses with pitched top chords and horizontal bottom chords have a smaller weak-axis moment of inertia than flat roofs, therefore the truss will bend more in weak axis bending during roof shear than a flat roof joist. Flat roof joists can be attached to gypsum and plywood on both the top and bottom of each joist, which restrains joist and reduces weak axis bending.

7) Though roof apparent stiffness (and therefore deflection) is affected by pitch, roof strength appears uniform for all pitches tested.

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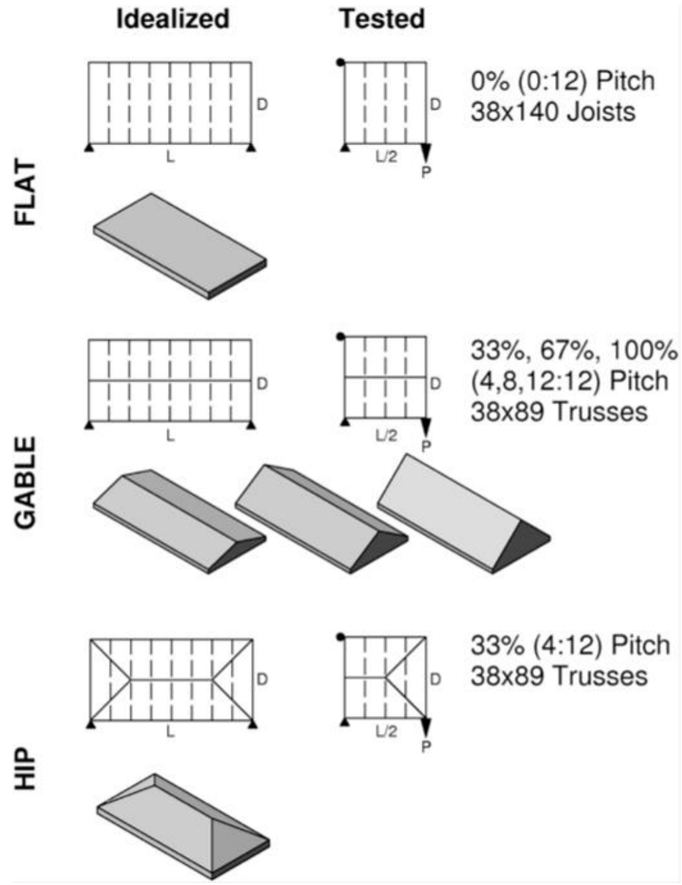
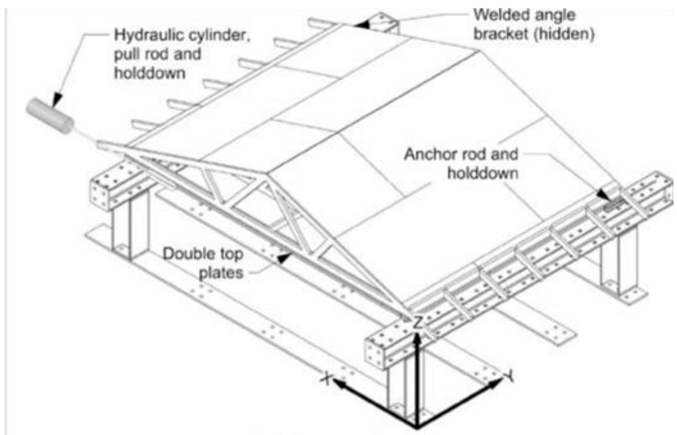
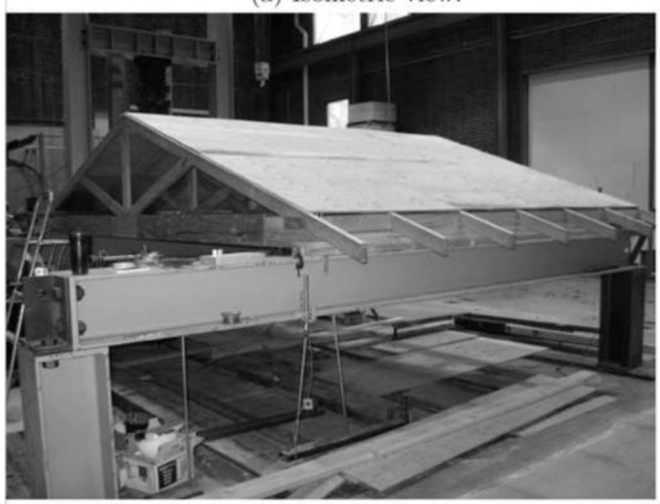


Figure 3.1. Test Specimens

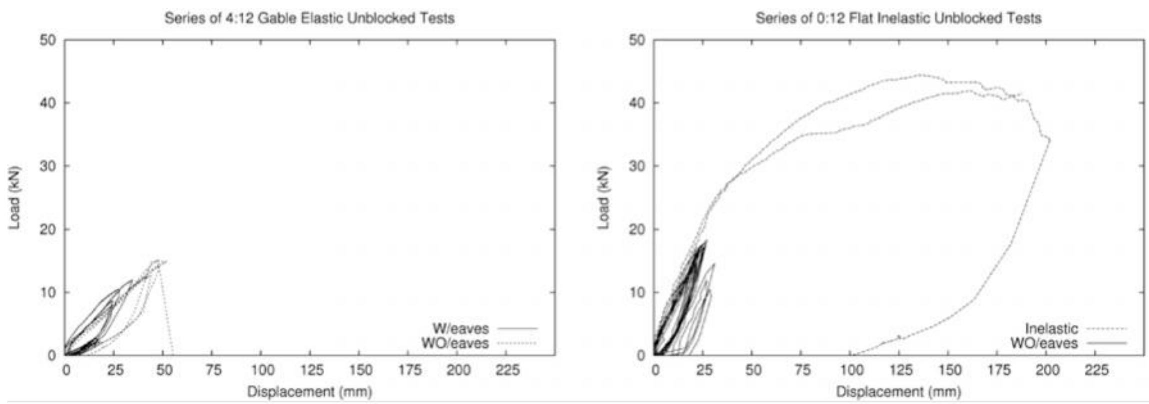


(a) Isometric view.



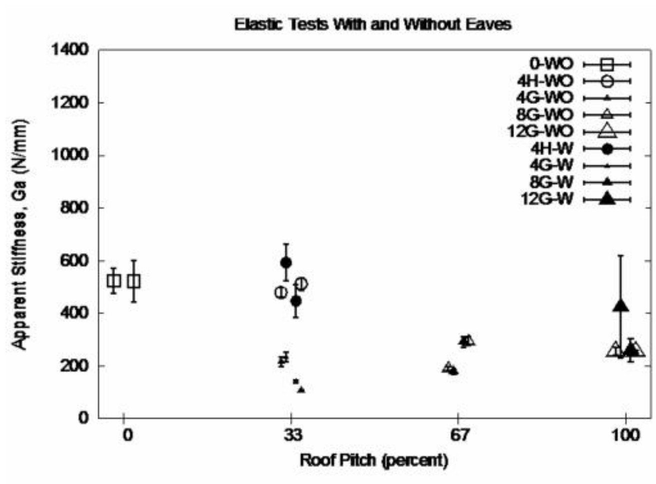
(b) Photo of test rig.

Figure 3.2. Test Rig Diagram and 33% (4:12) Test Sample without Eaves.



(a) 33% Gable Elastic. (b) 0% Flat Inelastic.

Figure 3.3. Examples of Load-deflection Curves for Individual Tests.



Legend is coded as follows: {n}{H or G}-{W or WO},

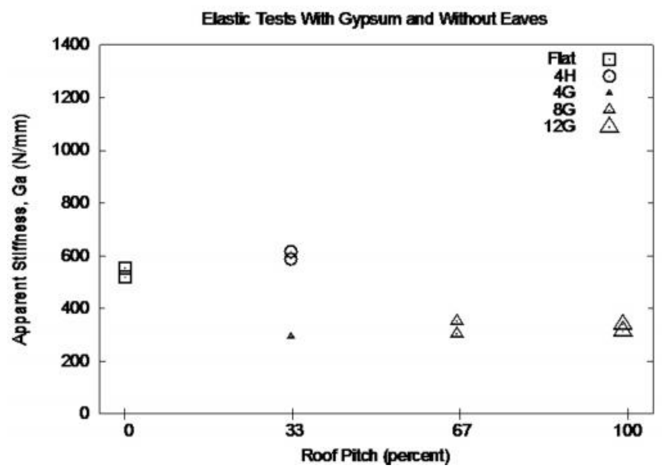
where, n – Pitch as n:12

H/G – for Hip or Gable (no letter for flat roof)

W/WO – for with or without eaves

Thus a 100% (12:12) pitch Gable without eaves is 12G-WO

Figure 3.4. Elastic Tests with and without Eaves.



Legend is coded as follows: {n}{H or G}-{W or WO},

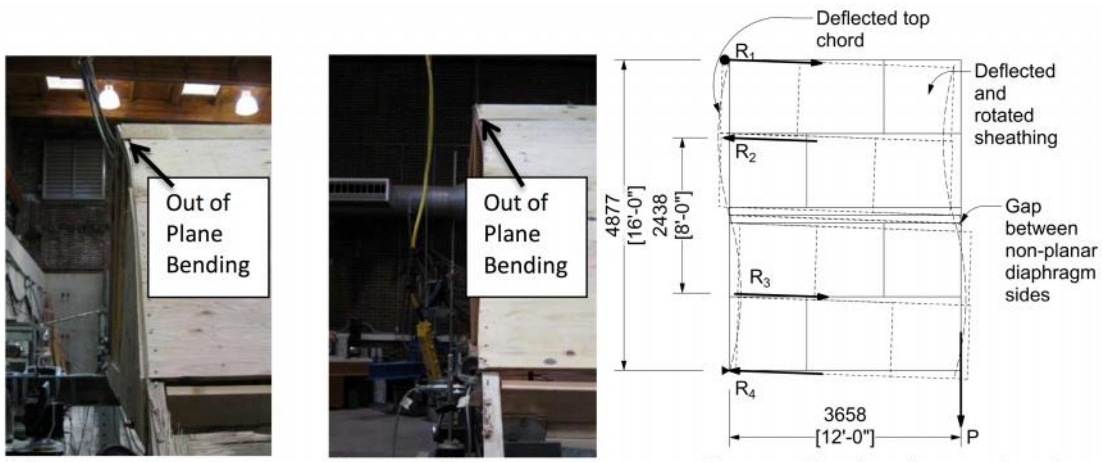
where, n – Pitch as n:12

H/G – for Hip or Gable (no letter for flat roof)

W/WO – for with or without eaves

Thus a 100% (12:12) pitch Gable without eaves is 12G-WO

Figure 3.5. Elastic Tests with Gypsum and without Eaves.



(a)West end from SW corner (b) East end from NE corner (c) Forces developed in tested roof.

Figure 3.6. Observed Out-of-Plane Truss Chord Bending

Table 3.1 - Comparing Apparent Diaphragm Stiffness with and without Gypsum Ceiling.

Shape	Pitch	Without Gypsum					With Gypsum				Increase	Change of
		Apparent Stiffness by Roof			Apparent Stiffness by Pitch		Apparent Stiffness					
		G_a ave. (N/mm)	no. tests	G_a Std. Dev.* (N/mm)	G_a ave. (N/mm)	G_a Std. Dev.* (N/mm)	G_a by roof (N/mm)	no. tests	G_a ave. (N/mm)	G_a Std. Dev.† (N/mm)	(N/mm)	Stiffness (D%)
Flat	0	524.6	8	47.90	523.11	65.37	553.5	1	536.23	24.45	13.11	2.51%
		521.6	8	79.04			518.9	1				
Hip	4	512.2	6	24.23	495.91	26.99	587.6	1	601.98	20.37	106.1	21.39%
		479.7	6	18.48			616.4	1				
Gable	4	215.7	4	18.64	161.52	65.37	294.7	1	294.70	-	78.92	36.58%
		107.3	4	2.324			~~116.5~~ ‡	~~1~~				
	8	190.5	4	7.010	256.43	53.61	303.7	1	318.06	20.35	75.75	31.26%
		294.1	7	18.29			332.5	1				
	12	256.0	4	13.11	255.20	9.80	313.6	1	326.18	17.82	70.85	27.75%
		254.7	6	4.393			338.8	1				

Average of all roofs 23.90%
Average of only gable roofs 31.86%

Table 3.1, cont'd. Comparing Apparent Diaphragm Stiffness with and without Gypsum Ceiling.

Notes:

1) * There were multiple elastic tests for each constructed roof without gypsum, so a standard deviation can be calculated for each constructed roof and shown in line with that constructed roof.

2) † There was only one test for each constructed roof with gypsum because that test was also for ultimate strength (nonlinear). Therefore the std. dev. for the tests with gypsum are calculated for two different constructed roofs and is shown centered vertically between the tests in the table.

3) ‡ It appears that the low strength was caused by a sensor malfunction so this test was not included in the analysi

Table 3.2 - Comparison of roof diaphragm stiffness with and without eave sheathing.

		w/o Eaves & Gypsum			With Eaves & w/o Gypsum			
Shape	**Pitch (%)**	**Apparent Stiffness (N/mm)**	**Std. Dev. (N/mm)**	**20% limit**	**Apparent Stiffness (N/mm)**	**Std. Dev. (N/mm)**	**20% limit**	**Change of Stiffness (D%)**
hip	33	512.2	24.2	102.4	446.8	62.9	89.35	-12.8%
hip	33	479.7	18.5	95.93	592.3	70.5	118.5	23.5%
flat §	0	524.6	47.9	104.9	(No flat roof tests with eaves.)			
flat §	0	521.6	79.0	104.3				
4:12 Gable	33	215.7	18.6	43.15	233.2	19.5	46.64	8.10%
~~4:12 Gable~~ *	~~33~~	~~107.3~~	~~2.32~~	~~21.47~~	~~140.1~~	~~5.94~~	~~28.02~~	~~30.5%~~
8:12 Gable	66	190.5	7.01	38.10	178.4	8.81	35.67	-6.38%
8:12 Gable	66	294.1	18.3	58.82	290.8	19.7	58.16	-1.12%
12:12 Gable ‡	100	256.0	13.1	51.19	424.1	193.2	84.81	65.7%
12:12 Gable	100	254.7	4.39	50.94	258.7	44.0	51.74	1.57%
		Ave. COV	6.04%		Ave. COV	14.1%	Average	13.6%
							Average w/o outliers	2.15%

Notes:

* Line deleted because of data acquisition system was out of calibration on test without eaves. Treated as outlier.

‡ High standard deviation suggests this line is not valid data. Any tests with a standard deviation exceeding 20% of the tested value was excluded.

§ Flat roofs were not tested with eaves. Without eave data are duplicated for comparision.

Table 3.3 - Roof Strength by Configuration and Pitch.

Configuration	Pitch	Roof Strength R_u (kN)	Mean by Config. & Pitch S_u (kN/m)	Deviation from Mean	Postframe Pitch Reduction
Flat	0	41.9 44.4	8.85	2.91%	0.00%
Hip	4	44.2 37.9	8.41	-2.20%	-10.00%
Gable *	4 ‡	8.54	1.75	-	-10.00%
	8	37.0	7.59	-11.70%	-30.77%
	12	46.2	9.48	10.27%	-50.00%
	Mean for all roofs		7.62		
	Std. deviation		3.13		
	Mean excluding 4 pitch gable		8.60		
	Std. deviation		0.79		
	COV		0.09		
AF&PA SDPWS Table A.4.2B, Case 1		Plywood	480 plf -->	7.01	kN/m
Estimated from Table 4.3B		Gypsum	111 plf -->	1.62	kN/m
Total Calculated Shear				8.63	kN/m

Notes:

Tables provided by AF&PA (2005) are only in U.S. Customary Units.

* Only one strength test for each gable roof was completed.

‡ This roof failed to reach the expected shear capacity and has been deleted.

Paper 3.

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A Practical Analysis Method for Partial Diaphragm Rigidity and Torsion in Wood Frame Single Family Dwellings

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ABSTRACT

Seismic and wind design of Wood Frame Single Family Dwellings' (WFSFD) lateral force resisting systems (LFRS) require determination of the stiffness of horizontal diaphragms and shear walls so that building performance can be evaluated. During the design process, sizes and locations of shear wall openings are frequently changed, requiring significant redesign by the engineer.

This study examines rigid and flexible diaphragm analyses for different geometries of L shaped WFSFD, and include stiffness reductions due to differing roof geometry and pitch. These methods are applied to historic earthquake damage reports and compared with a practical rigid, semi-rigid or flexible diaphragm analysis method.

This study determined that most WFSFD should be designed using an envelope method due to a mix of diaphragm types and the effects of roof pitch and geometry on the stiffness. Cases may occur where calculation of semi-rigid or flexible diaphragm behavior is difficult because the code prescribed analysis is contradictory or fallacious. This suggests that use of semi-rigid finite element model (FEM) or a manual envelope

method is prudent. The use of rigid plate, flexible plate or semi-rigid plate (RP, FP or SP) FEA methods can be simple and practical methods for analyzing WFSFD with a reasonable level of detail and effort.

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CE Database subject headings: Diaphragms, Residential Buildings, Stiffness; Seismic Design

INTRODUCTION

The seismic and wind design of Wood Frame Single Family Dwellings' (WFSFD) lateral force resisting systems (LFRS) require determination of the stiffness of the horizontal diaphragms and shear walls so that building performance can be evaluated. The actual rigidity of these flexible diaphragms depends on a number of geometric factors, so methods have been developed to estimate the stiffness of wood diaphragms.

In designing WFSFD, it is typical to move or resize openings (windows and doors). If WFSFD are analyzed with a tributary area method (flexible diaphragms) these changes are easily handled as only the altered wall needs reanalysis. However, this method of analysis ignores torsion effects. Rigid diaphragm methods require extensive reanalysis for each alteration because the force distribution for the walls changes. This has led some practitioners to advocate flexible diaphragm design as the building code approved standard in all cases. Although WFSFD seem to come in almost unlimited configurations, design rules of thumb reduce the number of possibilities so that the effects of the changes in opening size and location can be more readily evaluated.

OBJECTIVES

The specific goals of phase 1 of this study were to:

1. Examine the effects of roof geometry and pitch on LFRS design of WFSFD,

2. Examine existing code provisions with respect to LRFS design in WFSFD to determine areas of concern where strict application of the provisions could be misleading, unconservative or ambiguous.

And in phase 2, to

3. Compare calculated loads from the Tributary Area Method (flexible diaphragm analysis), rigid diaphragm analysis by hand, rigid plate FEM, and flexible plate FEM to selected seismic damage reports.

4. Develop a practical method to evaluate diaphragms of differing stiffness and to consider torsional effects for WFSFD.

LITERATURE REVIEW

WFSFD are complicated to analyze and design. There are two methods commonly used for design of WFSFD diaphragms; tributary area or flexible diaphragm analysis and rigid diaphragm analysis (Breyer et al. 2007). Where there is concern about the method to be used, some authors recommend performing an "envelope" analysis, using both types of analyses and designing for the greatest force on each element (SEAOC 2006; Breyer et al. 2007). Design of bearing wall lateral force resisting systems for WFSFD containing openings can also be designed by the perforated shearwall, segmented shearwall and by force-transfer-around-opening methods (Breyer et al. 2007). Additionally, the building codes have traditionally allowed WFSFD to be designed with somewhat less rigorous requirements than other structures. For example, ASCE 7-10 (ASCE 2010) exempts one and two family light frame dwellings from Section 12.2.3.1, 12.2.3.3 Response Modification Coefficients for Horizontal and Vertical Combinations, Section 12.10.2.1 Collector Elements...Requiring Overstrength Factors and Section 12.8.4.1 which amplifies torsional effects. WFSFD have numerous differences in shape and size as well. The following sections will summarize the recent literature discussing the analysis of WFSFD.

The Residential Structural Design Guide (NAHB 2000) indicates that "... the simple "20 percent" approach to addressing accidental torsion loads is not explicitly permitted in any current building code. But, for housing, where many redundancies exist, the "20 percent" rule seems to be a reasonable substitute for a more "exact" analysis of accidental torsion." No reference is provided. It is also unclear whether the 20% increase is applied to a rigid or flexible diaphragm analysis.

Building Code Requirements

ASCE 7 (ASCE 2010, 2006) allows wood structural panel diaphragms in WFSFD to be idealized as flexible in Section 12.3.1.1.b (prescriptive flexible). In lieu of this, determination of a flexible diaphragm condition can be made according to Section 12.3.1.3, where a horizontal diaphragm is considered flexible if diaphragm deflection is twice the average drift of the adjacent vertical supporting elements under equivalent tributary lateral loading ("calculated flexible"). Loads are determined in Section 12.8, Equivalent Lateral Force Procedure. This is a seismic load method, but ASCE 7 also includes diaphragm flexibility decisions in sections on wind load design without requiring a different evaluation. If a diaphragm consists of concrete slabs or concrete fill over metal deck with a span to depth ratio of 3 or less, Section 12.3.1.2 states that the diaphragm can be idealized as rigid (prescriptive rigid). According to Section 12.3.1, a diaphragm that does not meet either the prescriptive flexible, prescriptive rigid or calculated flexible diaphragm conditions must be modeled as a semi-rigid diaphragm.

Finite Element Models

Researchers have frequently used detailed finite element models (FEM) to study the behavior of experimental structures and used the calibrated FEM to evaluate additional structures with different dimensions or materials. For design of rigid or semi-rigid diaphragm structures, FEM presents challenges. FEM do not determine a COR or determine the eccentricity of the COR with respect to the COM. As a result, it is difficult to determine how to apply the torsion required by building codes because there is no direct way to determine how much torsion is already accounted for in the analysis (Goel and Chopra 1993). In nonlinear systems, the COR will change location as the various resisting elements reach their elastic limits and degrade (Kasal et al. 2004). FEM of WFSFD that include most components are complex because of irregular shapes and connections between different types of material, such as wood to sheet metal or wood to concrete. Thus, no presently available FEM methods can design complete WFSFD to code requirements with a reasonable level of effort (Skaggs and Martin, 2004).

Recent WFSFD Diaphragm Research

The CUREE CalTech Woodframe Project concluded that WFSFD designed with a flexible diaphragm assumption performed better in earthquakes than WFSFD designed according to a rigid diaphragm method (Cobeen et al. 2004). This opinion differs from SEAOC (2006) stating that a rigid/flexible diaphragm "...envelope method...will produce more predictable performance than will use of only flexible or rigid diaphragm assumptions." In this case, an envelope method would involve performing both rigid and flexible diaphragm analyses for the structure and then designing each element for the highest load determined for that element by either method.

Schierle (2003) reported on seismic damage of five WFSFDs in the 1994 Northridge earthquake, with diagrams showing locations and types of damage; one of the most detailed surveys of actual damage available.

Thompson (2000) demonstrated that most WFSFD would have rigid diaphragms if the criteria of the IBC (ICC 2003) or ASCE 7 (ASCE 2010, 2006) were applied.

Ceccotti and Karacabeyli (2002) constructed a 3D FEM with flexible and rigid floor diaphragms using DRAIN 3D. The flexible diaphragm FEM showed longer effective elastic periods than the rigid diaphragm FEM as expected. Increased periods resulted in reduced spectral amplitude and lower seismic forces. Thus, they concluded that flexible diaphragms lead to reduced seismic forces and better seismic performance than rigid diaphragms. They cautioned that their investigation was limited in scope and further work was required for a larger range of buildings.

Paevere et al. (2003) tested a full-scale laboratory WFSFD for lateral wind loads and concluded that the horizontal diaphragms behaved as rigid diaphragms. Kasal et al. (2004) discussed different analysis methods that could be used by designers by comparing each method with a detailed nonlinear FEM calibrated with the full-scale laboratory WFSFD model of Paevere et al. (2003). Among the methods used was a rigid plate model attached to a foundation with springs, based on wind loads applied to a small WFSFD. They concluded that the tributary area method (i.e. flexible diaphragm) was the least accurate in predicting the loads on the experimental WFSFD and methods that include some diaphragm stiffness (i.e. semi-rigid, semi-flexible diaphragms) were best.

Kirkham et al. (2013) tested 10 full-scale WFSFD roofs in flat, hip (4:12 pitch) and gable (4:12, 8:12 and 12:12 pitch) configurations. Pitch did not affect roof ultimate strength, but gable roofs were as much as 50% less stiff than hip or flat roofs, and stiffness of gable roofs depended on pitch. Hip roofs showed similar stiffness to tested flat roofs and were much stiffer than gable roofs of the same pitch. As many WFSFD are constructed with a gable roof system, this difference in stiffness can result in significantly different behavior than previously expected.

Geometric Characteristics of WFSFD

WFSFD come in many shapes and sizes, constructed with different materials and by different methods. Lucksiri et al. (2012) characterized the shape of WFSFD with the parameters shown in Figure 4.1. They found that randomly selected L shaped WFSFDs showed net floor areas varying from 72 to 293 m^2; the ratio of the short side to the long side, R varied from 0.48 to 1.0, averaging 0.82, and the ratio of cutoff area, Cp (shown in Figure 4.1), varied from 0.03 to 0.31, averaging 0.15. They also showed that most of the houses can be classified into 5 shape categories.

It may seem that there are nearly an infinite number of arrangements of walls, doors and window openings in WFSFD. On closer analysis, there are areas of consistency and regularity based on homeowner needs and building code requirements, as described in the Appendix B.

The CUREE Caltech Woodframe Project documented the condition of a number of buildings after the Northridge Earthquake:

Smith Residence (Schierle 2003). The Smith residence (Figure 4.2) was a 115 m^2 single-story WFSFD. Walls were constructed with cement plaster approximately 2 cm thick. The Smith residence suffered cracking of the plaster on the front of the structure. It was selected for this study as it was the only example of an L shaped single story structure with a 2:12 and 4:12 pitched gable roof in Schierle (2003). The Smith residence has an R of 0.69 and a Cp of 6%.

Olsen Residence (Schierle 2003). This was a two story 261 m^2 WFSFD (Figure 4.3), roughly L shaped with a gable roof having a pitch generally of 4:12 except 7:12 over the entry. A large portion of the second level roof extends over a living room with a high ceiling, sloping down to the first floor wall. Walls were constructed with cement plaster approximately 2 cm thick. Although not strictly L shaped, the Olsen residence has an R of about 0.93 and a Cp of about 10%.

Three additional WFSFD are documented in Schierle (2003), but each had damage that was not due to the wood frame design.

MATERIALS AND METHODS

Analysis for this study was performed in two phases. The first phase was an analytical study of the characteristics of L shaped WFSFD, and how the geometric characteristics and pitch affect code provisions and horizontal diaphragm flexibility in LFRS design. In the second phase, different methods of calculating loads in WFSFD were examined and applied to the WFSFD reported by Schierle (2003) and a method of considering diaphragm stiffness and torsion to WFSFD was developed

In the first phase, models were based on the characteristic dimensions and parameters indicated by Lucksiri et al. (2012) (Figure 4.1). An initial model was designed with wall a equal to 9.75 m, wall b 8.53 m, cutout area wall c equal to 5 m and wall d 2 m. A series of models was produced by increasing wall b in 2 m increments to 18.53 m in length, an increase of 217 %. Wall d increased similarly to maintain orthogonal corners while walls a and c remained at the initial conditions. A second set of models began with the initial model and increased length of wall a in 2 m increments to a length of 21.75 m, an increase of 223%. Again, wall c was increased by the same increment while walls b and d remained at the initial condition. These limits were selected as many large urban lots are roughly 30 m on a side, so maximum length of wall a was only 8.25 m less than the width of a large urban lot. These limits produced 44 model configurations, with three test cases each (Figure 4.4), for a total of 132 sample WFSFDs.

Development of Analytical models

Key Assumptions

1. Exterior walls are structural sheathed full length except for door or window openings.

2. Window area in the exterior walls was 10 % percent of the floor area and were evenly distributed to the exterior walls.

3. Two exterior doors occur on the front and back of the WFSFD. The doors are 1 m wide and 2 m tall and do not contain windows, therefore do not count as a portion of the window area.

4. WFSFDs are oriented with the major axis parallel to the street or sidewalk; hence the doors are located on the longest walls of the longest two sides.

5. Although windows, doors and walls have some differences in weight per unit area, the difference was neglected in the determination of the COM. Conversely, the stiffness of the walls is reduced by including windows and doors within the wall, so the COR is affected by the presence of "openings" even if the opening is filled with a window or door.

The Sugiyama and Matsumoto (1994) method as shown in Crandell et al. (1999), was used to evaluate the stiffness of the shearwalls. For the purposes of analyzing many different WFSFD configurations, the "perforated shear wall" method is the easiest to implement and is sufficiently accurate. The "perforated shear wall" design method also allows the practitioner to design shear walls with openings without knowing the precise size and position of each opening. However, if exact locations of openings are known other methods could be used.

The average difference between the methods of Sugiyama and Matsumoto (1994) as shown in Crandell et al. (1999), and that in the SDPWS (AF&PA 2006) is less than 10% for most models in this study. (Refer to Table B.1.)

Data Analysis

This study used the following WFSFD configurations with different distributions of window and door openings (see Figure 4.4).

Case 1:

1. Windows are evenly distributed to each wall based on its portion of the total perimeter. Conservatively assume that all windows are 0.91 m tall, thus a wall with 3 m^2 of windows would have a length of windows of 3 m^2 / 0.91 m = 3.3 m.

2. Wall a has one door and wall a\` (opposite side) has one door; thus the model has one door on the longest side and one door on the opposite side. Doors are 0.91 m wide by 2 m tall. Doors do not have windows, thus the door opening area is in addition to the window opening area in calculation of the total opening area and length of full height sheathing in perforated shear wall design.

Case 2:

Same as Case 1 except all openings are distributed to walls a, c, d and a`. No openings are on either wall b or b`.

Case 3:

Same as Case 2, but additionally assumes that the narrower of walls a or b contains the largest garage door (8.3 m or 4.6 m wide) that will fit. Thus, this wall was significantly affected as a lateral force resisting element.

Model analysis

The 132 WFSFD were generated in a tabular format using Excel. This format permitted detailed implementations of the analysis of each model and ensured that the same design methods were used for each model. The basic dimensional parameters were the lengths of shear walls a, b, c and d.

Characteristic shape parameters R and Cp were calculated from the basic dimensional parameters, as were net floor area, wall area and seismic mass. (Refer to Table B.2 and B.3.) For simplicity, the exterior wall area was assumed to equal the interior wall area in the calculation of the seismic mass. For this phase, the seismic design parameters for downtown Portland, Oregon (Ss=0.983, S1=0.345) were used and ASCE 7 (ASCE 2010) was used to evaluate the seismic load. (Refer to Table B.3.)The service level seismic load is 8% of the WFSFD mass. The Seismic Design Category was determined to be D. Also, as common for light wood framing, seismic mass includes all walls regardless of orientation (Breyer et al. 2007).

Wall and window areas were determined for each case above, and the perforated shearwall adjustment factor Cop was determined for each wall individually according to the Sugiyama-Matsumoto (1994) method as shown in Crandell et al. (1999). For the purposes of analyzing many different WFSFD configurations, the “perforated shear wall” method is the easiest to implement and is sufficiently accurate.

Fully rigid and tributary area (flexible diaphragm) analysis methods were applied to each analytical model. In the rigid diaphragm analysis, the shearwall stiffness was assumed to be proportional to the length of the wall as shown in Breyer et al. (2007). The rigid diaphragm analysis determines shear loads applied to the top of the wall due to direct shear and diaphragm torsion and combines those loads for each wall. The shear wall loads from the top of each wall were applied to the individual walls and to the supported diaphragms to determine the individual shear wall and diaphragm deflections using the method of Breyer et al. (2007). These methods were repeated using the tributary area (flexible diaphragm) analysis method for the condition where the diaphragms idealized as flexible.

Roof diaphragm eccentricities were evaluated for each different WFSFD model and the calculated eccentricities used in development of the graph of eccentricities shown in Figure 4.5. (Refer also to Table B.2.) Trend lines were determined which linked the greatest eccentricities and expressed the eccentricity as a function of shape factor R. (See Figure B.1 for the combination of all cases.)

Horizontal structural irregularities were evaluated in accordance with ASCE 7 Section 12.3 (ASCE 2006). (Refer to Table B.4.) Although some irregularities require components to be evaluated with the code redundancy factor, ρ, this usually only affects collector design and has been omitted from this study (ASCE 2006).

The criteria of ASCE 7 (2006) Section 12.3.1.3 were applied to the tables of shearwall and diaphragm deflections and each diaphragm on both orthogonal axes was determined to be semi-rigid or flexible. (Refer to Table B.5.)

To evaluate the effects of roof diaphragm geometry and pitch, the adjustment factor of Kirkham et al. (2013), which was identical for each case, was applied to diaphragm deflection previously discussed and new determinations of diaphragm flexibility were made for the WFSFD cases with a 33% (4:12) pitch gable roof. For a gable geometry and a 33% (4:12) pitch, the roof diaphragm stiffness was reduced to 42% of the stiffness of a flat roof or a hip roof of the same pitch.

Plate FEM

The second phase of this study involved the use of plate FEM models and applying rigid, flexible and semi-rigid plate FEM to the earthquake damage observed by Schierle (2003). In the current study, the plate model was implemented using a commercially available FEM package, RISA-3D, which is common in small engineering firms and has significant features related to wood structural design.

A Plate FEM in this study was a 2-D model of the horizontal diaphragm consisting of meshed plane stress plate elements with a thickness calibrated to model the stiffness of the diaphragm under investigation (Figure 4.6 and Figure 4.3). The example shown is a two story house modeled as two separate 2-D plates. The floor plans in Figure 4.3 were imported into a CAD program and nodes were created at each corner of each floor diaphragm and any deviations in the wall line. The nodes were exported to a .DXF file and imported into RISA 3D. Plate elements were created between nodes, but since RISA 3D supports only 3 or 4 node isotropic plates, multiple plates were drawn to assemble each floor model shape. The sizes of each of these plates (diaphragm parts) were rounded to the nearest 305 mm to match the grid size. The assembled plates were meshed into elements 305 mm square, taking care to ensure that all nodes at the boundaries of each plate occurred at the same location as the adjacent plates to ensure continuous plate behavior.

The rigid plate finite element model (RP FEM) was modeled as a continuous steel plate, 305 mm thick to provide rigidity. Wall stiffnesses were based on the linear-elastic stiffnesses determined for individual shearwalls as in the preceding discussion on perforated shear walls. Shearwalls were modeled as uniaxial springs connected to the plate at nodes along the shear wall length with resistance only along the axis of the shear wall. This thickness resulted in all nodes having approximately the same deflection under direct shear alone.

The flexible plate finite element model (FP FEM) used a steel thickness determined for each different model to ensure flexibility by the code definition (ASCE 2010, Section 12.3.1.3). This thickness gave a diaphragm deflection that was over 2 times the average wall deflections at that location. Trial and error was required as the distribution of load to the walls depends on diaphragm rigidity relative to the walls. In the RP FEM, nodes along each shear wall are relatively evenly loaded and displaced, due to the rigidity of the plate. The FP FEM places no constraint on the node displacement along each wall, nor does it evenly load those nodes, therefore tension/compression members (diaphragm chords) are required around the perimeter of the structure to provide this constraint.

The semi-rigid plate FEM (SP FEM) used a thickness selected to provide the deflection calculated using the equations given in Breyer et al. (2007). This calibration step was the only change required to convert the FP FEM to an SP FEM.

Walls shown schematically in Figure 4.6 do not exist directly in the FEM. To perform the simplified analysis on a 2 story WFSFD, forces on the uniaxial springs of the 2nd level were summed for each shear wall and applied manually to the shear wall on the 1st level. Seismic loads, P1 and P2 are applied to the diaphragm at the center of the diaphragm mass. By using springs and plates based on a unit size, resulting shear loads can be used directly to select shear wall designs from AF&PA (2006) tables.

If the designer wishes to add additional torsion to the structure to meet code minimum requirements, the torsion can be added in the form of a couple. Figure 4.6 shows a couple on the top level with arrows marked “T”. For models which conform to the assumptions of Case 1, Figure 4.5 can be used to estimate the upper bound on the expected eccentricity.

Case Studies

To further investigate the applicability of the plate FEM, it was applied to cases of documented earthquake damage in WFSFD. Framing was assumed to be 38 x 89 mm wood studs with doubled studs at the wall ends. Floor to floor height was 2.44 m, with a total wood floor thickness of 0.3 m. Average stiffness was from tests of stucco walls by the City of Los Angles and U.C. Irvine (Pardoen et al. 2000). The modulus of elasticity (MOE) of the plates was assumed to be 200000 MPa and the shear modulus was assumed to be 77000 MPa. (These are typical values for steel rather than wood. The diaphragm conditions being examined are primarily very rigid or very flexible, thus exact properties are not crucial, just as the manual rigid diaphragm analysis does not depend on a known value of MOE.) Walls that were offset from a line by less than 0.3 m were assumed to be collinear. Fireplaces in a shear wall were assumed to be additional openings. Seismic parameters for Northridge, CA, were taken from Zip Code 91327, near where the actual WFSFD were located. Seismic loads were based on diaphragm mass including the mass of the walls tributary to each diaphragm. It was not possible to know for certain whether the applied loads were greater or less than these parameters indicate, nor was it possible to know the exact intensity of ground motion in each orthogonal direction at the location of each WFSFD because there was no seismic monitoring in each WFSFD. Modeling was simplified by using a mesh of 305 mm square plates, hence dimensions were rounded to the nearest 305 mm. Portions of structures that appear to be overhangs or decks not supported by structure above or below were disregarded (Figures 4.3 and B.7).

Models include only the exterior walls. The effect of stiffness of the interior walls in a rigid diaphragm analysis is low because the distance of the wall from the center of rigidity is a major factor. Interior walls are closer to the COR unless the structure has significant irregularities. In tributary area analysis, including the interior walls may result in them being assigned much of the seismic load if the method is rigorously followed. Further, the boundary conditions of designed shear walls are fairly well understood, but not so for interior partitions. Interior walls frequently lack hold-downs, rigid support of the wall base and adequate blocking or bracing to transfer shear from the diaphragm into the interior partition.

Seismic loads were calculated using ASCE 7-05 (ASCE 2006), the current code in many areas. Load was applied to each level of the models at the diaphragm COM. This was not strictly accurate, but the diaphragm mass is often the larger component in the COM calculations, and the wall mass distribution is not usually much different unless substantial openings or different wall construction affects the mass of specific walls.

The report of seismic damage (Schierle 2003) contains a few discrepancies between published plans and elevations, so adjustment of the model was required. But, part of the benefit to perforated shear wall analysis is that exact opening positions are not critical.

RESULTS AND DISCUSSION

Eccentricity in WFSFD

In Figure 4.5, eccentricity as a function of the overall shape factor, R can be seen for a rigid diaphragm analysis of Case 1. Eccentricities as percentages, for both orthogonal axes, ex and ey, are plotted against R, so in some configurations, one axis will have near zero eccentricity while for the other axis it may be quite high. As R decreases from 1 (major and minor axes being identical), the maximum observed percentage of eccentricity increases as expected. Based on the shape factor R, the maximum percent design eccentricity, for Case 1 only, can be estimated as follows:

$$e = -0.63R + 0.80 \qquad (4.1)$$

Thus, maximum eccentricity can be estimated from the overall shape factor, R as shown in Figure 4.5. (Refer to Figure B.1 for a compilation of all cases.)

This study also reviewed the torsional shear increases for a WFSFD for a rigid diaphragm analysis. Maximum increase in wall shear was 136% over the calculated direct shear for a small Case 3 house where addition of the garage door effectively reduced the stiffness of that wall to near zero. (Refer to Table B.7.) Average increase was 16% for the addition of torsion to direct shear over all three case types. Thus, the assumption of a maximum increase for shear wall design load for a WFSFD of 20% for torsional effects (NAHB, 2000) appears to be unconservative as demonstrated by Table B.7.

Diaphragm flexibility

Determining diaphragm flexibility was more complex than it might appear. For an L shaped WFSFD, the diaphragms being designed are portions of the overall diaphragm (Figure 4.7). The L shaped diaphragm was divided into rectangular portions that occur between the shearwalls and collectors. The shearwalls and collectors involved depend on the direction of the lateral force. So, the distribution of seismic loads from diaphragms A, B, C and D into their supporting shear walls all need to be evaluated by both rigid and flexible analysis methods to determine the flexibility for each diaphragm as well as for the whole structure. Note that even though there are four diaphragms shown, the structure contains only one roof. The final roof design will be based on the worst case of seismic demand on the four idealized diaphragms.

Model results for individual flat or hip roof diaphragms are shown in Figure 4.8a. There were 132 models, each evaluated for diaphragms A, B, C and D, for a total of 528 individual diaphragms. Sixty-two percent of the individual flat roof diaphragms were flexible, 38% were semi-rigid. Considering the reduced stiffness due to pitch and geometry, individual 4:12 gable roofs (having the greatest stiffness reduction in Kirkham et al. 2013) were flexible for 91% of the models, and semi-rigid for 9%. The change in roof geometry and pitch resulted in a 47% increase in diaphragms being calculated as flexible due to the lower stiffness of the 4:12 gable roofs.

This study also examined whether all four diaphragms for each WFSFD were semi-rigid or flexible (Figure 4.9). For flat or hip roofs, all diaphragms were flexible for 6% of the models and there were no cases where all four diaphragms were semi-rigid. The 4:12 gable roof results showed all four diaphragms were flexible in 18% of the cases and no cases where all four diaphragms were semi-rigid. For the remaining cases, 94% of the flat or hip roof diaphragms and 82% of the gable roof diaphragms were a mix of semi-rigid and flexible, so both rigid and flexible analyses are required for design. (This may be confusing, but the 18% "all flexible" in Figure 4.9b, means that four times as many individual diaphragms (4 * 18% = 72%) were to be flexible in Figure 4.8b. Of the remaining groups of 19% of the diaphragms in Figure 4.9b, 91% - 72% = 19%, were in roof configurations where at least one of the four diaphragms was not flexible.) The change in roof geometry and pitch resulted in a 200% increase in WFSFD where all diaphragms were calculated as flexible. Thus, most WFSFD should be designed using an envelope method due to a mix of diaphragm types and the effects of roof pitch and geometry.

WFSFD diaphragms appear to be primarily flexible when the effects of roof pitch and geometry on stiffness are considered as shown in Figure 4.8b, but examination of Figure 4.9b makes it clear that the diaphragms are not all flexible in all orientations. If the choice of design methods includes only manual rigid diaphragm analysis or the tributary area method, an envelope analysis was required in most cases because the rigid diaphragm orientation cannot be designed by the tributary area method alone.

The assumption that WFSFD have flexible diaphragms has some basis when only some orientations are considered, but when seismic loads in all orientations are examined, a blanket assumption of flexibility seems unwarranted from this study. If more detailed methods are not used, it is advisable to evaluate WFSFD using an envelope method for both rigid and flexible diaphragm design.

A prescriptive method should have overwhelming support based on the professional judgment of experts in the field or substantially positive supporting research. It should be a method which is applicable "beyond a reasonable doubt." The prescriptive flexible method is supported by Cobeen et al. (2004) and Kasal et al. (2004), but cannot be uniformly assumed accurate according to Thompson (2000), Paevere et al. (2003), Christovasilis and Filiatrault (2010), Philips et al. (1993) and Skaggs and Martin (2004). Some researchers recommend an envelope method, basically choosing not to take either side on the issue (SEAOC 2006; Breyer et al. 2007). Breyer et al. (2007) revised a number of locations to consider the new research in this area, and concludes, "Study of the effect of diaphragm modeling choices…is ongoing." Thus the language of Breyer et al. (2007) is not as supportive of the prescriptive flexible method as Breyer et al. (2003), with respect to the actual performance of wood diaphragms. The prescriptive flexible method no longer has either overwhelming expert support or substantially positive supporting research, and thus there should be further discussion of its appropriate use in light of present opinion.

Neither rigid nor flexible

Some confusing situations arise that should be addressed in future code revisions. To determine whether a diaphragm is semi-rigid or flexible, the WFSFD needs to be evaluated at design loads to determine both diaphragm and shear wall deflections. Cases occur where the distribution of loads from flexible and rigid diaphragm analyses are significantly different. For a case with interor walls where the diaphragm was idealized as flexible, the greatest shear load occurs on the interior lateral-force resisting wall when it is aligned with the direction of seismic load application. When the diaphragm was idealized as rigid, the greatest shear loads occurred on the exterior walls. Any attempt to evaluate diaphragm flexibility using the shear loads distributed by either method has a high likelihood of disagreeing. Two potential problems could occur:

Diaphragm is neither flexible nor semi-rigid: A flexible diaphragm analysis may indicate the diaphragm is semi-rigid. When the designer performs a rigid diaphragm analysis, it may indicate the diaphragm is flexible. Hence, there is no definitive answer. This occurred in 62% of our 4:12 gable roof test cases, but obviously this depends on the stiffness of the assemblies involved, so the percentage will vary. (Refer to Table B.8.)

Diaphragm is both: A flexible diaphragm analysis may indicate the diaphragm is flexible, while a rigid diaphragm analysis indicates it is semi-rigid. Here, the designer may be misled to believe that the original "best guess" was correct. Therefore, they are unlikely to go through a complementary analysis and find the opposite conclusion. This did not occur in our analytical models but may still occur in L shaped WFSFD or in other configurations.

A detailed example of this condition appears in Appendix B, "Diaphragm Flexibility Calculations for Paevere House." and Table B.5. Hence, the present methods of analysis by hand do not always lead to definitive results and may leave the designer in a quandary as to how to proceed. Cases may occur where determination of semi-rigid or flexible diaphragm behavior is difficult because the code prescribed analysis is contradictory or fallacious. (Refer to Table B.5.) This situation also suggests that use of semi-rigid FEM or a manual envelope method is prudent.

Structural Irregularities in the models

Of the model WFSFDs examined, 94% demonstrated torsional irregularity type 1a and 40% demonstrated extreme torsional irregularity type 1b (ASCE 2010). (Refer to Table B.3.) It is interesting to review the requirements of ASCE 7-05 (2006) with respect to these types of irregularities. ASCE 7-05 would prohibit rigid or semi-rigid diaphragm structures of type 1b if they were in seismic design category E or F, and increase collector and connection forces in the LFRS by 25% in seismic design category D. In seismic design category D, forces would be increased as already noted but the structure would be permitted. Three dimensional FEM would be required for seismic design categories B, C and D, including the effects of diaphragm stiffness if semi-rigid. Torsion would need to be amplified per ASCE 7-05 eqn. 12.8-14 for most structures, but there is an exception for light framed structures. Special drift limits would be imposed on structures with either type 1a or 1b irregularities.

Re-entrant corner irregularity (type 2), likely for most L shaped structures, appeared in 98% of the models. (Refer to Table B.4.) According to ASCE 7-05 (2006) collector and connection design forces in the LFRS would need to be increased by 25% for seismic design categories D, E and F.

Designers should expect to consider the effects of these horizontal irregularities during the design process. If we wish to improve the performance of WFSFD, it may be useful to begin eliminating some of the exceptions included in ASCE 7 and the IBC that permit design of WFSFD without consideration of torsional effects and re-entrant corner effects.

Analysis by Plate FEM Methods

One of the goals of this study was to compare reported seismic damage to RP FEM, FP FEM, manual rigid diaphragm analysis and flexible diaphragm analysis.

Rigid diaphragm analysis tends to result in larger shear forces on the exterior walls or the walls furthest from the COR, and those with the greatest stiffness. For an L shaped WFSFD, it is reasonable to expect a rigid diaphragm analysis would result in the largest forces on walls a and b (Figure 4.1), which are the longest and most likely have the greatest stiffness. The next group of walls significantly loaded will be walls a` and b`, the next longest walls, probably of intermediate stiffness. Rigid diaphragm analysis assigns the least load to walls c and d, the shortest and also to walls nearest the COR and COM and least affected by torsional forces.

Flexible diaphragm analysis using tributary areas will assign most of the load to interior walls. So, one would expect it to assign the highest loads to walls c and d.

Referring to Table 4.1, a comparison of results for the Paevere et al. (2003) house is shown. Four different analyses were performed: RP FEM, manual calculation of a rigid diaphragm analysis, FP FEM and a hand calculation for a flexible diaphragm analysis by the tributary area method. RP FEM attributes most of the load to walls a, b and b` (Figure 4.10 and Figure 4.1), the stiffest exterior walls furthest from the COR. Three of the greatest loads appear in the RP FEM. Manual rigid diaphragm analysis shows a similar pattern but magnitudes of some loads are different by 50% to 200% from the RP FEM as shown in Table 4.1. FP FEM shows a similar load pattern to the RP FEM, but lower on walls a, b and b`, and higher on walls a`, c and d. The tributary area (flexible diaphragm) analysis shows the highest loads on walls c and d, as expected.

The tendency of the tributary area method to assign most of the load to the interior walls is of concern. Damage surveys (Schierle 2003) showed significant damage on the exterior walls of the subject WFSFD, more consistent with results of the RP FEM. Both plate FEM methods examined here seem to distribute loads proportionately to the stiffest walls, as intuition would suggest, therefore these methods may be best for WFSFD diaphragm analysis. FP FEM loads are similar to the semi-rigid model although more load is transferred to the interior walls; thus if the diaphragm is essentially flexible, this method may appear best.

In Table 4.1, both FEM analyses show torsional loading on walls perpendicular to the axis where the seismic load is applied. For x-axis acceleration RP FEM, walls a, c and a` are parallel to the direction of loading. Therefore, loads shown on walls b, d and b` are due to torsional effects. The same is true for the FP FEM, but in different magnitudes. Only the commonly used tributary area method does not show any torsional loads. Using the FP FEM model, it would also be possible to apply a torsional load if the designer wished by applying a couple as shown in Figure 4.6.

Unaddressed in Table 4.1 are cases where diaphragms cannot be idealized as either rigid or flexible. Practically speaking, the RP and FP FEM are both somewhere between truly rigid and truly flexible and only share some characteristics of each extreme. The distinction between the methods is made only for comparing with manual calculations. There is no difference in implementation of rigid, flexible or semi-rigid diaphragm plate models from an FEM standpoint. The only difference in the models is the thickness of the plate used, so analysis of diaphragms of any rigidity is practical if the thickness can be calibrated to the required stiffness.

Table 4.2 shows the diaphragm deflections at midspan and at each end for the Smith residence using the RP and FP FEM models (Figure 4.2). Plate thickness for each model is shown in the table, as are the calculations for the code criteria to determine whether the calculated flexible criterion is met. Though the code criterion indicates the RP FEM model is semi-rigid, the model was designed to be so stiff as to be very rigid to be more comparable to the manual rigid analysis method. Deflection of the rigid diaphragm is so low that the table shows zero deflection. Both RP and FP FEM of the Smith residence showed wall deflections on the transverse axis over twice those of the longitudinal axis (Figure 4.2). Further, the greatest diaphragm deflection occurs at midspan of the diaphragm for Y-axis seismic loading in both models, but the only damage shown on the actual structure (Schierle 2003) is on the front which the authors expect would be caused by X-axis seismic loading. Thus, the midspan and wall end deflections of the models do not appear to correlate to the reported seismic damage on the Smith residence. It is possible that the cracking reported was not due to shear wall deflection, but may be due to some other mechanism of damage. But at present, the methods available to estimate building component deflections do not appear to correlate with observed damage.

In Table 4.3, resulting loads from applying the 5 models to the Smith residence are shown. The largest design values are shown on walls b and a` in the manual rigid diaphragm, RP FEM and FP FEM methods, and on walls c and d in the flexible diaphragm methods. Even though the diaphragm may be prescriptively flexible according to the code, the FP FEM still has some stiffness and therefore develops some torsion and redistributes a portion of the seismic load to elements based on rigidity. (Again, direct shear occurs on the walls parallel to the direction of loading, along with induced torsion, but the walls perpendicular to the direction of loading show only torsional load which is summarized in the bottom rows of the table.) The manual rigid diaphragm analysis provided the lowest design loads on the cutoff area walls c and d, and the highest loads on exterior walls a, b, a` and b`. Damage reported by Schierle (2003) on this WFSFD was on wall b. The RP FEM, FP FEM and manual rigid diaphragm methods all provided a high design load for wall b. The highest load on wall b by these three methods was from the manual rigid diaphragm method, but it was only 19% higher than the lowest load of the three methods, so all three methods were fairly close. RP FEM, FP FEM and manual rigid diaphragm analysis all provided a high design load to the damaged wall of the Smith residence. The tributary area method also indicated a high load on wall b but showed the highest loads on walls c and d.

The results of a calibrated semi-rigid plate FEM (SP FEM) are also shown in Table 4.3. The pattern of loading is similar to the manual rigid diaphragm analysis and to the RP and FP FEM methods. The loads shown for the SP FEM are within the limits of range shown by the RP and FP FEM. This demonstrates that an envelope method of rigid and flexible diaphragm analysis methods does encompass all maximum loads, By comparing the SP FEM model with the maximum loads shown on walls a, b, a` and b` in the Max. Loads column of Table 4.3, it can be demonstrated that a SP FEM will produce lower design loads than the envelope method if knowledge of diaphragm stiffness is adequately reflected in the model, The envelope method is a reasonable approach where true diaphragm behavior is not known or understood, but may result in overdesign of some elements.

Application of the RP and FP FEM to the Olsen residence (Figure 4.3) is shown in Tables 4.4 through 4.6. Table 4.4 shows the deflections of walls and diaphragms required to verify that semi-rigid or flexible diaphragm conditions of the code were met by the models.

Wall deflections are shown in Table 4.5 for each wall and building level. Bold and shaded numbers show deflections exceeding 24 mm, the recommended 1% story maximum drift to reduce the risk of damage (Pardoen, 2000). Neither method suggests any damage to the walls on Level 2 based on deflection, consistent with the damage report (Schierle 2003). On Level 1, FP FEM deflection suggests damage to 5 walls, but two of the walls had no reported damage, and on one wall that had damage, the FP FEM assigned the lowest drift. So, it is not clear whether the FP FEM correctly described the damage potential, or whether it merely over-predicted deflections on most walls. In fact, the lowest deflection shown in the maximum column for the FP FEM is 23 mm, less than 10% lower than the damage criteria limit, so a slight increase in load would cause all walls to exceed the deflection limit. The RP FEM predicted no damage to Level 1 walls at all based on deflection exceeding the drift limit, however the two walls with the highest drifts were behind brick veneer and could have been concealed from view.

Part of the issue here is also that the damage report indicates no damage to any Level 2 wall, and damage to all Level 1 walls that do not have brick veneer, so it is not clear whether both methods would have been correct or neither correct if the seismic acceleration were higher or lower. It is possible that there was concealed damage behind the brick veneer, or that the brick veneer provided sufficient stiffness to prevent damage on the front walls. Without some differentiation of damage around the structure, it becomes difficult to verify improved computational models. And finally, there is a possibility that the discrepancies in this report were not correctly accounted for in this model.

In Table 4.6, calculated loads are presented for both levels from the RP and FP plate FEM models. Although it is not possible to confirm the actual loading of the structure, the results are interesting. The FP FEM attributes significant load to the level 1 wall H1, with the shortest length but located at diaphragm mid-span and showing no damage. Only one of the three damaged walls, E1, had a calculated load (22.57 kN) that was above the wall average for the FP FEM, and only 22% of the total design load was applied to walls with reported damage.

Contrary to the FP FEM, the RP FEM model attributes almost no load to undamaged wall H1 on level 1. Further, two of the three most highly loaded walls, E1 and F1, were ones where damage was reported, and 57% of the total Level 1 Rigid X-Axis design load (72.12 kN) was applied to these damaged walls.

The RP FEM model was somewhat more accurate for Level 1 in that it did attribute more of the design load to walls which were observed as damaged. Therefore, the RP FEM seems to be producing more reasonable results for the Olsen residence.

The tributary area method will over-predict loading of interior walls if diaphragms are treated as simple span beams between parallel shear walls, and as a result, the loads on the exterior walls may be too low.

None of the methods examined predicted high displacements on walls with the greatest damage. It is not clear why, but at present, it does not appear that any of these methods is very accurate in predicting damage based on exceedance of a deflection standard.

Rigid plate (RP) and flexible plate (FP) FEM produce similar design loads because the FP FEM is not truly fully flexible. Plate FEM and manual rigid analysis methods are the only methods that seem to distribute loads in some proportion to the stiffest walls, therefore these methods appear to be reasonable methods for performing diaphragm analysis. RP and FP FEM both allow some load sharing between walls that is not inherent in either the manual rigid diaphragm or tributary area methods.

The total time required to create and perform the RP and FP FEM models for the the Smith residence was about 90 minutes, while the Olsen residence required about 2.5 hours due to having two stories and a pitched diaphragm at the entry. Performing full tributary area and rigid diaphragm analysis methods would have taken longer. The use of RP, FP or SP FEM methods can be simple and practical methods to add to the engineer's toolbox for analyzing WFSFD with a reasonable level of detail and effort.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions can be drawn based on this study:

- Eccentricity can be correlated with the overall shape factor, R. The assumption of a maximum increase in load for WFSFD shear wall design of 20% appears to be unconservative.

- Most WFSFD should be designed using an envelope method due to a mix of diaphragm types and the effects of roof pitch and geometry.

- The assumption that WFSFD have flexible diaphragms has some basis when only some orientations are considered, but when seismic loads in all orientations are examined, a blanket assumption of flexibility seems unwarranted from this study.

- The prescriptive flexible method no longer has either overwhelming expert support nor substantially positive supporting research, and thus there should be further discussion of its appropriate use in light of present opinion.

- If more detailed methods are not used, it is advisable to evaluate WFSFD using an envelope method using both rigid and flexible diaphragm design.

- Cases may occur where determination of semi-rigid or flexible diaphragm behavior is difficult because the code prescribed analysis is contradictory or fallacious. This suggests that use of semi-rigid finite element model (FEM) or a manual envelope method is prudent.

- Designers should expect to consider the effects of horizontal irregularities during the design process. If we wish to improve the performance of WFSFD, it may be useful to begin eliminating some of the exceptions included in ASCE 7 and the IBC that permit design of WFSFD without consideration of torsional effects and re-entrant corner effects.

- FP FEM makes it possible to apply a torsional load, even though this loading is not easily performed in the Tributary Area Method.

- There is no difference in implementation of rigid, flexible or semi-rigid diaphragm plate models from an FEM standpoint. The only difference in the models is the required thickness of the plate, so analysis of diaphragms of any rigidity is practical if the thickness can be calibrated to the required stiffness.

- The tributary area method will over-predict loading of interior walls if diaphragms are treated as simple span beams between parallel shear walls, and as a result, the loads on the exterior walls may be too low.

- None of the methods examined predicted high displacements on the walls with the greatest damage in 1994 Northridge earthquake residences. It is not clear why this is the case, but at present, it does not appear that any of these methods is very accurate in predicting damage based on exceedance of a deflection standard.

- Rigid plate (RP) and flexible plate (FP) FEM produce similar design loads because the FP FEM is not truly fully flexible.

- Plate FEM methods are the only methods that seem to distribute loads in some proportion to the stiffest walls, therefore these methods appear to be the reasonable methods for performing diaphragm analysis.

- RP and FP FEM both allow some load sharing between walls not inherent in either the manual rigid diaphragm or tributary area methods.

- A semi-rigid plate FEM will produce lower design loads than the envelope method if knowledge of diaphragm stiffness is adequately reflected in the model,

- The use of RP, FP or SP FEM methods can be simple and practical methods to add to the engineer's toolbox for analyzing WFSFD with a reasonable level of detail and effort.

Recommendations:

- Designers of WFSFD should consider the effects of roof pitch in making a determination of a calculated flexible diaphragm condition.

- Designers of WFSFD should be wary of using the Prescriptive Flexible provisions in ASCE 7 and there should be further discussion of its applicability in light of present opinion.

- WFSFD should be designed using an envelope method, or a method such as the plate FEM which considers the contribution of the relative stiffness of different LFRS components.

- In determining whether a calculated flexible condition exists, both rigid and tributary area methods must be used to determine whether the calculated flexible condition truly exists.

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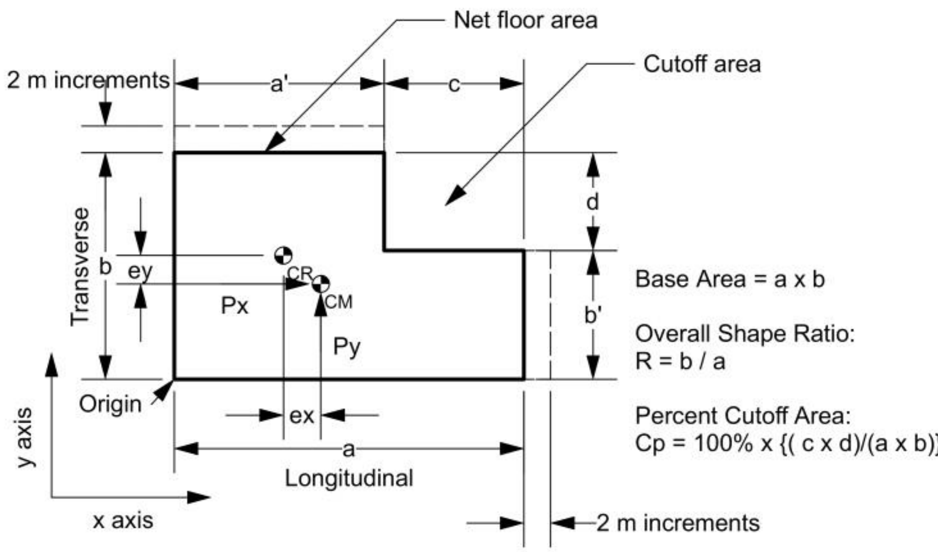


Figure 4.1. Shape parameters for L-shaped WFSFD.

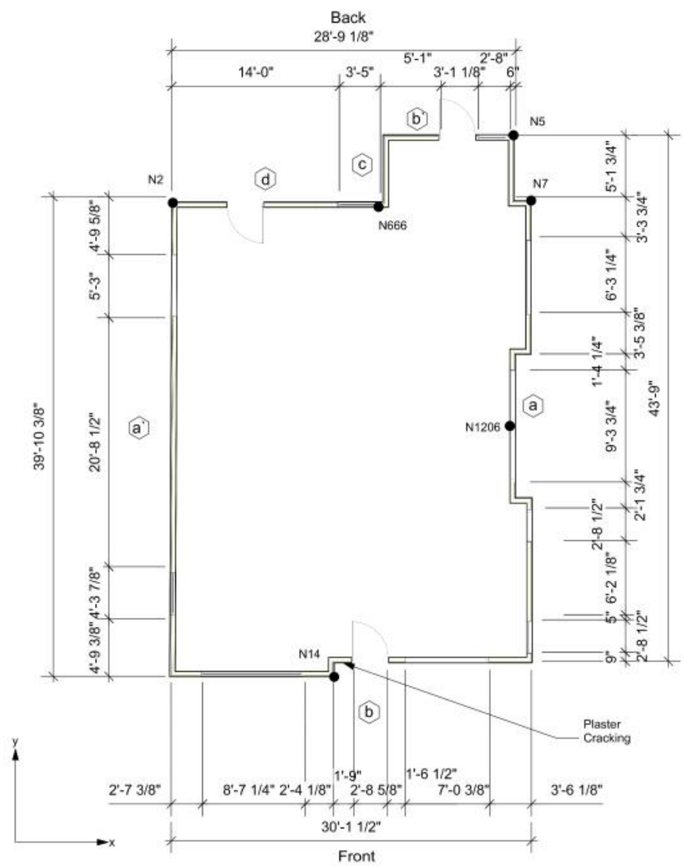


Figure 4.2. Smith Residence.

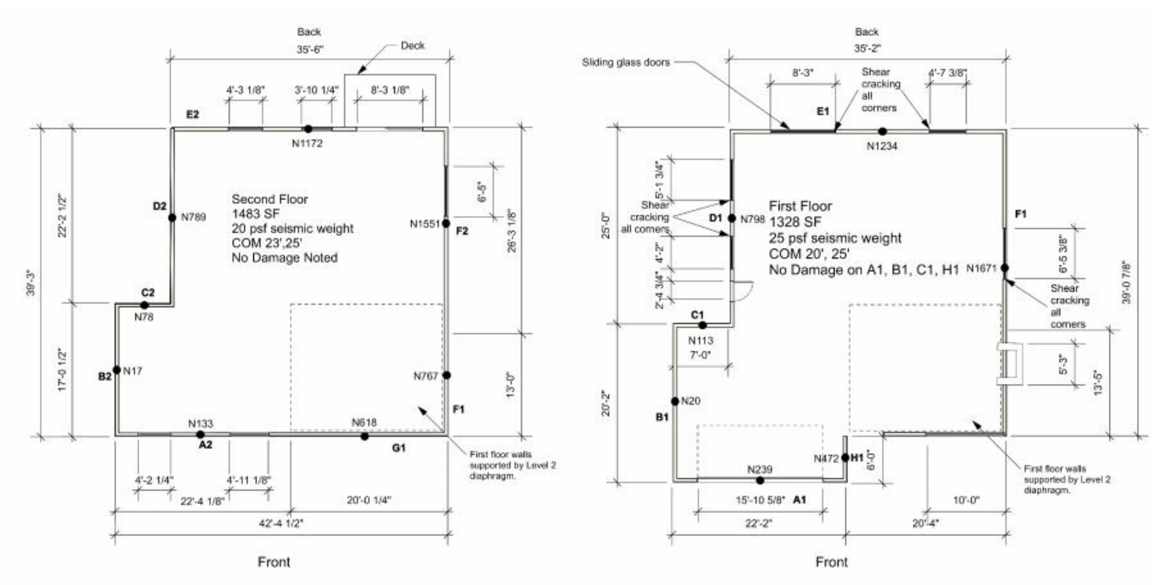
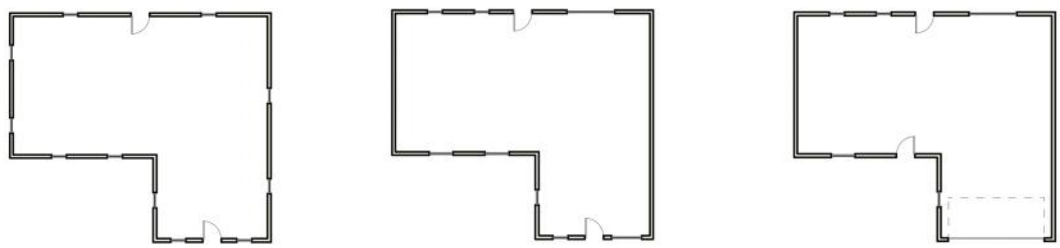


Figure 4.3. Olsen Residence.



(a) Case 1 Uniform Distribution.

(b) Case 2 Front and Rear Distribution.

(c) Case 3 Front and Rear Distribution with Garage door.

Figure 4.4. Analysis cases.

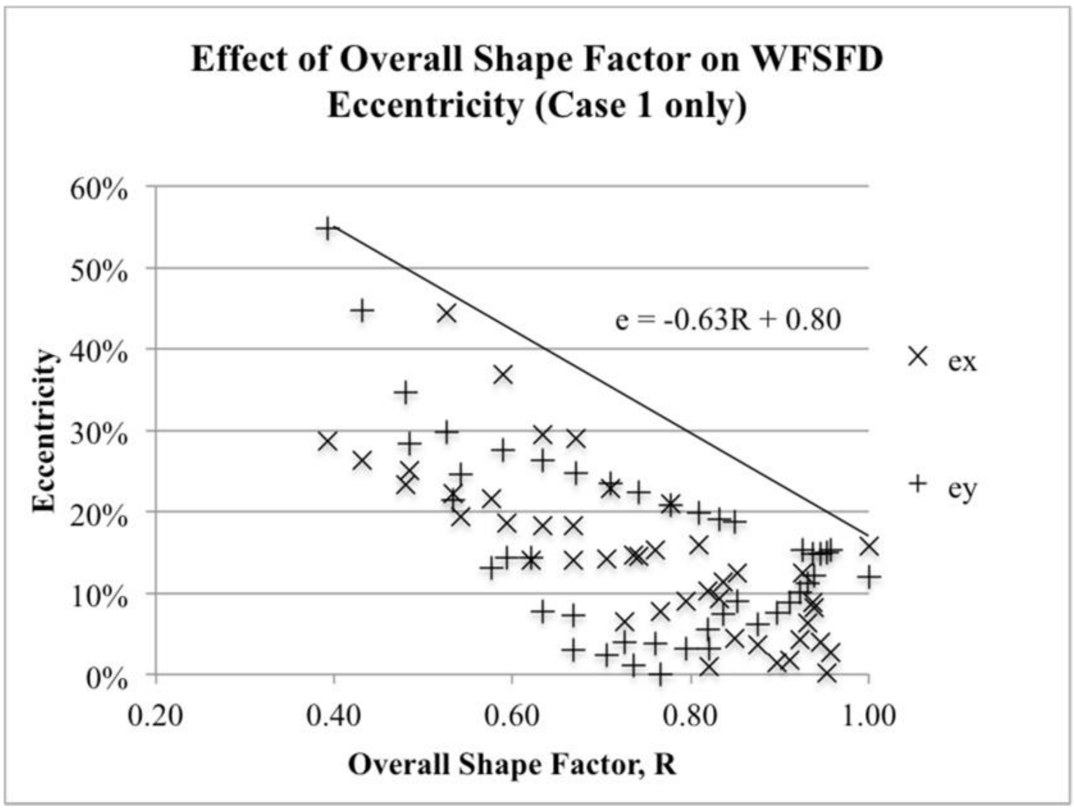


Figure 4.5. Effect of Overall Shape Factor on WFSFD Eccentricity.

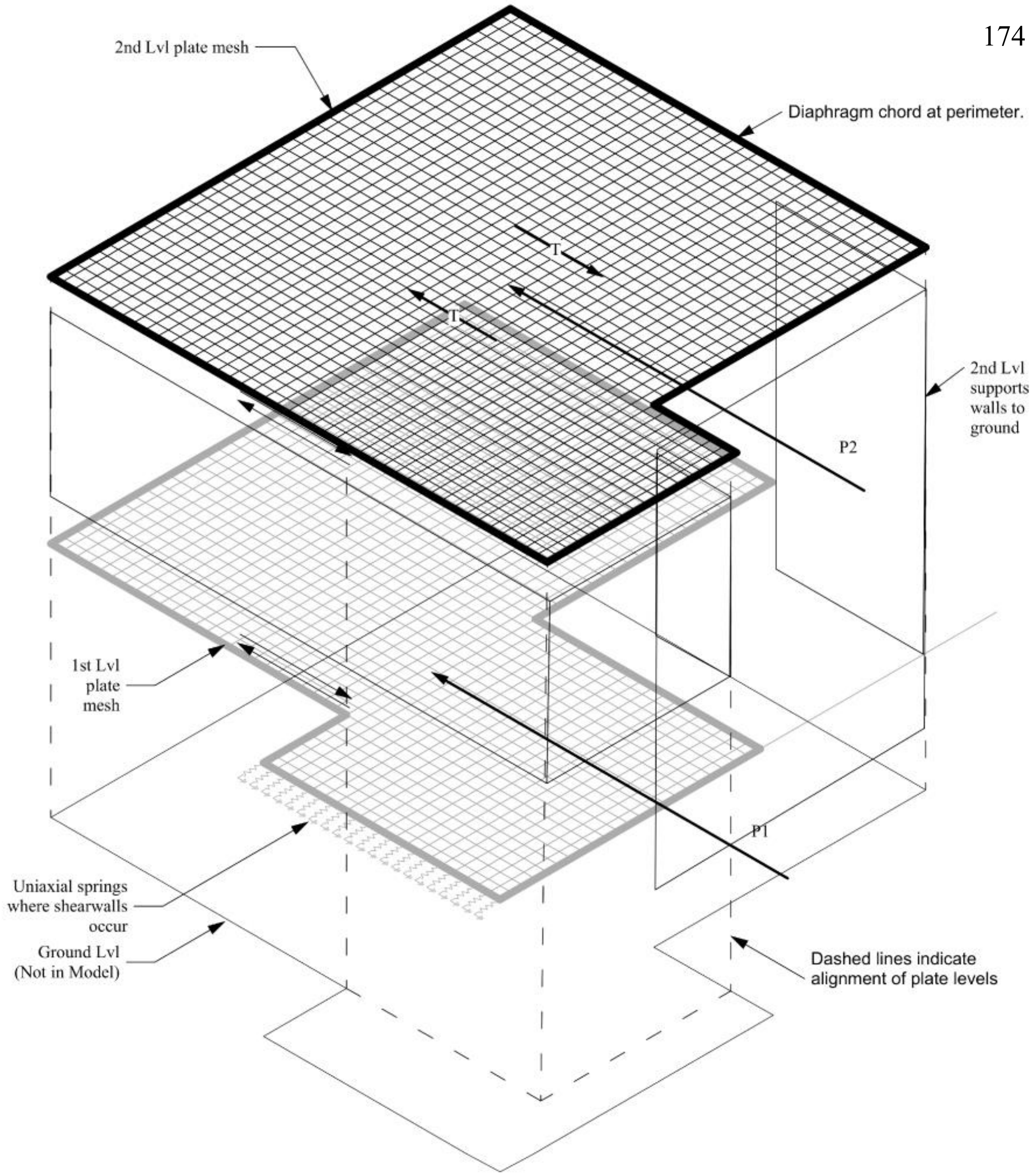


Figure 4.6. Plate FEM Model for the Olsen Residence.

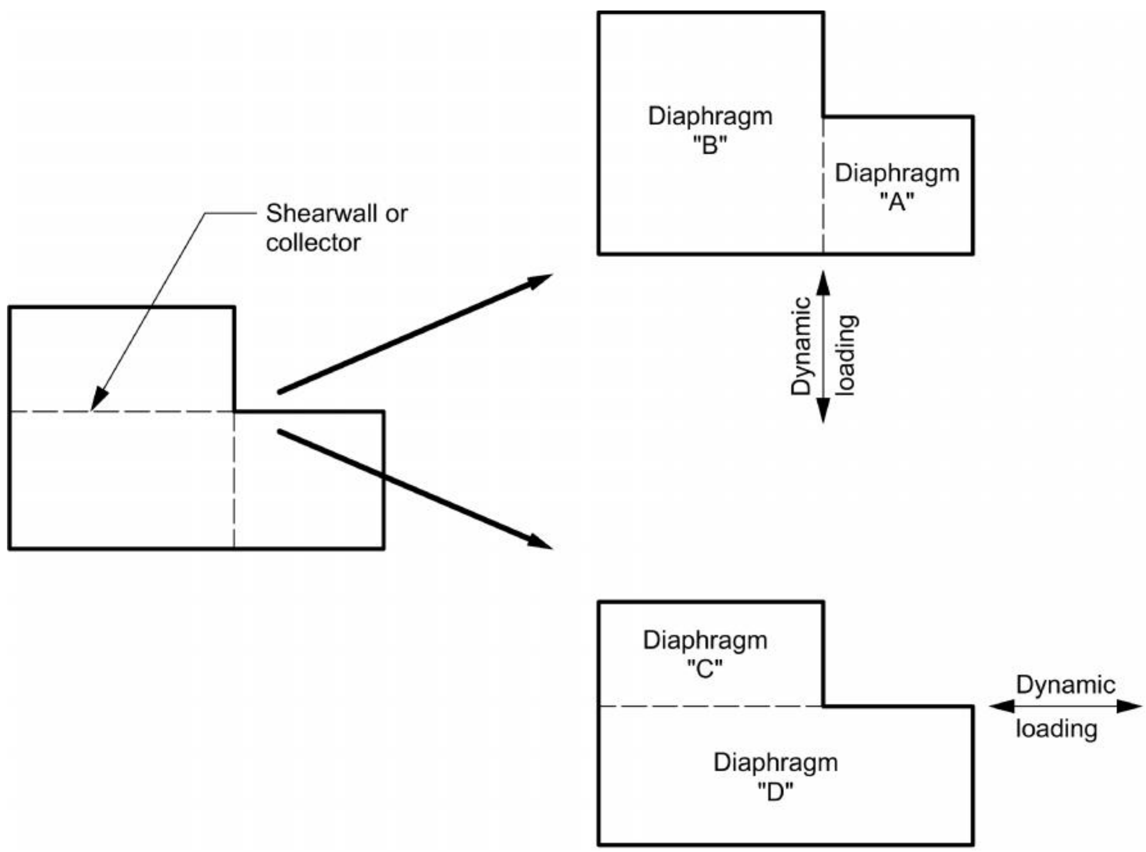
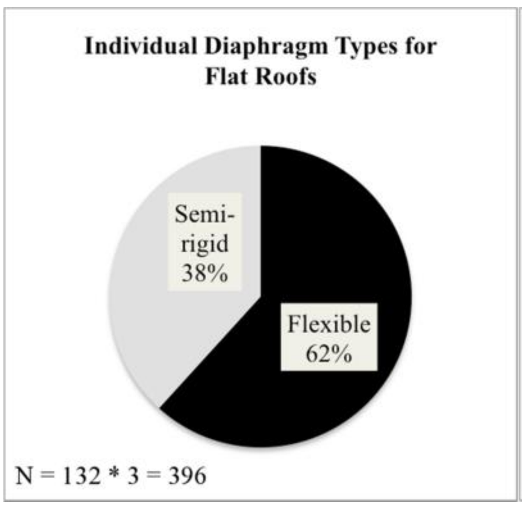
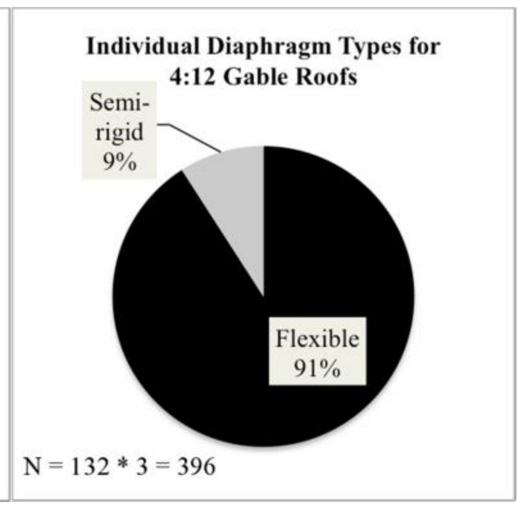


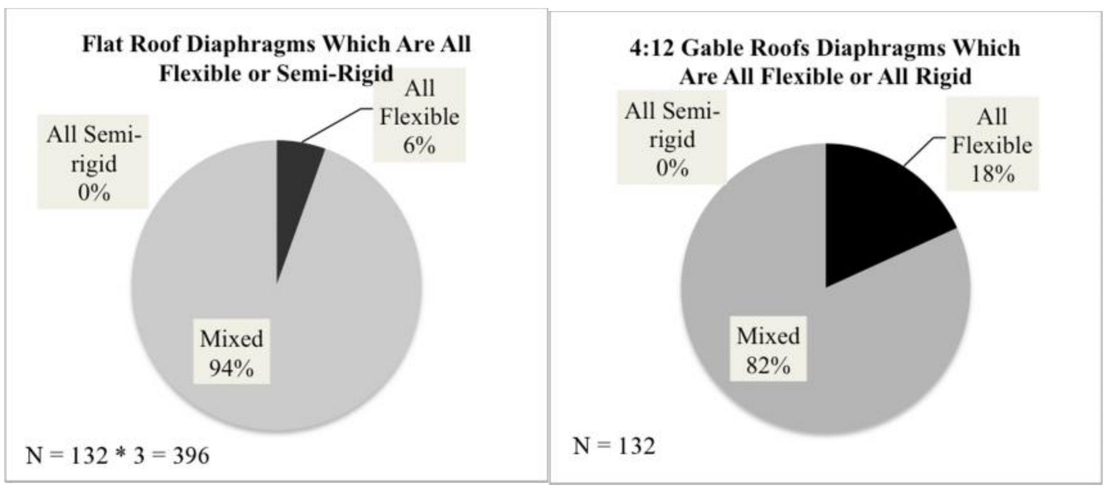
Figure 4.7. Diaphragm configurations studied.





(a) Flat and Hip Roof Diaphragms. (b) 4:12 Gable Roof Diaphragms.

Figure 4.8. Evaluation of Individual Roof Diaphragms.



(a) Flat and Hip Roof Diaphragms. (b) 4:12 Gable Roof Diaphragms.

Figure 4.9. Portion of WFSFD with Roof Diaphragms either all Semi-rigid or all Flexible.

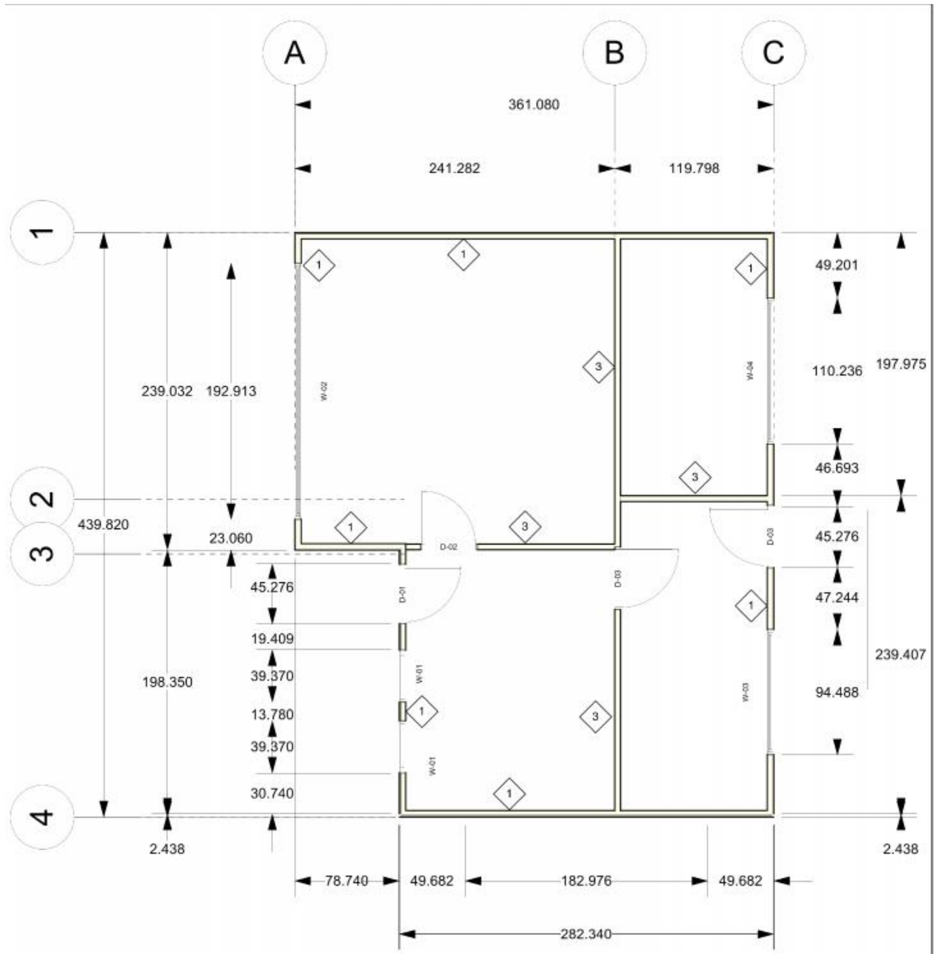


Figure 4.10. Paevere et al. (2003) House.

Table 4.1, Comparison of Diaphragm Analysis Methods for Paevere WFSFD (Paevere et al. 2003).

Wall	Rigid Diaphragm Analysis				Flexible Diaphragm Analysis				Maximum	Minimum
	X-axis accel.	Y-axis accel.	X-axis accel.	Y-axis accel.	X-axis accel.	Y-axis accel.	X-axis accel.	Y-axis accel.		
	Rigid PL FEM		Manual		Flex PL FEM		Flex Trib Area			
	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
a	**6.38**	0.33	4.36	0.11	4.35	*0.02*	3.79		6.38	0.02
b	1.28	**4.12**	0.05	4.52	*0.08*	3.52		2.67	4.52	0.05
c	0.83	0.10	2.48	*0.05*	2.76	1.49	**4.37**		4.37	0.05
d	*0.01*	0.09	0.00	0.96	0.22	1.64		**4.37**	4.37	0.00
a`	1.53	*0.23*	**1.98**	0.07	1.64	1.50	0.58		1.98	0.07
b`	1.52	**3.72**	0.05	3.55	*0.31*	2.88		1.71	3.72	0.05

Direct shear	8.75	7.92	8.82	9.04	8.74	8.04	8.74	8.74
Torsional shear	2.81	0.66	0.10	0.23	0.61	3.00	0.00	0.00
Torsional	32%	8%	1%	3%	7%	37%	0%	0%

BOLD are maxima, *Italic* are minima.

Table 4.2. Diaphragm Deflections for RP and FP FEM for Smith Residence (Schierle 2003).

Level	Load Axis	Plate Thk. (mm)	Location	Deflection on Load (mm)	Ave. Defl. Ends (mm)	Net Defl Center (mm)	Ratio (Ctr Defl to Ave Defl)
Flexible	X-Axis	0.0004	end	4	5	15	3.00
			midspan	20			
			end	6			
	Y-Axis		end	6	10	77	8.01
			midspan	87			
			end	13			
Rigid	X-Axis	305	end	5	6	0	0.00
			midspan	6			
			end	7			
	Y-Axis		end	11	12	0	0.00
			midspan	12			
			end	13			

Table 4.3. Comparison of Diaphragm Analysis Methods for Smith Residence (Schierle 2003)

Wall	Rigid Diaphragm Analysis				Flexible Diaphragm Analysis				Semi-Rigid		Max.
	X-axis accel.	Y-axis accel.	X-axis accel.	Y-axis accel.	X-axis accel.	Y-axis accel.	X-axis accel.	Y-axis accel.	X-axis accel.	Y-axis accel.	
	Rigid PL FEM		Manual		Flex PL FEM		Trib. Area		Semi-Rigid PL FEM		
	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
a	7.98	0.97	6.25	5.14	**9.73**	0.00	4.50	0.00	9.54	0.39	9.73
b	1.07	9.92	1.01	**11.85**	2.41	10.99	0.00	10.14	1.88	10.32	11.85
c	2.25	0.14	0.79	0.28	3.01	0.30	**10.68**	0.00	2.94	0.25	10.68
d	0.62	7.71	1.58	4.66	0.86	8.72	0.00	**10.68**	0.86	8.33	10.68
a`	**11.21**	1.25	10.17	0.60	9.79	0.00	6.17	0.00	9.79	0.53	11.21
b`	0.40	3.65	1.21	**8.25**	0.60	2.05	0.00	0.53	0.50	2.70	8.25

Direct shear	21.44	21.28	17.21	24.76	22.53	21.76	21.35	21.35	22.26	21.35
Torsion shear	2.10	2.36	3.79	6.02	3.87	0.30	0.00	0.00	3.23	1.18
Torsion Percent	10%	11%	22%	24%	17%	1%	0%	0%	15%	6%

BOLD are maxima, *Italic* are minima.

indicates highest two loads for each model type regardless of direction.

Table 4.4. Diaphragm Deflections for RP and FP FEM for Olsen Residence (Schierle 2003).

	Load Axis	Plate Thk. (mm)	Location	Deflection on Load Axis (mm)	Ave. Defl. Ends (mm)	Net Defl Center (mm)	Ratio (Ctr Defl to End Defl)
2nd Level							
Flexible	**X-Axis**	0.038	end	8	7	16	2.143
			midspan	23			
			end	7			
	Y-Axis		end	6	7	14	1.967
			midspan	21			
			end	8			
Rigid	**X-Axis**	305	end	8	8	0	0.005
			midspan	8			
			end	8			
	Y-Axis		end	5	6	0	0.002
			midspan	6			
			end	6			
1st Level *							
Flexible	**X-Axis**	0.038	end	13	18	39	2.133
			midspan	57			
			end	24			
	Y-Axis		end	4	6	17	2.857
			midspan	23			
			end	8			
Rigid	**X-Axis**	305	end	15	19	0	0.005
			midspan	19			
			end	23			
	Y-Axis		end	8	9	0	0.014
			midspan	9			
			end	10			

* Level 2 loads transferred to Level 1 were the "Flexible" load case above.

Table 4.5. Olsen Residence Wall Deflections (Schierle 2003).

			Wall	Deflection		
				X-Axis Seismic (mm)	**Y-Axis Seismic (mm)**	**Maximum (mm)**
LEVEL 2	FP FEM	X-Axis Seismic	A2	7	10	10
			B2	11	3	11
			C2	11	5	11
			D2	9	6	9
			E2	8	21	21
			F2	23	6	23
			F1	14	6	14
			G1	7	16	16
	RP FEM	X-Axis Seismic	A2	8	5	8
			B2	8	5	8
			C2	8	5	8
			D2	8	5	8
			E2	8	6	8
			F2	8	6	8
			F1	8	6	8
			G1	8	6	8
LEVEL 1 *	FP FEM	X-Axis Seismic	A1	24	12	24
			B1	35	4	**35**
			C1	43	10	**43**
			D1	42	17	**42**
			E1	13	23	23
			F1	39	8	**39**
			H1	28	13	**28**
	RP FEM	X-Axis Seismic	A1	23	9	23
			B1	21	8	21
			C1	19	8	19
			D1	17	8	17
			E1	15	9	15
			F1	18	10	18
			H1	22	9	22

BOLD wall identifiers indicate walls with observed damage.
BOLD SHADED numbers highlight walls where models indicate deflection > 1% at wall top.
* Level 2 loads transferred to Level 1 were the FP FEM load case above.

Table 4.6. Calculated Loads for Olsen Residence by FP and RP FEM (Schierle 2003).

			Wall	Length (m)	Unit Load (kN/m)	Load (kN)	Design Load (kN)	Percent of Flexural
LEVEL 2	Flexible	X-Axis	A2	6.71	-1.14	-7.63	7.63	
			B2	5.18	-0.34	-1.74	3.86	
			C2	2.13	-2.67	-5.70	5.70	
			D2	6.78	0.26	1.78	11.18	
			E2	11.13	-1.05	-11.69	11.69	
			F2	7.92	0.00	0.00	9.60	
			F1	3.96	0.04	0.17	3.76	
			G1	6.10	-0.64	-3.91	3.91	
			Total			-28.72	57.33	
		Y-Axis	A2	6.71	-0.09	-0.59		
			B2	5.18	-0.74	-3.86		
			C2	2.13	0.22	0.47		
			D2	6.78	-1.65	-11.18		
			E2	11.13	0.00	0.00		
			F2	7.92	-1.21	-9.60		
			F1	3.96	-0.95	-3.76		
			G1	6.10	0.00	0.00		
			Total			-28.52		
	Rigid	X-Axis	A2	6.71	-1.26	-8.42	8.42	110%
			B2	5.18	-0.06	-0.30	6.50	169%
			C2	2.13	-1.97	-4.20	4.20	74%
			D2	6.78	-0.03	-0.20	9.01	81%
			E2	11.13	-1.09	-12.18	12.18	104%
			F2	7.92	0.04	0.35	9.48	99%
			F1	3.96	0.04	0.17	3.64	97%
			G1	6.10	-0.69	-4.18	4.18	107%
			Total			-28.96	57.61	
		Y-Axis	A2	6.71	-0.10	-0.69		
			B2	5.18	-1.26	-6.50		
			C2	2.13	-0.01	-0.03		
			D2	6.78	-1.33	-9.01		
			E2	11.13	0.10	1.14		
			F2	7.92	-1.20	-9.48		
			F1	3.96	-0.92	-3.64		
			G1	6.10	-0.06	-0.36		
			Total			-28.57		

Table 4.6 cont'd. Calculated Loads for Olsen Residence by FP and RP FEM.

			Wall	Length (m)	Unit Load (kN/m)	Load (kN)	Design Load (kN)	Percent of Flexural
LEVEL 1 *	Flexible	X-Axis	A1	6.71	-1.58	-10.57	10.57	
			B1	5.18	-0.55	-2.87	5.14	
			C1	2.13	-3.66	-7.82	7.82	
			D1	6.78	0.66	4.45	12.07	
			E1	11.13	-2.03	-22.57	**22.57**	
			F1	7.92	-0.61	-4.86	11.80	
			H1	1.83	1.33	2.43	**35.15**	
			Total			-41.80	105.11	
		Y-Axis	A1	6.71	-0.15	-0.98		
			B1	5.18	-0.99	-5.14		
			C1	2.13	0.34	0.72		
			D1	6.78	-1.78	-12.07		
			E1	11.13	0.01	0.16		
			F1	7.92	-1.49	-11.80		
			H1	1.83	-3.30	-6.03		
			Total			-35.15		
	Rigid	X-Axis	A1	6.71	-1.50	-10.08	10.08	95%
			B1	5.18	0.72	3.71	**10.96**	213%
			C1	2.13	-1.63	-3.49	3.49	45%
			D1	6.78	0.18	1.19	6.14	51%
			E1	11.13	-2.45	-27.28	**27.28**	121%
			F1	7.92	-0.67	-5.32	**13.88**	118%
			H1	1.83	-0.16	-0.29	0.29	1%
			Total §			-41.56	72.12	
		Y-Axis	A1	6.71	-0.10	-0.69		
			B1	5.18	-2.12	-10.96		
			C1	2.13	-0.06	-0.12		
			D1	6.78	-0.90	-6.14		
			E1	11.13	0.06	0.65		
			F1	7.92	-1.75	-13.88		
			H1	1.83	-2.29	-4.19		
			Total §			-35.33		

BOLD wall identifiers indicate walls with observed damage.

BOLD and shaded numbers are walls with loads above the average of all walls.

* Level 2 loads transferred to Level 1 were the "Flexible" load case above.

§ Total base shear is 43.15 kN, but walls F2 and G1 are not

Chapter 6: General Conclusions

Knowledge needs to be created to ensure that WFSFD can be designed and built to resist seismic loads to the level expected by building owners, civil authorities and society expectations. Improvements in the following areas are crucial to improving seismic performance of WFSFD. Research has not addressed many areas in seismic behavior of WFSFD.

Innovative Methods. Conventional construction methods were developed to be cost-effective and easily installed. For example, the use of short or 'pony' walls to span vertically from a short concrete foundation to the first level of a house built on sloping terrain. But these methods have been difficult to analyze and research has shown some of them to be ineffective. Thus, there is a significant need to develop new and innovative construction materials, connections, fasteners and techniques to overcome the limitations of wood, such as increasing system ductility.

Brittle Finishes. Present research has concluded that brittle materials are of limited value in providing seismic resistance. There has been limited research to improve seismic performance of brittle materials, such as gypsum wallboard, including the effects of openings, nor to improve ductility in the construction of shearwalls designed with brittle materials. Brittle finishes may have stiffness and strength that can be exploited if ductile methods of connection can be developed. Use of elastomeric sheets, resilient channels or ductile fasteners could be routes to achieve this. Examination of using all of the gypsum walls in an SFD may result in elastic (non-damaging) performance. There is also substantial opportunity to study the behavior of MPCT (metal-plate-connected trusses) in WFSFD lateral force resisting systems (LFRS), as wells as combinations of gypsum board ceilings with structural wood sheathing on the MPCT and on flat roofs.

Horizontal/Pitched Diaphragms. Abundant data exist on rectangular horizontal or low pitch gable roofs particularly with a heavy timber supporting framework. Different configurations (L, T and U shapes, for example) need to be tested, as do roofs of differing pitches and hip roofs. It needs to be determined whether a shear stiffness or strength reduction factor similar to post-frame design is applicable to WFSFD. OSB and structural insulated panels (SIP) roofs should also be tested to verify whether existing data are applicable to their design. Assumptions of flexible diaphragm behavior continue to persist in the building codes in spite of research indicating that the assumption is not valid for all structures; therefore additional research is needed to show that the assessment of diaphragm flexibility needs to be made in each case by the designer. If pitch results in a stiffness reduction factor, some rigid roofs could be flexible or semi-rigid or rigid at different pitches. Research should address how a stiffness reduction factor, if any, affects design of SFD horizontal diaphragms.

Finite Element Methods. Researchers have contributed much effort in finite element modeling of wood structures, but have not yet developed consensus methods and elements that should be used. PBD may result in better designs for buildings than the present code based methods. However, to date, different researchers have used different methods of analysis and design. As a result, it is difficult to compare the accuracy of models of structures using PBD, different FE elements and techniques. Research comparing these methods may assist in determining which would be most useful to the practitioner. For many practitioners, PBD methods will need to be codified to result in widespread use. But at present, few of these methods have been adopted or provided in the commercially available finite element software, limiting use by design practitioners. Synthesizing the existing research and disseminating this research to the designers is the greatest challenge here.

Whole Structure Testing. Historic tests of WFSFD have been of limited use because it has been so difficult to completely quantify the structure so as to allow an independent researcher to refine their analysis methods. To date, there have not been enough consistent, comparable data to permit evaluation of the significance of building geometric factors on the behavior of the structure. Whole house testing had primarily measured damage to the WFSFD components, rather than determining whether a limiting behavior has been reached by the WFSFD as a whole. Therefore, such testing is not easily correlated with the testing of individual components. Research on shearwalls, diaphragms and other components is usually based on yielding performance as a method of determining whether life-safety goals are being met. Such tests generally do not measure the amount or type of damage at various loading intervals. Substantial recent progress in testing large structures has been made. Understanding and integrating the measured results into present analysis methods remains the major challenge.

Damage Estimation Methods: Damage estimation methods seem to be well developed at present, and are mainly products after the Northridge earthquake. Additional opportunities for research in this area will require further comparison to concurrent experiments (such as application to a shake table structure before testing) or await the next significant earthquake in the U.S.

Damage Surveys. More complete reports of damaged WFSFD are needed. Open access to California plans and documents on WFSFD for research would assist this effort greatly. (California is not the only state affected by earthquakes, but earthquakes are common and the laws restricting release of the original plans affect researcher's access to data that might improve design.) There is a challenge to define the required document sufficiently to permit detailed analysis while protecting the designer from the risk of losing their intellectual property within the plans. It's important to include more information in the future because these structures are of typical construction and have gone through major natural events, characteristics not necessarily true of WFSFD constructed for laboratory research. Further refinement in methods of documenting the existing structure and communicating that data to future researchers is needed so that the present or future PBD models can be applied to real structures with real damage. It would be helpful to test some structures or portions of structures using the different testing protocols developed to date, to determine which protocol(s) best simulate(s) actual seismic stresses, deflection and damage. Application of FEA to sample damaged structures before demolition, would allow more accurate modeling to be performed.

Collaboration. Research continues along paths that seem most likely to improve design and evaluation of WFSFD. The following trends seem very positive: full-scale shake table tests of large structures; comparison of tested structure performance with results from finite element design programs, both for strength and prediction of deformation of components; and multi-researcher projects where test results have been analyzed, and finite element models produced by researchers either from the same institution or operating under the same grant, thus ensuring access to sufficient structural detail to permit accurate modeling. There is a strong need to develop a better understanding of the effects of WDSFD components, attachment, LFRS and how loads are distributed to these elements within the structure. A consensus needs to be developed on performance objectives for WDSFD with respect to damage and repair costs, including new design techniques which balance life-safety with the effects of damage to building finish materials. Finite element analyses of seismically damaged WFSFD would lead to a better understanding of component performance and allow evaluation of seismic testing protocols.

Few experiments have been performed on roof construction that is currently common in WFSFD. The following conclusions can be drawn based on the testing of pitched wood roof diaphragms:

8) Gable roof systems have lower apparent stiffnesses than flat or hip roof systems. Gable roof apparent stiffness can be as low as half the apparent stiffness of flat or hip roof systems, and gable roof systems increase in apparent stiffness with increasing pitch within the range of 33% to 100% pitch.

9) Eave plywood resulted in a net increase of 13.6%, but if outliers were excluded, the average improvement was only 2.2%. Therefore the contribution of eave plywood to the strength of a roof diaphragm should be disregarded.

10) Hip and flat roof configurations have similar apparent stiffness.

11) Diaphragm shear deformations result in double curvature bending of the top truss chords, significantly reducing diaphragm apparent stiffness.

12) Gypsum increases the apparent stiffness of gable roofs by an average of 32% and hip roofs by 21%. The increase in apparent stiffness for flat roofs in negligible. Gable roofs with gypsum show increasing apparent stiffness with increasing pitch.

13) Common trusses with pitched top chords and horizontal bottom chords have a smaller weak-axis moment of inertia than flat roofs, therefore the truss will bend more in weak axis bending during roof shear than a flat roof joist. Flat roof joists can be attached to gypsum and plywood on both the top and bottom of each joist, which restrains joist and reduces weak axis bending.

14) Though roof apparent stiffness (and therefore deflection) is affected by pitch, roof strength appears uniform for all pitches tested.

Even the best experimental program is of limited value if the results cannot be understood and applied. In the final portion of this dissertation, the results of the experimental program are evaluated with respect to the existing building codes and typical engineering practice. The following conclusions can be drawn based on this study and the testing of pitched wood roof diaphragms:

15) Eccentricity can be correlated with the overall shape factor, R. The assumption of a maximum increase in load for (wood frame single family dwelling) WFSFD shear wall design of 20% appears to be unconservative.

16) Most WFSFD should be designed using an envelope method due to a mix of diaphragm types and the effects of roof pitch and geometry.

17) The assumption that WFSFD have flexible diaphragms has some basis when only some orientations are considered, but when seismic loads in all orientations are examined, a blanket assumption of flexibility seems unwarranted from this study.

18) If more detailed methods are not used, it is advisable to evaluate WFSFD using an envelope method using both rigid and flexible diaphragm design.

19) Cases may occur where determination of semi-rigid or flexible diaphragm behavior is difficult because the code prescribed analysis is contradictory or fallacious. This suggests that use of semi-rigid finite element model (FEM) or a manual envelope method is prudent.

20) Designers should expect to consider the effects of horizontal irregularities during the design process. If we wish to improve the performance of WFSFD, it may be useful to begin eliminating some of the exceptions included in ASCE 7 and the IBC that permit design of WFSFD without consideration of torsional effects and re-entrant corner effects.

21) FP FEM makes it possible to apply a torsional load, even though this loading is not easily performed in the Tributary Area Method.

22) There is no difference in implementation of rigid, flexible or semi-rigid diaphragm plate models from an FEM standpoint. The only difference in the models is the required thickness of the plate, so analysis of diaphragms of any rigidity is practical if the thickness can be calibrated to the required stiffness.

23) The tributary area method will over-predict loading of interior walls if diaphragms are treated as simple span beams between parallel shear walls, and as a result, the loads on the exterior walls may be too low.

24) None of the methods examined predicted high displacements on the walls with the greatest damage in 1994 Northridge earthquake residences. It is not clear why this is the case, but at present, it does not appear that any of these methods is very accurate in predicting damage based on exceedance of a deflection standard.

25) Rigid plate (RP) and flexible plate (FP) FEM produce similar design loads because the FP FEM is not truly fully flexible.

26) Plate FEM methods are the only methods that seem to distribute loads in some proportion to the stiffest walls, therefore these methods appear to be the reasonable methods for performing diaphragm analysis.

27) RP and FP FEM both allow some load sharing between walls not inherent in either the manual rigid diaphragm or tributary area methods.

28) A semi-rigid plate FEM will produce lower design loads than the envelope method if knowledge of diaphragm stiffness is adequately reflected in the model,

29) The use of RP, FP or SP FEM methods can be simple and practical methods to add to the engineer's toolbox for analyzing WFSFD with a reasonable level of detail and effort.

Recommendations for Future Research

1) Further experiments investigating the effects of roof pitch and geometry are needed. There are a wide variety of such experiments that could be performed. For example, experiments of U or L shaped roofs with gable trusses may show that the right angle shape provides similar truss bracing that is provided by the hip roof configuration. Mansard and gambrel roofs are not common but have not been examined.

2) Present Code provisions for addressing diaphragm irregularities involve increasing the connection forces between diaphragms or around diaphragm irregularities. These connections are often sheet metal straps added to the roof sheathing surface. Additional experiments should be performed to determine the effectiveness of strap connections between diaphragms because the non-planar aspects of the connections have not been examined.

3) Examination and verification of the required truss bracing that may be needed to prevent the truss warping observed in the gable roof tests. Such bracing may allow gable roofs to regain much or all of the stiffness loss seen in this study.

4) Roof experiments in this study were also simple roofs without openings or changes in width that frequently occur. The effects of openings or width changes on the strength and stiffness of roof diaphragms needs better investigation.

5) All roof diaphragm experiments performed herein were monotonic loading. Experiments with dynamic loading cycles would be useful to determine whether the stiffness degrades due to cycling of loads on nails and connections.

6) Application of plate FEM to seismic damage reports was difficult and inconclusive in some aspects. Application of this method to concurrently tested structures on shake tables could provide better information on the important parameters relating to diaphragm deflection that eluded this study.

7) Application of the predominant present analysis methods (rigid diaphragm analysis and tributary area analysis) to concurrently tested structures is needed to assist in determining is more useful for design.

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Appendices

Appendix A

Test Setup

All roofs were tested at the Gene D. Knudson Wood Engineering Laboratory in Richardson Hall, the location of the Department of Wood Science and Engineering at Oregon State University. The laboratory has steel beams and columns pre-fabricated with end plates, beams and columns having standard hole sizes and patterns to allow easy assembly into a variety of configurations for any test requirement.

The specific test rig (see Figure A.1) was assembled from two, horizontal W310x97 (W12x65) beams to support the structure under the eaves and four vertical W310x97 (W12x65) columns at each corner. The columns had plates welded on each end with holes matching the standard gage pattern. Beams had holes along their length matching the standard gage pattern in the column base and top plates. Columns at each beam end were attached to embedded plates in the building concrete floor. Each column was attached to the floor or beam with (4) ASTM 325 bolts, 38 mm (1.5 in) diameter.

The experiments were designed to determine the stiffness and strength of various pitched wood roof diaphragms for lateral loading, based on a cantilevered beam model. In this experimental setup, the southwest corner is restrained in all three orthogonal axes for translation, but is free to rotate about any of those axes. The northwest corner is restrained against translation in the east-west direction, but free to rotate. Both of the leftmost ends of the roof are free to move or rotate under the loading but are restrained against uplift. The hydraulic cylinder shown on the southeast corner applies the test load.

Displacement sensors (LVDTs) were installed at the corners of the wood roof diaphragm, measuring the displacement of the truss corners as close as possible to the supports. (See Figure A.1.) At the southwest corner and the northwest corner, LVDTs measured displacement of that roof corner laterally and longitudinally. At the northeast corner, LVDTs measure displacement of that corner on all three orthogonal axes (X, Y and Z). At the hydraulic cylinder on the southeast corner, LVDTs measure horizontal movement laterally (X) and longitudinally (Y) while an additional load cell measures uplift forces (Z) at that corner. At the remaining two corners, the roof system is restrained in the vertical (Z) axis, but LVDTs measure movement in either horizontal axis (X and Y). Though theoretically, motion will not occur in some of these directions, sensors were installed to measure any movement that might occur due to non-ideal conditions.

Once the instrumentation system was assembled, each LVDT, string potentiometer or load cell was removed from the test rig for calibration. Known loads or displacements were imposed on each sensor, a series of data points were sampled, and linear regressions were performed to determine the appropriate scaling factors for each data channel in the installed instrumentation system.

During a test, the roof moves towards the hydraulic cylinder, providing a relatively constant rate of increasing deflection, while the load cells on the cylinder and at the southeast corner measure loads and LVDTs at each corner, measure the load and deflections of the roof structure.

Roofs were assembled nearby at floor level for ease of assembly, lifted into place on the test rig using a gantry crane and attached to the restraints at the southwest and northwest corners. The heels of the trusses rested on a pair of plastic plates that were lubricated with high-pressure grease to allow free movement of the eaves.

Analysis of Torsion in Trusses.

With trusses spaced at 610 mm (24 in.) O.C., there were 7 trusses over the length of the tested roof. For the flat roof, using 38x140 mm (nominal 2x6 in.) Douglas Fir-Larch (DF-L) lumber, #1 grade for diaphragm chords and joists and a 25 kN load which is the typical maximum elastic test load:

$$P = 25kN \tag{A.1}$$

$$n = 7 \tag{A.2}$$

$$E = 11.721GPa \tag{A.3}$$

$$L = 2.438m \quad \text{(A.4)}$$

$$R = \frac{P}{2} = 26.689kN \quad \text{(A.5)}$$

$$I = \frac{bd^3}{12}n = \frac{139.7mm(38.1mm)^3}{12}(7) = 4.507x10^6 mm^4 \quad \text{(A.6)}$$

Then assuming the truss is fixed at one end and supported but not fixed at the other:

$$\Delta = \frac{RL^3}{48\sqrt{5}EI} = 31.963mm \quad \text{(A.7)}$$

The same calculations can be made for the gable roofs, but the problem is more complex because the truss top chords have pitches that vary. This method analyzes pitched roofs based on their projected area, so as the pitch increases, the projected cross-sectional area of the top chord increases, resulting in a higher moment of inertia. This allows adjustment for that consideration, using trigonometry.

It is also necessary to adjust for aspect ratio, as shown in Eq. 26 of ASTM E455. In this case, a = 3557.6 mm and but to adjust b for two “half ” diaphragms that occur out-of-plane, so b = 4876.8 mm / 2, and a/b = 1.5.

For a 4:12 pitch, using 38x89 mm (nominal 2x4 in.) Douglas Fir-Larch (DF-L) lumber, #1 grade for the truss chords:

$$T = \arctan(\frac{4}{12}) = 18.43\deg \quad \text{(A.8)}$$

$$I = \frac{1}{\cos(\Theta)} \frac{bd^3}{12} n$$
$$= \frac{1}{0.9487} \frac{88.9mm(38.1mm)^3}{12}(7) = 3.023x10^6 mm^4 \quad (A.9)$$

$$\Delta = \frac{RL^3}{48\sqrt{5}EI} = 47.65mm \quad (A.10)$$

Similarly, for an 8:12 gable roof:

$$\Delta = \frac{RL^3}{48\sqrt{5}EI} = 41.79mm \quad (A.11)$$

and for a 12:12 gable roof:

$$\Delta = \frac{RL^3}{48\sqrt{5}EI} = 35.52mm \quad (A.12)$$

Figure 3.6 shows deflection perhaps exceeding 50 mm, so this calculation is at least in the correct order of magnitude.

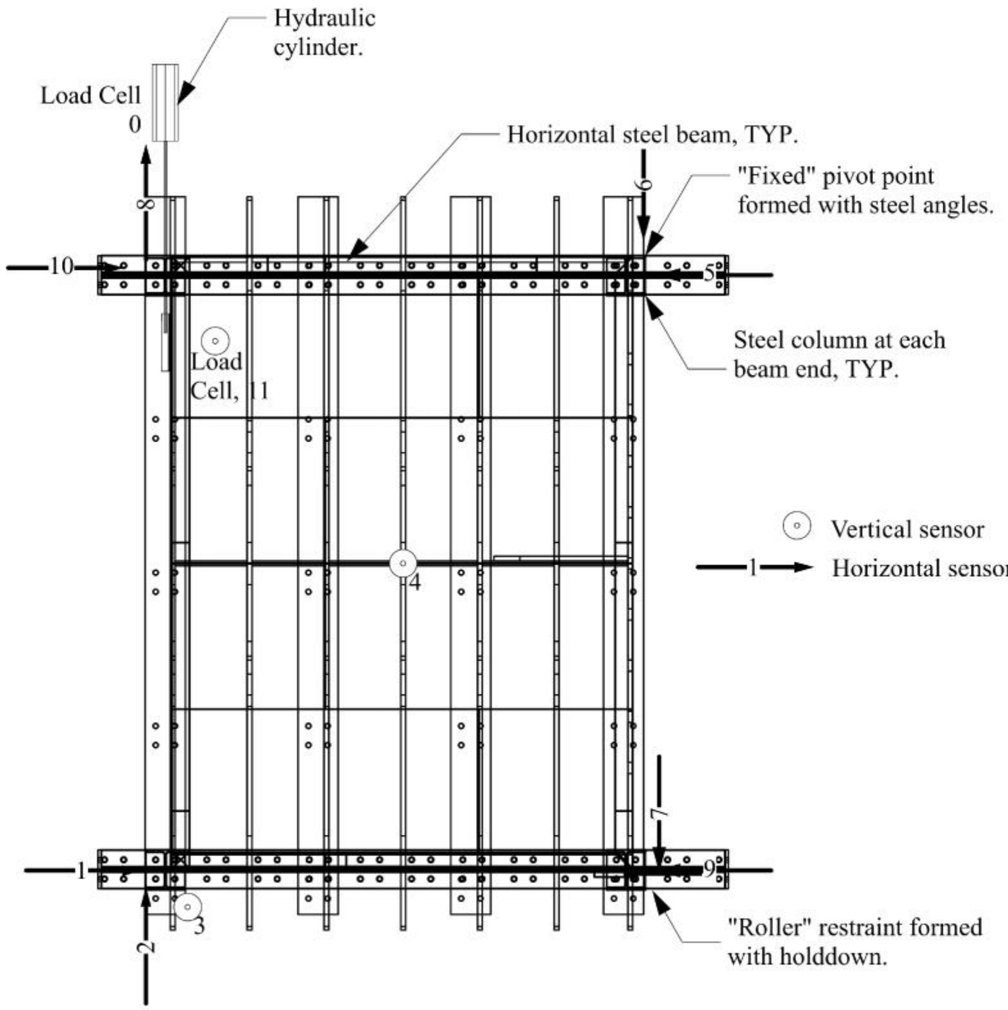


Figure A.1. Sensor Location and Data Channel Numbers.

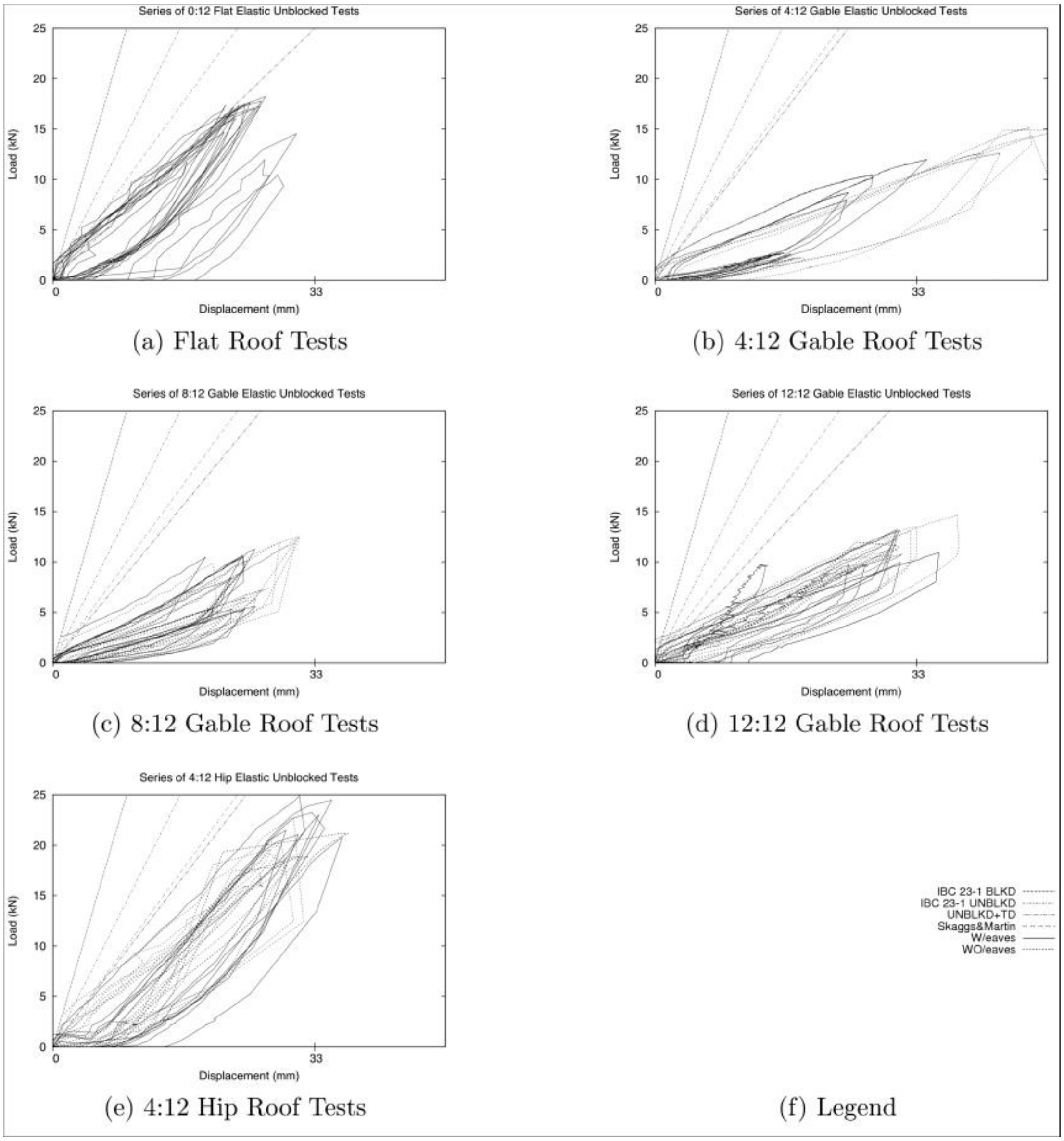


Figure A.2. Elastic Tests of Various Roof Configurations and Pitches Compared to IBC 23-1 Deflection Calculation Equation.

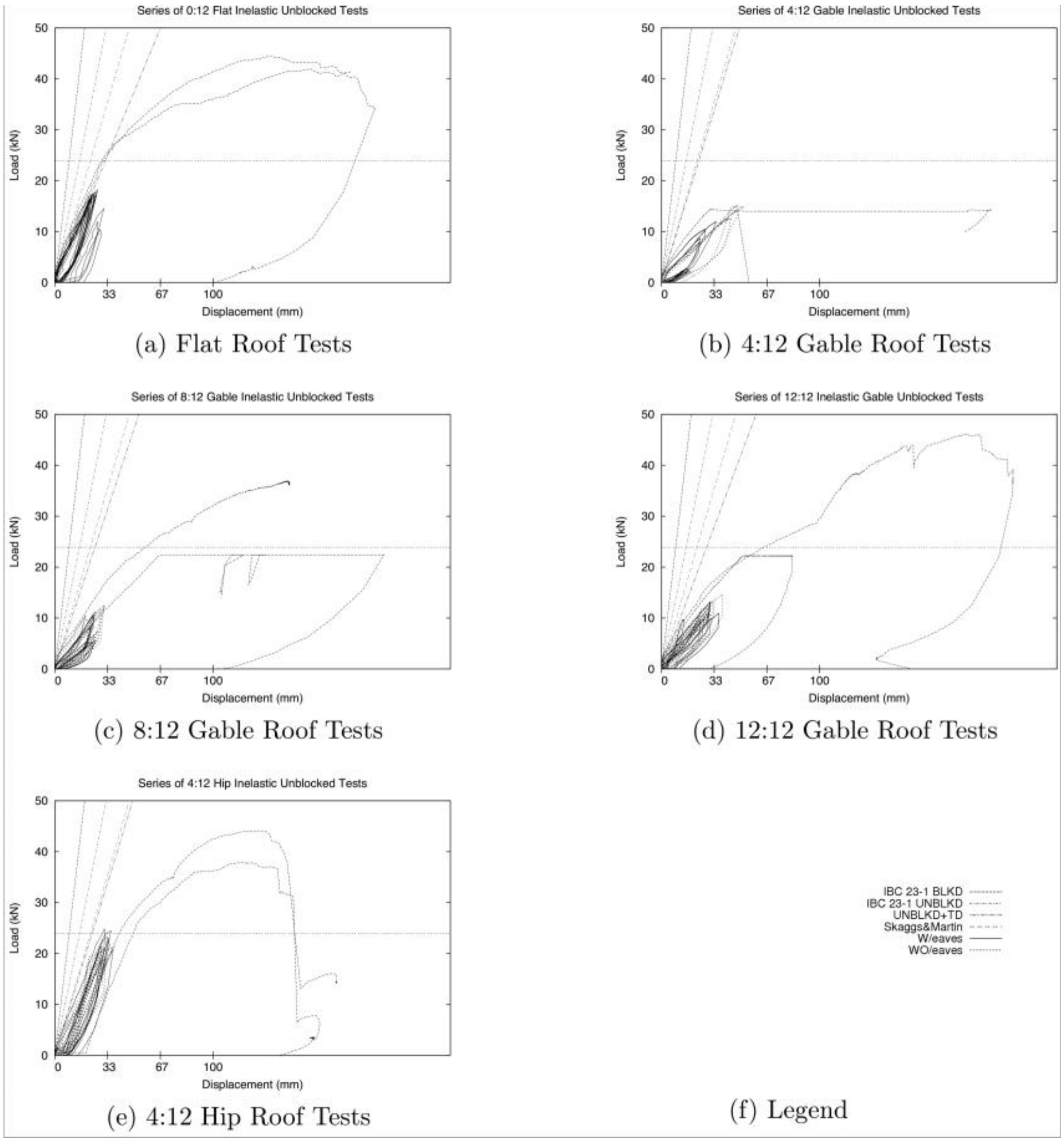


Figure A.3. Inelastic Tests of Various Roof Configurations and Pitchs Compared to IBC 23-1 Deflection Calculation Equation.

Table A.1. Experimental Configuration.

Configuration	Pitch	Loading
1. Flat roofs	0% (0:12) 38x140 mm joists (2x6 in, joists)	Static, elastic w/o eave plywood, Static inelastic w/gypsum ceiling.
2. Gable roofs.	a: 33% (4:12) b: 67% (8:12) c: 100% (12:12) 38x89 mm trusses (2x4 in. trusses)	Static, elastic w/eave plywood, Static, elastic w/o eave plywood, Static inelastic w/gypsum ceiling.
3. Hip roofs.	33% (4:12) 38x89 mm trusses (2x4 in. trusses)	Static, elastic w/eave plywood, Static, elastic w/o eave plywood, Static inelastic w/gypsum ceiling.

Note: Each test was repeated per ASTM E455 unless otherwise noted.

Table A.2. Experimental Schedule.

Date		Pitch	Type	Elastic/Inelastic	Eaves
Start	**Finish**				
2-Oct-09	13-Oct-09	12:12	Gable	E	Y
14-Oct-09	15-Oct-09	12:12	Gable	E	N
16-Oct-09	26-Oct-09	12:12	Gable	I	N
27-Oct-09	27-Oct-09	4:12	Gable	E	Y
28-Oct-09	29-Oct-09	4:12	Gable	E	N
30-Oct-09	4-Nov-09	4:12	Gable	I	N
5-Nov-09	4-Nov-09	8:12	Gable	E	Y
5-Nov-09	8-Nov-09	8:12	Gable	E	N
9-Nov-09	10-Nov-09	8:12	Gable	I	N
11-Nov-09	10-Nov-09	4:12	Hip	E	Y
11-Nov-09	11-Nov-09	4:12	Hip	E	N
12-Nov-09	13-Nov-09	4:12	Hip	I	N
22-Feb-10	21-Feb-10	4:12	Gable	E	Y
22-Feb-10	23-Feb-10	4:12	Gable	E	N
24-Feb-10	25-Feb-10	4:12	Gable	I	N
26-Feb-10	25-Feb-10	8:12	Gable	E	Y
26-Feb-10	1-Mar-10	8:12	Gable	E	N
2-Mar-10	9-Mar-10	8:12	Gable	I	N
10-Mar-10	9-Mar-10	12:12	Gable	E	Y
10-Mar-10	11-Mar-10	12:12	Gable	E	N
12-Mar-10	22-Mar-10	12:12	Gable	I	N
23-Mar-10	24-Mar-10	4:12	Hip	E	Y
25-Mar-10	29-Mar-10	4:12	Hip	E	N
30-Mar-10	4-Apr-10	4:12	Hip	I	N
5-Apr-10	5-Apr-10	0:12	Flat	E	N
6-Apr-10	7-Apr-10	0:12	Flat	I	N
8-Apr-10	8-Apr-10	0:12	Flat	E	N
9-Apr-10	10-Apr-10	0:12	Flat	I	N

Appendix B

Flexible vs. Rigid Diaphragm Analysis

A flexible diaphragm analysis assumes that a floor or roof structure has no in-plane stiffness. It distributes horizontal loads to the nearest vertical elements, which transmit loads to the foundation. Flexible diaphragms can be readily analyzed "by hand."

A rigid diaphragm analysis assumes the floor or roof is infinitely stiff and loads can be distributed to vertical elements based on their relative stiffnesses. Although rigid diaphragm analysis can be performed by hand, use of spreadsheets or other computer software helps organize the often tedious calculations.

The tributary area (flexible diaphragm) method determines shearwall loads by allocating half the load on each diaphragm/diaphragm element between the nearest shearwalls. In the example, load on half of the building width is attributed to the center wall. Thus, the rigid diaphragm analysis results in the same force in each wall (1/3 total) while the tributary area method results in ½ of the load on the center wall and ¼ on the exterior walls. Simplicity of the tributary area method makes it preferable for most engineers where its use is deemed appropriate.

Diaphragms idealized as flexible or rigid comprise the opposite extreme limits of design methods and force distributions. Where there is concern about the method to be used, some authors recommend performing both types of analyses and designing for the greatest force on each element. (SEAOC 2006; Breyer et al. 2007) This is often referred to as an "envelope method" because all possible design loads are considered and the structure is then designed for the extreme loadings within the envelope.

Additional Building Code Requirements

The 2003 IBC (ICC 2003) allowed flexible diaphragm analysis of wood structural panel diaphragms, but only under the seismic Simplified Analysis Procedure. The 2006 IBC (ICC 2006a) amended provisions of ASCE 7 (ASCE 2006) to permit flexible diaphragm design of wood structural panels for other occupancies under certain conditions (Ghosh and Dowty 2006). Skaggs and Martin (2004) questioned whether this exception is based on actual performance or a requirement due to inadequate design fees .

Although not stated in the design codes, several sources indicate that the deflection of an unblocked horizontal wood structural panel diaphragm is 2.5 times that of a blocked diaphragm of similar materials at the tabulated allowable load (Skaggs and Martin, 2004; Shea, 1999).

Horizontal Diaphragm Irregularities

A single story L shaped WFSFD can have any of three types of horizontal diaphragm irregularities as explained in ASCE 7 (ASCE 2006, 2010). A Torsional Irregularity is defined as a condition where story drift at one end of the structure exceeds 20% of the average drift of both extreme ends of the structure. An Extreme Torsional Irregularity is defined as a condition where the story drift at one end exceeds 40% of the average drift. A Re-entrant Corner Irregularity occurs when the cutout of the L exceeds 15% of the base dimension for both transverse and longitudinal legs of the L.

Effect of Pitch or Geometry on Horizontal Diaphragm Design

It has been typical in wood diaphragm design to treat gable or hip roof diaphragms with pitches as if the effect of projecting the roof on a horizontal plane resolves any strength or stiffness loses due to these factors. Examples showing this treatment appear in Breyer et al. (2007) and SEAOC (2006). The practice is so common that none of the cited examples state the assumption, but proceed instead to apply it without explanation or justification.

Rigid diaphragm analysis

The rigid diaphragm analysis determines center of mass (COM) and center of rigidity (COR), distributes base shear to each wall based on relative rigidity of the wall, and torsional loads from the diaphragm to each shearwall based on the eccentricity of the

COM relative to the COR (Figure 4.1.) COM is determined by calculating the first moment of the masses tributary to the horizontal diaphragm:

$$\bar{x}_m = \frac{\Sigma Mx}{M} \tag{B.1}$$

$$\bar{y}_m = \frac{\Sigma My}{M} \tag{B.2}$$

where M is the mass tributary to the diaphragm under investigation, Mx and My are the first moments of mass about the principal axes, and $\bar{x}_m$ and $\bar{y}_m$ are principal axis components of the COM.

For a single story WFSFD, half of the mass of the walls is included in the mass of the roof. For a two story WFSFD, the first level mass includes half of the mass of the walls above and below that diaphragm level and the roof level includes half the mass of the top floor walls and all of the roof mass.

COR is determined from the first moment of the rigidity of the elements that support a horizontal diaphragm:

$$\bar{x}_r = \frac{\Sigma R_y x}{\Sigma R_y} \tag{B.3}$$

$$\bar{y}_r = \frac{\Sigma R_x y}{\Sigma R_x} \tag{B.4}$$

where R_x and R_y are the relative rigidities in their respective axes, and x and y are the distances from the origin to each component on the same axes. Although wall mass is divided between floors, the relative shear stiffness is linear with height, so no height adjustment is required.

Polar moment of inertia (MOI), J_r, is calculated as the second moment of the rigidity of each element supporting the horizontal diaphragm:

$$J_r = \sum\left(R_y \bar{x}^2 + R_x \bar{y}^2\right) \tag{B.5}$$

and, finally, the shear wall forces are calculated by combining the direct shear and torsional shear (based on Schneider and Dickey 1994):

$$V = \frac{R_y}{\Sigma R_y} P_x + Max(R_y \bar{x} \frac{P_x e_x}{J_r}, R_x \bar{y} \frac{P_y e_y}{J_r}) \tag{B.6}$$

$$V = \frac{R_x}{\Sigma R_x} P_y + Max(R_y \bar{x} \frac{P_y e_y}{J_r}, R_x \bar{y} \frac{P_y e_y}{J_r}) \tag{B.7}$$

where e_x and e_y are the distances between the COR and COM on each axis, P_x and P_y are the seismic forces in each axis direction, V is the shear on the wall under design and $\bar{x}$

and $\bar{y}$ are the distances from the COR to the wall under design (Figure 4.1). This method is often performed in a spreadsheet to simplify redesigns.

Arrangement of Walls and Windows

The arrangement of walls, windows and doors in WFSFD follow trends or guidelines which are based on privacy, need for access from public routes, natural lighting and egress in emergencies. For example, WFSFD typically have most windows on the front and back walls, which do not face the closest neighbors, unless there is no other way to include a window (such as providing required ventilation to a bathroom with only one exterior wall on a side with limited setback). This avoids the possibility that a close neighbor might be able to look into the subject WFSFD windows from a short distance. WFSFD also typically have doors on the front (near the street) and back (near the patio). Fire exits are typically located on different sides of WFSFD, usually opposite sides. Area of windows in an exterior wall is usually above 8 percent of the room floor area where it is located and may exceed 12% of the floor area. (It is common to express the required window area as a percentage of the floor area in WFSFD code and design books. Refer to Section R303 in ICC (2006b), for example.)

Typical Room Sizes and WFSFD Dimensions

Sizes of rooms in WFSFD are limited by local regulations and minimum code requirements as well as availability of lumber and material sizes. Minimum size for most bedrooms is 3 x 3 m. For most WFSFD, two 3 m wide bedrooms will be accessed by a 1 m wide hallway, giving a minimum WFSFD width of 3 m + 3 m + 1 m = 7 m. A pair of lapped joists or a metal plate connected truss can be easily obtained to span these distances. More typical bedrooms are 3.6 or 4.2 m in their smaller plan dimension.

Large master bedrooms may have a smaller plan dimension of 4.2 m. If located across the hall from a small bedroom, the roof structure would span (4.2 m + 1 m + 3 m) = 8.2 m, approaching the limit of readily available dimension lumber for ceiling joists.

Modeling perforated shear wall behavior

Shear wall design is complicated by various openings that are necessary for WFSFD. As openings are introduced, wall strength and stiffness diminish, and these reductions must be evaluated and considered in the design of the WFSFD LFRS. Methods commonly used to design shear walls are:

Force transfer around opening (FTAO): In this method, the shear wall is broken up into rectangular elements around openings such as windows and doors. Rigorous mechanical analysis of each of these components is performed to determine shear and tension that must be transferred around the opening to ensure the wall operates as a single unit (Diekmann, 1989). Because this method is rigorous, the exact position of each opening and the height and length of each wall need to be known. Straps and additional connectors are used to transmit loads around the openings. Hold-downs and metal ties to rafters or trusses may also be required at each side of a window or door

Perforated shear wall: Originally proposed by Sugiyama and Matsumoto (1994), but also employed in Crandell et al. (1999), NAHB (2000), and Dolan and Heine (1997a,b), the perforated shear wall design method uses empirical formulae to adjust shear wall strength without detailed analysis of interconnections around openings. This method determines a capacity reduction factor to account for omission of straps and hold-downs except at the ends of the wall segment being designed. Hold-downs, straps and metal ties required for FTAO are often not needed, but sheathing strength and required nailing may need to be increased due to the reduction factor. Hold-downs may still be required at the extreme ends of each wall but not at each opening. Exact sizes and locations of openings are usually not critical in this method.

Segmented: Assumes that shear walls are effective only between openings, and the wall portions above and below openings are totally ineffective. This is the most conservative of these methods because it treats only full-height segments with no openings as effective. It does not require transfer of loads around openings. Exact positions of each opening and exact height and length of each wall need to be known for accuracy.

Conservatism may result in increases in required sheathing strength and nailing. Hold-downs are likely to be required at both ends of all segments and are likely to need higher capacities than for other methods.

The perforated shear wall method of Sugiyama and Matsumoto (1994) as shown in Crandell et al. (1999), uses the following factor to adjust shearwall capacity:

Opening Adjustment Factor (Cop) $\quad Cop = r / (3 - 2*r) \quad$ (B.8)

where

$r = 1 / (1 + \alpha/\beta)$, the sheathing area ratio,

$\alpha = \Sigma Ao / (H \times L)$, the ratio of the openings area to the total wall area,

$\beta = \Sigma Li / L$, the ratio of the length of full height sheathing to the total length of the wall.

A different implementation of this method was developed and included in Table 4.3.3.4 of the Special Design Provisions for Wind and Seismic (SDPWS) (AF&PA 2006), the IBC (ICC 2003) and Breyer et al. (2007). The method has a similar capacity reduction, but based on a design table of maximum opening sizes and percentages of full-height sheathing in a wall to determine the effective shear capacity ratio. The deflection of a perforated shear wall modeled as a linear elastic element is also adjusted by the same factor as the shear wall strength (APA 2007).

Deflection of a shearwall is calculated with the following equation (Breyer et al. 2007):

$$\Delta = \Delta_b + \Delta_v + \Delta_n + \Delta_a \qquad \text{(B.9)}$$

Overall shearwall deflection is the sum of bending deflection Δ_b, shear deflection Δ_v, effect of nail slip Δ_n and deflection due to hold-down slack Δ_a, adjusted for a perforated shearwall Δ_s (ICC 2006):

$$\Delta_s = \frac{\Delta}{Cop} \qquad \text{(B.10)}$$

Horizontal diaphragm deflection

Horizontal diaphragm deflection is similarly calculated as (Breyer et al. 2007):

$$\Delta = \Delta_b + \Delta_v + \Delta_n + \Delta_c \qquad \text{(B.11)}$$

In this case, the diaphragm deflection is a sum of bending deflection Δ_b, shear deflection Δ_v, effect of nail slip Δ_n and deflection due to slack in the diaphragm chords Δ_c.

Shear wall deflection

Although the research cited above primarily discusses effects of openings on shear wall strength, the perforated shear wall method is a linear-elastic method. The IBC (ICC 2006a) indicates the opening adjustment factor can therefore be applied to wall deflections as well:

> ***2305.3.8.2.9 Deflection of shear walls with openings.*** *The controlling deflection of a blocked shear wall with openings uniformly fastened throughout shall be taken as the maximum individual deflection of the shear wall segments calculated in accordance with Section 2305.3.2, divided by the appropriate shear resistance adjustment factors of Table 2305.3.8.2.(ICC 2006a).*

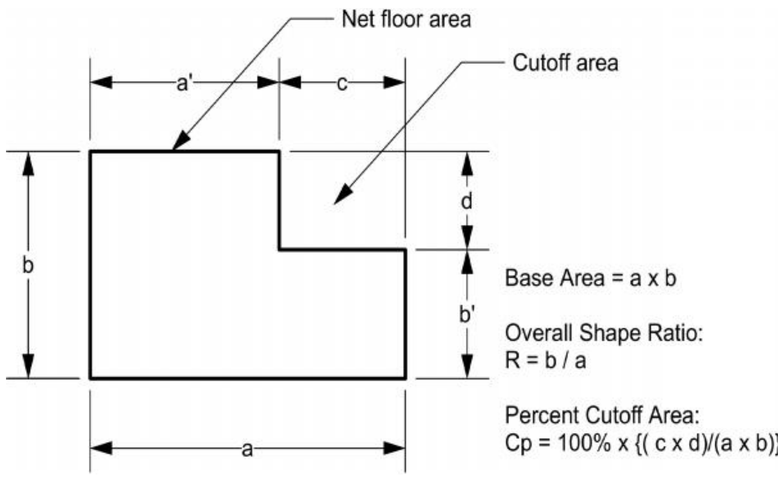
Thus, deflection of a perforated shear wall modeled as a linear elastic element is also adjusted by the same factor as the shear wall strength (APA 2007).

Tables for Analytical Models

Tables B.1 through B7 provide background support for the figures shown in the main paper. The name of each analytical model is coded from the R and Cp ratios, thus P82_15 indicates a WFSFD with R=0.82 and Cp = 15%.

Diaphragm Flexibility Calculations for Paevere House.

PAEVERE ET AL. (2003) HOUSE



CASE 1 – Windows evenly distributed to walls.

Given dimensions:

a = 36.652 ft
b = 30.090 ft
c = 16.5 ft
d = 6.583 ft

Wall height	ht = 8 ft
Wall thickness	th = 6 in
Door height	dht = 7 ft
Door width	dw = 3 ft
Door area	da = dht * dw = **21.000** ft^2

R = b / a = **0.821**
Cp = 100% * (c * d)/(a * b) = **9.849**%

a‘ = a – c = **20.152** ft
b‘ = b – d = **23.507** ft

TA = a * b – c * d = **994.239** ft^2

P = 1.9779psf*TA = **1966.506** lbf

RIGID DIAPHRAGM ANALYSIS:

Locate the center of mass:

Use the lower left corner as the origin. Center of mass is calculated using the upper ½ of the walls, disregarding window and openings. Mass of floor is considered equivalent in mass to the walls on a wall/floor area basis.

Center of mass, x coordinate:

Element	Element area	X Offset from Origin	First moment of area
a	A1=a*ht/2=**146.608** ft^2	O1=a/2=**18.326** ft	M1=A1*O1=**2686.738** ft^3
b	A2=b*ht/2=**120.360** ft^2	O2=th/2=**0.250** ft	M2=A2*O2=**30.090** ft^3
c	A3=c*ht/2=**66.000** ft^2	O3=a-c+c/2=**28.402** ft	M3=A3*O3=**1874.532** ft^3
d	A4=d*ht/2=**26.332** ft^2	O4=a'-th/2=**19.902** ft	M4=A4*O4=**524.059** ft^3
a'	A5=a'*ht/2=**80.608** ft^2	O5=a'/2=**10.076** ft	M5=A5*O5=**812.206** ft^3
b'	A6=b'*ht/2=**94.028** ft^2	O6=a-th/2=**36.402** ft	M6=A6*O6=**3422.807** ft^3
D1	AD1=a*b'=**861.579** ft^2	O7=a/2=**18.326** ft	MD1=AD1*O7=**15789.289** ft^3
D2	AD2=a'*d=**132.661** ft^2	O8=a'/2=**10.076** ft	MD2=AD2*O8=**1336.688** ft^3
	AT=A1+A2+A3+A4+A5+A6 +AD1+AD2= **1528.175** ft^2		MT=M1+M2+M3+M4+M5+M6+ MD1 +MD2=**26476.410** ft^3

xm = MT/AT = **5.281** m xm = **17.326** ft

Center of mass, y coordinate:

Wall	Wall area	Y Offset from Origin	First moment of area
a	A1=a*ht/2=**146.608** ft^2	O1=th/2=**0.250** ft	M1=A1*O1=**36.652** ft^3
b	A2=b*ht/2=**120.360** ft^2	O2=b/2=**15.045** ft	M2=A2*O2=**1810.816** ft^3
c	A3=c*ht/2=**66.000** ft^2	O3=b'-th/2=**23.257** ft	M3=A3*O3=**1534.962** ft^3
d	A4=d*ht/2=**26.332** ft^2	O4=b'+d/2=**26.799** ft	M4=A4*O4=**705.658** ft^3
a'	A5=a'*ht/2=**80.608** ft^2	O5=b-th/2=**29.840** ft	M5=A5*O5=**2405.343** ft^3
b'	A6=b'*ht/2=**94.028** ft^2	O6=b'/2=**11.754** ft	M6=A6*O6=**1105.158** ft^3
D1	AD1=a*b'=**861.579** ft^2	O7=b'/2=**11.754** ft	MD1=AD1*O7=**10126.564** ft^3
D2	AD2=a'*d=**132.661** ft^2	O8=b'+d/2=**26.799** ft	MD2=AD2*O8=**3555.106** ft^3
	AT=A1+A2+A3+A4+A5+A6+ AD1+AD2= **1528.175** ft^2		MT=M1+M2+M3+M4+M5+M6+ MD1 +MD2=**21280.258** ft^3

ym = MT/AT = **4.244** m ym = **13.925** ft

<u>Locate the center of stiffness:</u>

Perforated shear wall design:

1) Assume window area is 10% of floor area.
2) Assume 3 ft. window ht.
3) Wall a has one door and wall a' (opposite side) has one door. Essentially one door on each longest side. A door is 3 ft. wide by 7 ft. tall. Doors are not windows so they are in addition to window in opening area.
4) Windows are evenly distributed around the structure based on wall length.

Window area percentage WAP = 10%

Window area WA = WAP * (AD1 +AD2) = **99.424** ft^2

Perforated shear wall adjustment factors:

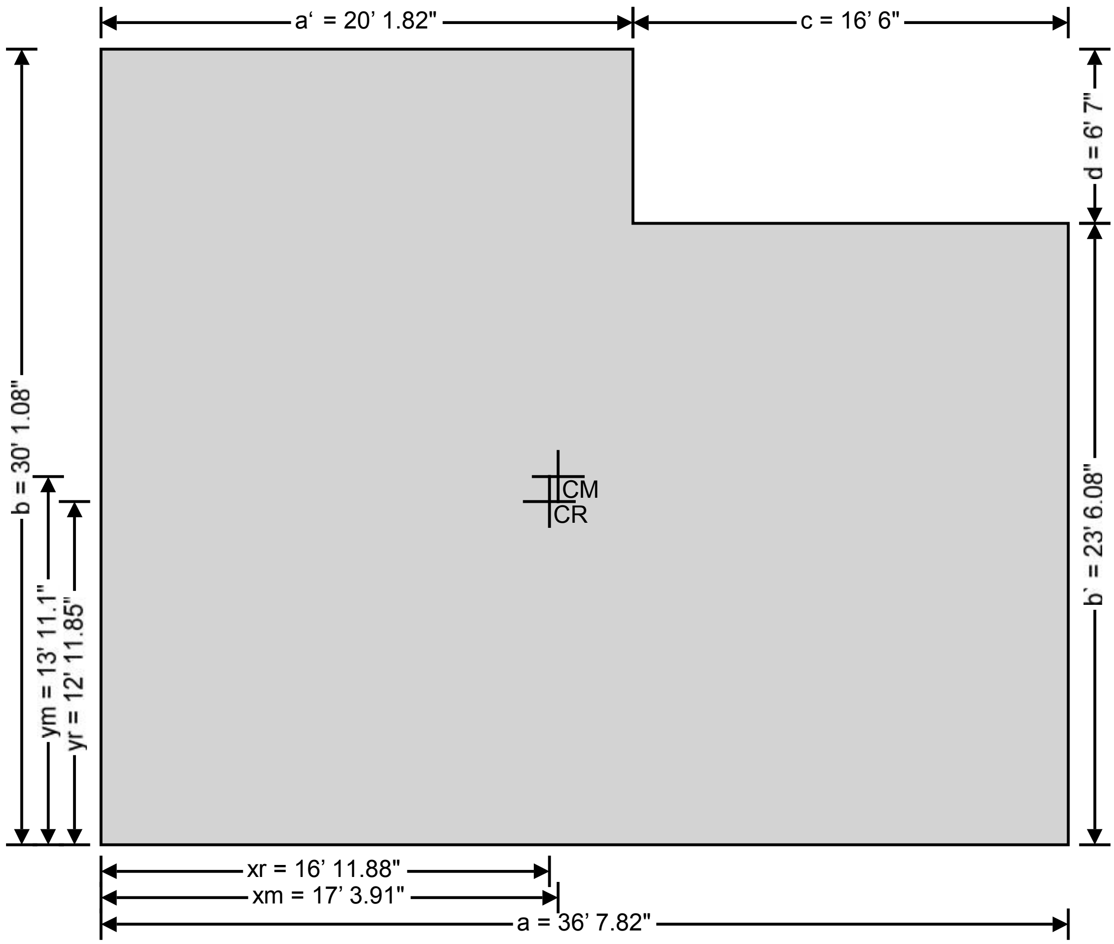
Wall	Wall area ratio, α	Wall length ratio, β	Sheathing area ratio, r	Opening adj. factor, COP
a	A1=(a/(2*a+2*b)*WA+da)/(a*ht)=**0.165**	B1=(a-a/(2*a+2*b)*WA/3ft-dw)/a=**0.670**	R1=1/(1+A1/B1)= **0.803**	C1=R1/(3-2*R1)=**0.575**
b	A2=(b/(2*a+2*b)*WA)/(b*ht)=**0.093**	B2=(b-b/(2*a+2*b)*WA/3ft)/b=**0.752**	R2=1/(1+A2/B2)= **0.890**	C2=R2/(3-2*R2)=**0.729**
c	A3=(c/(2*a+2*b)*WA)/(c*ht)=**0.093**	B3=(c-c/(2*a+2*b)*WA/3ft)/c=**0.752**	R3=1/(1+A3/B3)= **0.890**	C3=R3/(3-2*R3)=**0.729**
d	A4=(d/(2*a+2*b)*WA)/(d*ht)=**0.093**	B4=(d-d/(2*a+2*b)*WA/3ft)/d=**0.752**	R4=1/(1+A4/B4)= **0.890**	C4=R4/(3-2*R4)=**0.729**
a'	A5=(a'/(2*a+2*b)*WA+da)/(a'*ht)=**0.223**	B5=(a'-a'/(2*a+2*b)*WA/3ft-dw)/a'=**0.603**	R5=1/(1+A5/B5)= **0.730**	C5=R5/(3-2*R5)=**0.474**
b'	A6=(b'/(2*a+2*b)*WA)/(b'*ht)=**0.093**	B6=(b'-b'/(2*a+2*b)*WA/3ft)/b'=**0.752**	R6=1/(1+A6/B6)= **0.890**	C6=R6/(3-2*R6)=**0.729**

Center of rigidity

Wall	**Relative stiffness, Sx**	**Y Offset from Origin**	**Relative stiffness, Sy**	**X Offset from Origin**	**First moment of stiffness**	**First moment of stiffness**
a	S1=a* C1=**21.092** ft	O1=th/2=**0.250** ft			Sy1=S1*O1= **5.273** ft^2	
b			S2=b* C2=**21.938** ft	O2=th/2=**0.250** ft		Sx2=S2*O2=**5.485** ft^2
c	S3=c* C3=**12.030** ft	O3=b'-th/2=**23.257** ft			Sy3=S3*O3= **279.783** ft^2	
d			S4=d*C4=**4.800** ft	O4=a'-th/2=**19.902** ft		Sx4=S4*O4=**95.522** ft^2
a'	S5=a'*C5=**9.544** ft	O5=b-th/2=**29.840** ft			Sy5=S5*O5= **284.785** ft^2	
b'			S6=b'*C6=**17.139** ft	O6=a-th/2=**36.402** ft		Sx6=S6*O6=**623.887** ft^2
				S_yT=Sy1+Sy3+Sy5=**569.841** ft^2		S_xT=Sx2+Sx4+Sx6=**724.893** ft^2

xr = S_xT / S_x = **16.990** ft
yr = S_yT / S_y = **12.987** ft

ex = xr – xm = **-0.335** ft
ey = yr – ym = **-0.938** ft


a' = 20' 1.82"
c = 16' 6"
d = 6' 7"
b = 30' 1.08"
ym = 13' 11.1"
yr = 12' 11.85"
CM
CR
b' = 23' 6.08"
xr = 16' 11.88"
xm = 17' 3.91"
a = 36' 7.82"

Determine torsional MOI:

Wall	**Xbar**	**Ybar**	**2nd moment of stiffness, $Rxbar^2$**	**2nd moment of stiffness, $Rybar^2$**
a		Yb1=yr-0.25ft=**12.737** ft		Rybar2_1=S1*$Yb1^2$=**3422** ft^3
b	Xb2=xr-0.25ft=**16.740** ft		Rxbar2_2=S2*$Xb2^2$= **6148** ft^3	
c		Yb3=yr-(b'-0.25ft)=**-10.270** ft		Rybar2_3=S3*$Yb3^2$=**1269** ft^3
d	Xb4=xr-(a'-0.25ft)=**-2.912** ft		Rxbar2_4=S4*$Xb4^2$=**41** ft^3	
a'		Yb5=yr-(b-0.25ft)=**-16.853** ft		Rybar2_5=S5*$Yb5^2$=**2711** ft^3
b'	Xb6=xr-(a-0.25ft)=**-19.412** ft		Rxbar2_6=S6*$Xb6^2$=**6458** ft^3	

J_r = Rybar2_1+Rxbar2_2+Rybar2_3+Rxbar2_4+Rybar2_5+Rxbar2_6 = **20048.066** ft^3

Determine Torsional Shear

Wall	Ry	Xbar	RyXbar	Rx	Ybar	RxYbar	Seismic along y axis	Seismic along x axis
a				S1=a* C1=**21.092** ft	Yb1=yr-0.25ft= **12.737** ft	RxY1=S1*Yb1= **268.656** ft^2	Sy1=RxY1*P*ex/J_r= **-8.841** lbf	Sx1=RxY1*P*ey/J_r= **-24.718** lbf
b	S2=b* C2= **21.938** ft	Xb2=xr-0.25ft= **16.740** ft	RyX2=S2*Xb2= **367.249** ft^2				Sy2=RyX2*P*ex/J_r= **-12.086** lbf	Sx2=RyX2*P*ey/J_r= **-33.789** lbf
c				S3=c* C3=**12.030** ft	Yb3=yr-(b‘-0.25ft)= **-10.270** ft	RxY3=S3*Yb3= **-123.545** ft^2	Sy3=RxY3*P*ex/J_r= **4.066** lbf	Sx3=RxY3*P*ey/J_r= **11.367** lbf
d	S4=d*C4= **4.800** ft	Xb4=xr-(a‘-0.25ft)= **-2.912** ft	RyX4=S4*Xb4= **-13.976** ft^2				Sy4=RyX4*P*ex/J_r= **0.460** lbf	Sx4=RyX4*P*ey/J_r= **1.286** lbf
a’				S5=a‘*C5=**9.544** ft	Yb5=yr-(b-0.25ft)= **-16.853** ft	RxY5=S5*Yb5= **-160.838** ft^2	Sy5=RxY5*P*ex/J_r= **5.293** lbf	Sx5=RxY5*P*ey/J_r= **14.798** lbf
b’	S6=b‘*C6= **17.139** ft	Xb6=xr-(a-0.25ft)= **-19.412** ft	RyX6=S6*Xb6= **-332.698** ft^2				Sy6=RyX6*P*ex/J_r= **10.949** lbf	Sx6=RyX6*P*ey/J_r= **30.611** lbf

Total Shear Due to X-axis Ground Motion:

Wall	Direct Shear	Torsional Shear	Total Shear
a	D1=S1/ S_x*P=**972.151** lbf	Sx1= **-24.718** lbf	Vx1=D1+abs(Sx1)= **996.870** lbf
b	D2= 0 lbf	Sx2= **-33.789** lbf	Vx2=D2+abs(Sx2)= **33.789** lbf
c	D3=S3/ S_x*P=**554.475** lbf	Sx3= **11.367** lbf	Vx3=D3+abs(Sx3)= **565.842** lbf
d	D4= 0 lbf	Sx4= **1.286** lbf	Vx4=D4+abs(Sx4)= **1.286** lbf
a'	D5=S5/ S_x*P=**439.879** lbf	Sx5= **14.798** lbf	Vx5=D5+abs(Sx5)= **454.678** lbf
b'	D6= 0 lbf	Sx6= **30.611** lbf	Vx6=D6+abs(Sx6)= **30.611** lbf

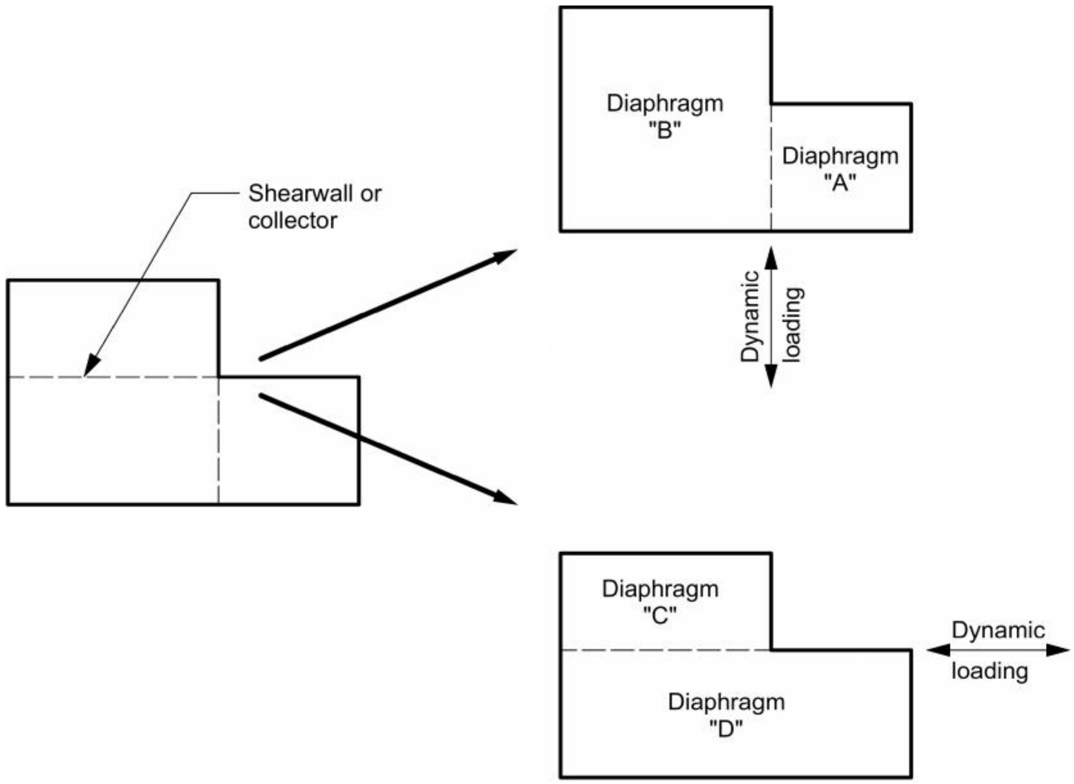
Total Shear Due to Y-axis Ground Motion:

Wall	Direct Shear	Torsional Shear	Total Shear
a	D1= 0 lbf	Sy1= **-8.841** lbf	Vy1=D1+abs(Sy1)= **8.841** lbf
b	D2=S2/ S_y*P=**983.253** lbf	Sy2= **-12.086** lbf	Vy2=D2+abs(Sy2)= **995.339** lbf
c	D3= 0 lbf	Sy3= **4.066** lbf	Vy3=D3+abs(Sy3)= **4.066** lbf
d	D4=S4/ S_y*P=**215.113** lbf	Sy4= **0.460** lbf	Vy4=D4+abs(Sy4)= **215.573** lbf
a'	D5= 0 lbf	Sy5= **5.293** lbf	Vy5=D5+abs(Sy5)= **5.293** lbf
b'	D6=S6/ S_y*P=**768.140** lbf	Sy6= **10.949** lbf	Vy6=D6+abs(Sy6)= **779.089** lbf

Design Summary:

Wall	Total Design Shear
a	V1=max(Vx1,Vy1)= **996.870** lbf
b	V2=max(Vx2,Vy2)= **995.339** lbf
c	V3=max(Vx3,Vy3)= **565.842** lbf
d	V4=max(Vx4,Vy4)= **215.573** lbf
a'	V5=max(Vx5,Vy5)= **454.678** lbf
b'	V6=max(Vx6,Vy6)= **779.089** lbf

So now we are ready to determine diaphragm rigidity!



Assumptions:

1) 2 No. 2, DF-L 2x6 studs are used as chords for shearwalls.

   A = (1.5in*5.5in)*2 = **16.500** in^2

   E = 1600000psi

2) Wall sheathing is 7/16" CDX w/8d nails at 6" O.C.

   Ga=11.0 kips/in

3) Hold down slip is assumed to be 1/8 inch

   d_a = 0.125in

4) Roof sheathing is 7/16" CDX w/8d nails at 6" O.C.
   a. Case 1 - Ga_d is 6.0 kips/in
   b. Case 2-6 - Ga_d is 4.0 kips/in

Y-Axis Seismic

$h = ht$

Diaphragm A

Deflection of wall b`:

$v = max(Vy6,Vx6) / b` = $ **33.143** plf

$v_n = v/(2ft^{-1}) = $ **16.571** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018} = $ **0.000** lbf

; $(8 * v * h^3) / (E * A * b`)/12 = $ **0.000** in

; $(v * h) / (Ga) = $ **0.024** in

; $0.75 * h * (e_n/1lbf)/12 = $ **0.000** in

; $h / b` * d_a = $ **0.043** in

$\Delta_{b`} = (8 * v * h^3 ) / (E * A * b`)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / b` * d_a = $ **0.067** in

$\Delta_{b`\_Cop} = \Delta_{b`} / C6 = $ **0.092** in

Deflection of wall d:

$v = max(Vy4,Vx4) / d = $ **32.747** plf

$v_n = v/(2ft^{-1}) = $ **16.373** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018} = $ **0.000** lbf

; $(8 * v * h^3) / (E * A * d)/12 = $ **0.001** in

; $(v * h) / (Ga) = $ **0.024** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.000** in

; $h / d * d_a$ = **0.152** in

$\Delta_d = (8 * v * h^3) / (E * A * d)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / d * d_a$ = **0.177** in

$\Delta_{d\_Cop} = \Delta_d / C4$ = **0.242** in

$\Delta_S = (\Delta_{b`\_Cop} + \Delta_{d\_Cop}) / 2$ = **0.167** in

Deflection of diaphragm a

L = c

Ga_d = if(L>=b`,6kips/in,4kips/in) = **4.000** kips/in

v = (max(Vy6,Vx6) + max(Vy4,Vx4)/2) / L = **53.750** plf

$v_n = v/(2ft^{-1})$ = **26.875** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf

n = Round((L/20ft-1)/2+0.49,0) = **0.000**

$\Delta_c X$ = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.000** ft^2

; $(5 * v * L^3) / (8 * E * A * b`) * 0.0246$ = **0.000** in

; (v * L) / (4 * Ga_{d}) = **0.055** in

; $0.188 * L * (e_n/1lbf)*0.0832$ = **0.000** in

; $\Delta_c X /(2*b`)$ = **0.000** in

$\Delta_a = (5 * v * L^3) / (8 * E * A * b`) * 0.0246$ + (v * L) / (4 * Ga_{d} ) + $0.188 * L * (e_n/1lbf)*0.0832 + \Delta_c X /(2*b`)$ = **0.056** in

Times 2.5 for an unblocked diaphragm...$\Delta_{a\_unblocked} = \Delta_a * 2.5$ = **0.139** in

DAr = if($\Delta_{a\_unblocked}$ > 2*Δ_S,"Flexible", "Semi-rigid") = **"Semi-rigid"**

Deflection of wall b:

v = max(Vy2,Vx2) / b = **33.079** plf

v_n = v/(2ft^{-1}) = **16.539** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.000** lbf

; (8 * v * h^3) / (E * A * b)/12 = **0.000** in

; (v * h) / (Ga) = **0.024** in

; 0.75 * h * (e_n/1lbf)/12 = **0.000** in

; h / b * d_a = **0.033** in

Δ_b = (8 * v * h^3) / (E * A * b) /12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / b * d_a = **0.058** in

Δ_{b_Cop} = Δ_b / C2 = **0.079** in

Δ_S = (Δ_{b_Cop} + Δ_{d_Cop}) / 2 = **0.161** in

Diaphragm b

L = a`

Ga_d = if(L>=b,6kips/in,4kips/in) = **4.000** kips/in

v = (max(Vy2,Vx2) + max(Vy4,Vx4)/2) / L = **54.740** plf

v_n = v/(2ft^{-1}) = **27.370** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.000** lbf

n = Round((L/20ft-1)/2+0.49,0) = **0.000**

Δ_cX = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.000** ft^2

; (5 * v * L^3) / (8 * E * A * b) * 0.0246 = **0.000** in

; $(v * L) / (4 * Ga\_\{d\})$ = **0.069** in

; $0.188 * L * (e_n/1lbf)*0.0832$ = **0.000** in

; $\Delta_c X /(2*b)$ = **0.000** in

$\Delta_b = (5 * v * L^3) / (8 * E * A * b) * 0.0246 + (v * L) / (4 * Ga\_\{d\}) + 0.188 * L * (e_n/1lbf)*0.0832 + \Delta_c X /(2*b)$ = **0.069** in

Times 2.5 for an unblocked diaphragm...$\Delta_{b\_unblocked} = \Delta_b * 2.5$ = **0.174** in

DBr =if($\Delta_{b\_unblocked} > 2*\Delta_S$,"Flexible", "Semi-rigid") = **"Semi-rigid"**

X-Axis Seismic.

Diaphragm C

Deflection of wall a`:

v = max(Vy5,Vx5) / a` = **22.562** plf

$v_n = v/(2ft^{-1})$ = **11.281** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf

; $(8 * v * h^3) / (E * A * a`)/12$ = **0.000** in

; $(v * h) / (Ga)$ = **0.016** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.000** in

; $h / a` * d_a$ = **0.050** in

$\Delta_{a`} = (8 * v * h^3) / (E * A * a`)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / a` * d_a$ = **0.066** in

$\Delta_{a`\_Cop} = \Delta_{a`} / C5$ = **0.140** in

Deflection of wall c:

$v = max(Vy3,Vx3) / c =$ **34.293** plf

$v_n = v/(2ft^{-1}) =$ **17.147** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018} =$ **0.000** lbf

$(8 * v * h^3) / (E * A * c)/12 =$ **0.000** in

; $(v * h) / (Ga) =$ **0.025** in

; $0.75 * h * (e_n/1lbf)/12 =$ **0.000** in

; $h / c * d_a =$ **0.061** in

$\Delta_c = (8 * v * h^3) / (E * A * c)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / c * d_a =$ **0.086** in

$\Delta_{c\_Cop} = \Delta_c / C3 =$ **0.118** in

$\Delta_S = (\Delta_{a`\_Cop} + \Delta_{c\_Cop}) / 2 =$ **0.129** in

Deflection of diaphragm c

$L = d$

$Ga_d = if(L>=a`,6kips/in,4kips/in) =$ **4.000** kips/in

$v = (max(Vy5,Vx5) + max(Vy3,Vx3)/2) / L =$ **112.046** plf

$v_n = v/(2ft^{-1}) =$ **56.023** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018} =$ **0.001** lbf

$n = Round((L/20ft-1)/2+0.49,0) =$ **0.000**

$\Delta_c X$ = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.000** ft^2

; (5 * v * L^3) / (8 * E * A * a`) * 0.0246 = **0.000** in

; (v * L) / (4 * Ga_{d}) = **0.046** in

; 0.188 * L * (e_n/1lbf)*0.0832 = **0.001** in

; $\Delta_c X$ /(2*a`) = **0.000** in

Δ_c = (5 * v * L^3) / (8 * E * A * a`) * 0.0246 + (v * L) / (4 * Ga_{d} ) + 0.188 * L * ($e_n$/1lbf)*0.0832 + $\Delta_c X$ /(2*a`) = **0.047** in

Times 2.5 for an unblocked diaphragm...$\Delta_{c\_unblocked}$ = Δ_c * 2.5 = **0.118** in

DCr =if($\Delta_{c\_unblocked}$ > 2*Δ_S,"Flexible", "Semi-rigid") = **"Semi-rigid"**

Deflection of wall a:

v = max(Vy1,Vx1) / a = **27.198** plf

v_n = v/(2ft^{-1}) = **13.599** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.000** lbf

; (8 * v * h^3) / (E * A * a)/12 = **0.000** in

; (v * h) / (Ga) = **0.020** in

; 0.75 * h * (e_n/1lbf)/12 = **0.000** in

; h / a * d_a = **0.027** in

Δ_a = (8 * v * h^3) / (E * A * a) /12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / a * d_a = **0.047** in

Δ_{a_Cop} = Δ_a / C1 = **0.082** in

Δ_S = (Δ_{a_Cop} + Δ_{c_Cop}) / 2 = **0.100** in

Deflection of diaphragm d

L = b`

Ga_d = if(L>=a,6kips/in,4kips/in) = **4.000** kips/in

v = (max(Vy3,Vx3)/2 + max(Vy1,Vx1)) / L = **54.443** plf

$v_n = v/(2ft^{-1})$ = **27.221** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf

n = Round((L/20ft-1)/2+0.49,0) = **1.000**

$\Delta_c X$ = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.208** ft^2

; $(5 * v * L^3) / (8 * E * A * a) * 0.0246$ = **0.000** in

; (v * L) / (4 * Ga_{d}) = **0.080** in

; $0.188 * L * (e_n/1lbf)*0.0832$ = **0.000** in

; $\Delta_c X /(2*a)$ = **0.034** in

$\Delta_d = (5 * v * L^3) / (8 * E * A * a) * 0.0246$ + (v * L) / (4 * Ga_{d}) + $0.188 * L * (e_n/1lbf)*0.0832 + \Delta_c X /(2*a)$ = **0.115** in

Times 2.5 for an unblocked diaphragm...$\Delta_{d\_unblocked} = \Delta_d * 2.5$ = **0.287** in

DDr =if($\Delta_{d\_unblocked} > 2*\Delta_S$,"Flexible", "Semi-rigid") = **"Flexible"**

FLEXIBLE DIAPHRAGM ANALYSIS:

Y-Axis Seismic

h = ht

Diaphragm A

Deflection of wall b`:

Calculate uniform load from diaph. depth — V = c / 2 * b‘ / TA * P = **383.580** lbf

v = V / b‘ = **16.318** plf

Nail load at 2/ft — $v_n = v/(2ft^{-1})$ = **8.159** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf

; $(8 * v * h^3) / (E * A * b`)/12$ = **0.000** in

; (v * h) / (Ga) = **0.012** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.000** in

; $h / b` * d_a$ = **0.043** in

$\Delta_{b`} = (8 * v * h^3) / (E * A * b`)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / b` * d_a$ = **0.055** in

$\Delta_{b`\_Cop} = \Delta_{b`} / C6$ = **0.075** in

Deflection of wall d:

Calculate uniform load from diaph. depth — V = (a‘*b + c*b‘) / 2 /TA * P = **983.253** lbf

v = V / d = **149.362** plf

Nail load at 2/ft — $v_n = v/(2ft^{-1})$ = **74.681** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.002** lbf

; $(8 * v * h^3) / (E * A * d)/12$ = **0.004** in

; (v * h) / (Ga) = **0.109** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.012** in

; $h / d * d_a$ = **0.152** in

Δ_d = (8 * v * h^3) / (E * A * d)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / d * d_a = **0.276** in

Δ_{d_Cop} = Δ_d / C4 = **0.379** in

Δ_S = ($\Delta_{b`\_Cop}$ + Δ_{d_Cop}) / 2 = **0.227** in

Deflection of diaphragm a

L = c

Ga_d = if(L>=b`,6kips/in,4kips/in) = **4.000** kips/in

v = c * b` / TA * P / 2 / b` = **16.318** plf

v_n = v/(2ft^{-1}) = **8.159** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.000** lbf

n = Round((L/20ft-1)/2+0.49,0) = **0.000**

Δ_cX = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.000** ft^2

; (5 * v * L^3) / (8 * E * A * b`) * 0.0246 = **0.000** in
; (v * L) / (4 * Ga_{d}) = **0.017** in
; 0.188 * L * (e_n/1lbf)*0.0832 = **0.000** in
; Δ_cX /(2*b`) = **0.000** in

Δ_a = (5 * v * L^3) / (8 * E * A * b`) * 0.0246 + (v * L) / (4 * Ga_{d} ) + 0.188 * L * ($e_n$/1lbf)*0.0832 + $\Delta_c$X /(2*b`) = **0.017** in

Times 2.5 for an unblocked diaphragm...$\Delta_{a\_unblocked}$ = Δ_a * 2.5 = **0.042** in

DAf = if($\Delta_{a\_unblocked}$ > 2*Δ_S,"Flexible", "Semi-rigid") = **"Semi-rigid"**

Deflection of wall b:

Calculate uniform load from diaph. depth $V = (a'*b) / 2 / TA * P =$ **599.673** lbf

$v = V / b =$ **19.929** plf

Nail load at 2/ft $v_n = v/(2ft^{-1}) =$ **9.965** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018} =$ **0.000** lbf

; $(8 * v * h^3) / (E * A * b)/12 =$ **0.000** in

; $(v * h) / (Ga) =$ **0.014** in

; $0.75 * h * (e_n/1lbf)/12 =$ **0.000** in

; $h / b * d_a =$ **0.033** in

$\Delta_b = (8 * v * h^3) / (E * A * b) /12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / b * d_a =$ **0.048** in

$\Delta_{b\_Cop} = \Delta_b / C2 =$ **0.066** in

$\Delta_S = (\Delta_{b\_Cop} + \Delta_{d\_Cop}) / 2 =$ **0.222** in

Diaphragm b

L = a`

Ga_d = if(L>=b,6kips/in,4kips/in) = **4.000** kips/in

v = (b*a`) / TA * P / 2 / b = **19.929** plf

$v_n = v/(2ft^{-1}) =$ **9.965** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018} =$ **0.000** lbf

n = Round((L/20ft-1)/2+0.49,0) = **0.000**

$\Delta_c X$ = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.000** ft^2

; $(5 * v * L^3) / (8 * E * A * b) * 0.0246 =$ **0.000** in

; (v * L) / (4 * Ga_{d}) = **0.025** in

; $0.188 * L * (e_n/1lbf)*0.0832 =$ **0.000** in

; $\Delta_c X /(2*b) =$ **0.000** in

Δ_b = (5 * v * L^3) / (8 * E * A * b) * 0.0246 + (v * L) / (4 * Ga_{d}) + 0.188 * L * (e_n/1lbf)*0.0832 + Δ_cX /(2*b) = **0.025** in

Times 2.5 for an unblocked diaphragm...$\Delta_{b\_unblocked}$ = Δ_b * 2.5 = **0.063** in

DBf = if($\Delta_{b\_unblocked}$ > 2*Δ_S,"Flexible", "Semi-rigid") = **"Semi-rigid"**

<u>X-Axis Seismic.</u>

Diaphragm C

Deflection of wall a`:

Calculate uniform load from diaph. depth	V = (a'*d) / 2 /TA * P = **131.195** lbf
	v = V / a' = **6.510** plf
Nail load at 2/ft	v_n = v/(2ft^{-1}) = **3.255** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.000** lbf

; (8 * v * h^3) / (E * A * a`)/12 = **0.000** in

; (v * h) / (Ga) = **0.005** in

; 0.75 * h * (e_n/1lbf)/12 = **0.000** in

; h / a` * d_a = **0.050** in

$\Delta_{a`}$ = (8 * v * $h^3$ ) / (E * A * a`)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / a` * d_a = **0.054** in

$\Delta_{a`\_Cop}$ = $\Delta_{a`}$ / C5 = **0.115** in

Deflection of wall c:

Calculate uniform load from diaph. depth	V = (b'*a + a'*d) / 2 /TA * P = **983.253** lbf
	v = V / c = **59.591** plf

Nail load at 2/ft $v_n = v/(2ft^{-1})$ = **29.796** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf

; $(8 * v * h^3) / (E * A * c)/12$ = **0.001** in

; $(v * h) / (Ga)$ = **0.043** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.001** in

; $h / c * d_a$ = **0.061** in

$\Delta_c = (8 * v * h^3) / (E * A * c)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / c * d_a$ = **0.105** in

$\Delta_{c\_Cop} = \Delta_c / C3$ = **0.144** in

$\Delta_S = (\Delta_{a`\_Cop} + \Delta_{c\_Cop}) / 2$ = **0.130** in

Deflection of diaphragm c

L = d

Ga_d = if(L>=a`,6kips/in,4kips/in) = **4.000** kips/in

v = (d*a`) / TA * P / 2 / a` = **6.510** plf

$v_n = v/(2ft^{-1})$ = **3.255** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf

n = Round((L/20ft-1)/2+0.49,0) = **0.000**

Δ_cX = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.000** ft^2

; $(5 * v * L^3) / (8 * E * A * a`) * 0.0246$ = **0.000** in

; (v * L) / (4 * Ga_{d}) = **0.003** in

; $0.188 * L * (e_n/1lbf)*0.0832$ = **0.000** in

; $\Delta_cX /(2*a`)$ = **0.000** in

Δ_c = (5 * v * L^3) / (8 * E * A * a`) * 0.0246 + (v * L) / (4 * Ga_{d} ) + 0.188 * L * ($e_n$/1lbf)*0.0832 + $\Delta_c$X /(2*a`) = **0.003** in

Times 2.5 for an unblocked diaphragm...$\Delta_{c\_unblocked}$ = Δ_c * 2.5 = **0.007** in

DCf = if($\Delta_{c\_unblocked}$ > 2*Δ_S,"Flexible", "Semi-rigid") = **"Semi-rigid"**

Deflection of wall a:

Calculate uniform load from diaph. depth   V = (b'*a) / 2 /TA * P = **852.058** lbf

v = V / a = **23.247** plf

Nail load at 2/ft v_n = v/(2ft^{-1}) = **11.624** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.000** lbf

; (8 * v * h^3) / (E * A * a)/12 = **0.000** in

; (v * h) / (Ga) = **0.017** in

; 0.75 * h * (e_n/1lbf)/12 = **0.000** in

; h / a * d_a = **0.027** in

Δ_a = (8 * v * h^3) / (E * A * a) /12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / a * d_a = **0.044** in

Δ_{a_Cop} = Δ_a / C1 = **0.077** in

Δ_S = (Δ_{a_Cop} + Δ_{c_Cop}) / 2 = **0.111** in

Deflection of diaphragm d

L = b`

Ga_d = if(L>=a,6kips/in,4kips/in) = **4.000** kips/in

v = (a * b`) / TA * P / 2 / a= **23.247** plf

$v_n = v/(2ft^{-1})$ = **11.624** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf

n = Round((L/20ft-1)/2+0.49,0) = **1.000**

$\Delta_c X$ = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.208** ft^2

; $(5 * v * L^3) / (8 * E * A * a) * 0.0246$ = **0.000** in

; (v * L) / (4 * Ga_{d}) = **0.034** in

; $0.188 * L * (e_n/1lbf)*0.0832$ = **0.000** in

; $\Delta_c X /(2*a)$ = **0.034** in

$\Delta_d = (5 * v * L^3) / (8 * E * A * a) * 0.0246 + (v * L) / (4 * Ga\_\{d\}) + 0.188 * L * (e_n/1lbf)*0.0832 + \Delta_c X /(2*a)$ = **0.068** in

Times 2.5 for an unblocked diaphragm...$\Delta_{d\_unblocked} = \Delta_d * 2.5$ = **0.171** in

DDf = if($\Delta_{d\_unblocked} > 2*\Delta_S$,"Flexible", "Semi-rigid") = **"Semi-rigid"**

SUMMARY:

Diaphragm	Rigid Analysis Says...	Flexible Analysis Says...
A	DAr = **"Semi-rigid"**	DAf = **"Semi-rigid"**
B	DBr = **"Semi-rigid"**	DBf = **"Semi-rigid"**
C	DCr = **"Semi-rigid"**	DCf = **"Semi-rigid"**
D	DDr = **"Flexible"**	DDf = **"Semi-rigid"**

Note that we have a problem here. For Diaphragm D, rigid analysis says the diaphragm is flexible, but tributary area (flexible) analysis says it is semi-rigid. So how does one design it?

SEISMIC LOAD CALCULATIONS FOR THE ANALYTICAL MODELS.

SEISMIC FORCES (ASCE 7)

Tedds calculation version 3.0.03

Site parameters

Site class; D

Mapped acceleration parameters (Section 11.4.1)

at short period; S_S = **0.983**

at 1 sec period; S_1 = **0.345**

Site coefficientat short period (Table 11.4-1); F_a = **1.1**

at 1 sec period (Table 11.4-2); F_v = **1.7**

Spectral response acceleration parameters

at short period (Eq. 11.4-1); $S_{MS} = F_a \times S_S$ = **1.088**

at 1 sec period (Eq. 11.4-2); $S_{M1} = F_v \times S_1$ = **0.590**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); $S_{DS} = 2 / 3 \times S_{MS}$ = **0.725**

at 1 sec period (Eq. 11.4-4); $S_{D1} = 2 / 3 \times S_{M1}$ = **0.393**

Seismic design category

Risk category (Table 1.5-1); II

Seismic design category based on short period response acceleration (Table 11.6-1)

D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

D

Seismic design category; D

Approximate fundamental period

Height above base to highest level of building; h_n = **10** ft

From Table 12.8-2:

Structure type; All other systems

Building period parameter C_t; C_t = **0.02**

Building period parameter x; x = **0.75**

Approximate fundamental period (Eq 12.8-7); $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$= **0.112** sec

Building fundamental period (Sect 12.8.2); $T = T_a$ = **0.112** sec

Long-period transition period; T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.14-1); A. Bearing_Wall_Systems

15. Light-frame (wood) walls sheathed with wood structural panels

Response modification factor (Table 12.14-1); R = **6.5**

Seismic importance factor (Table 11.5-2); I_e = **1.000**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2);	$C_{s\_calc} = S_{DS} / (R / I_e)$= **0.112**
Maximum (Eq 12.8-3);	$C_{s\_max} = S_{D1} / (T \times (R / I_e))$ = **0.538**
Minimum (Eq 12.8-5);	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.032**
Seismic response coefficient;	C_s = **0.112**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure;	W = **1.0** kips
Seismic response coefficient;	C_s = **0.112**
Seismic base shear (Eq 12.8-1);	$V = C_s \times W$ = **0.1** kips

;

Calculations of Wall Stiffness for the Smith Residence.

SMITH HOUSE

Wall A

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **39.833** ft
Window lengths	W1 = **4.333** ft
	W2 = **5.250** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) –
ND*dorwid= **30.250** ft	
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **28.749** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.090**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.759**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.894**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.737**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.001** in

; (v * h) / (Ga) = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.025** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.621** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **3.555** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.338** kips/ft

k = **0.028** kips/in
k = **28.126** lbf/in

WALL B

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **17.500** ft
Window lengths	W1 = **3.417** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **1.000**
Length of full-ht sheathing	$\Sigma Li = L - (W1 + W2 + W3 + W4 + W5 + W6) - ND*dorwid$ = **11.083** ft
Area of openings	$Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid$ = **31.250** ft^2
Ratio of area of openings to total wall area	$\alpha = Ao / (L * H)$ = **0.223**
Ratio of full-ht sheathing to total wall sheathing	$\beta = \Sigma Li / L$ = **0.633**
Sheathing area ratio	$r = 1 / (1 + \alpha / \beta)$ = **0.739**
Opening Adjustment Factor	$Cop = r / (3 - 2 * r)$ = **0.486**

v = 100 plf   h = H   E = 1600000psi d_a = 0.125in   A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.001** in

; $(v * h) / (Ga)$ = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.057** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.654** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **5.460** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.220** kips/ft

k = **0.018** kips/in
k = **18.314** lbf/in

WALL C

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **5.167** ft
Window lengths	W1 = **0.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **5.167** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **0.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.000**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **1.000**

Sheathing area ratio	$r = 1 / (1 + \alpha / \beta) =$ **1.000**
Opening Adjustment Factor	$Cop = r / (3 – 2 * r) =$ **1.000**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.005** in

; (v * h) / (Ga) = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.194** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.794** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **2.794** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.430** kips/ft

k = **0.036** kips/in
k = **35.793** lbf/in

WALL D

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **11.417** ft
Window lengths	W1 = **2.250** ft
	W2 = **0.000** ft
	W3 = **0.000** ft

W4 = **0.000** ft
W5 = **0.000** ft
W6 = **0.000** ft

Number of doors ND = **1.000**

Length of full-ht sheathing ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **6.167** ft

Area of openings Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **27.750** ft^2

Ratio of area of openings to total wall area α = Ao / (L * H) = **0.304**

Ratio of full-ht sheathing to total wall sheathing β = ΣLi / L = **0.540**

Sheathing area ratio r = 1 / (1 + α / β) = **0.640**

Opening Adjustment Factor Cop = r / (3 – 2 * r) = **0.372**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.002** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; h / L * d_a = **0.088** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.685** in

Δ_{s_Cop} = Δ_s / Cop = **7.217** in

k = v * 1ft / Δ_{s_Cop} = **0.166** kips/ft

k = **0.014** kips/in
k = **13.857** lbf/in

WALL E

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **43.750** ft
Window lengths	W1 = **6.250** ft
	W2 = **9.250** ft
	W3 = **2.750** ft
	W4 = **2.750** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**

Length of full-ht sheathing ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **22.750** ft

Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **63.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.180**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.520**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.743**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.491**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.001** in

; (v * h) / (Ga) = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.023** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.619** in

Δ_{s_Cop} = Δ_s / Cop = **5.339** in

k = v * 1ft / Δ_{s_Cop} = **0.225** kips/ft

k = **0.019** kips/in
k = **18.731** lbf/in

WALL F

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **30.200** ft
Window lengths	W1 = **8.670** ft
	W2 = **7.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **1.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **11.530** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **68.010** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.281**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.382**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.576**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.311**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.001** in

; $(v * h) / (Ga)$ = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.033** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.629** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **8.446** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.142** kips/ft

k = **0.012** kips/in

k = **11.840** lbf/in

CALIBRATE PLATE THICKNESS TO DIAPHRAGM FLEXIBILITY

Assumptions:

5) 2 No. 2, DF-L 2x6 studs are used as chords.

   A = (1.5in*5.5in)*2 = **16.500** in^2

   E = 1600000psi

6) Roof sheathing is 7/16" CDX w/8d nails at 6" O.C.

   a. Case 1 - Ga_d is 6.0 kips/in

   b. Case 2-6 - Ga_d is 4.0 kips/in

Deflection of diaphragm on X-axis:

Diaphragm long axis length	a = 45 ft
Diaphragm breadth	b = 30.1 ft
Diaphragm length	L = a

Sheathing stiffness	Ga_d = if(L>=max(a,b),6kips/in,4kips/in) = **6.000** kips/in
Diaphragm seismic load	v = 4.8kips / L * 0.7 = **74.667** plf
Shear per nail	$v_n = v/(2ft^{-1})$ = **37.333** lbf
Nail slip	$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.000** lbf
Number of chord splices	n = Round((L/20ft-1)/2+0.49,0) = **1.000**
Chord slip factor	$\Delta_c X$ = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.208** ft^2
Flexural deflection	$(5 * v * L^3) / (8 * E * A * b) * 0.0246$ = **0.002** in
Shear deflection	$(v * L) / (4 * Ga_d)$ = **0.140** in
Nail slip	$0.188 * L * (e_n/1lbf)*0.0832$ = **0.002** in
Chord slip	$\Delta_c X /(2*b)$ = **0.042** in
Blocked diaphragm deflection	$\Delta_d = (5 * v * L^3) / (8 * E * A * b) * 0.0246 + (v * L) / (4 * Ga_d) + 0.188 * L * (e_n/1lbf)*0.0832 + \Delta_c X /(2*b)$ = **0.185** in
Unblocked diaphragm deflection	$\Delta_{d\_unblocked} = \Delta_d * 2.5$ = **0.463** in

Deflection of diaphragm on y-axis:

Diaphragm long axis length	a = 30.1 ft
Diaphragm breadth	b = 45 ft
Diaphragm length	L = a
Sheathing stiffness	Ga_d = if(L>= max(a,b),6kips/in,4kips/in) = **4.000** kips/in
Diaphragm seismic load	v = 4.8kips / L * 0.7 = **111.628** plf
Shear per nail	$v_n = v/(2ft^{-1})$ = **55.814** lbf
Nail slip	$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf
Number of chord splices	n = Round((L/20ft-1)/2+0.49,0) = **1.000**
Chord slip factor	$\Delta_c X$ = 2*(1in/32*(2*(max(n-4,0) + max(n-3,0) + max(n-2,0) + max(n-1,0) + max(n,0))*20ft)) = **0.208** ft^2
Flexural deflection	$(5 * v * L^3) / (8 * E * A * b) * 0.0246$ = **0.000** in
Shear deflection	$(v * L) / (4 * Ga_d)$ = **0.210** in
Nail slip	$0.188 * L * (e_n/1lbf)*0.0832$ = **0.005** in
Chord slip	$\Delta_c X /(2*b)$ = **0.028** in
Blocked diaphragm deflection	$\Delta_d = (5 * v * L^3) / (8 * E * A * b) * 0.0246 + (v * L) / (4 * Ga_d) + 0.188 * L * (e_n/1lbf)*0.0832 + \Delta_c X /(2*b)$ = **0.243** in
Unblocked diaphragm deflection	$\Delta_{d\_unblocked} = \Delta_d * 2.5$ = **0.608** in

Northridge California Seismic Design Parameters.

SEISMIC FORCES (ASCE 7)

Conterminous 48 States
2009 International Building Code
Zip Code: 91327
Spectral Response Accelerations Ss and S1
Ss and S1: Mapped Spectral Acceleration Values
Data are based on a 0.01 deg grid spacing

Period (sec)	Centroid Sa (g)	
0.2	2.139	(Ss)
1.0	0.744	(S1)

Period (sec)	Maximum Sa (g)	
0.2	2.139	(Ss)
1.0	0.744	(S1)

Period (sec)	Minimum Sa (g)	
0.2	2.139	(Ss)
1.0	0.744	(S1)

Conterminous 48 States
2009 International Building Code
Zip Code: 91327
Spectral Response Accelerations SMs and SM1
SMs: Fa x Ss and SM1: Fv x S1
Site Class D

Period (sec)	Centroid Sa (g)	
0.2	2.139	(SMs, Fa: 1.000)
1.0	1.116	(SM1, Fv: 1.500)

Period (sec)	Maximum Sa (g)	
0.2	2.139	(SMs, Fa: 1.000)
1.0	1.116	(SM1, Fv: 1.500)

Period Minimum Sa

(sec)	(g)	
0.2	2.139	(SMs, Fa: 1.000)
1.0	1.116	(SM1, Fv: 1.500)

Conterminous 48 States
2009 International Building Code
Zip Code: 91327
Spectral Response Accelerations SDs and SD1
SDs: 2/3 x SMs and SD1: 2/3 x SM1
Site Class D

Period	Centroid Sa	
(sec)	(g)	
0.2	1.426	(SDs)
1.0	0.744	(SD1)

Period	Maximum Sa	
(sec)	(g)	
0.2	1.426	(SDs)
1.0	0.744	(SD1)

Period	Minimum Sa	
(sec)	(g)	
0.2	1.426	(SDs)
1.0	0.744	(SD1)

Seismic Design Loads for Smith Residence.

SEISMIC FORCES (ASCE 7-10)

Tedds calculation version 3.0.03

Site parameters

Site class	D
Mapped acceleration parameters (Section 11.4.1)	
at short period	S_S = **2.139**
at 1 sec period	S_1 = **0.744**
Site coefficientat short period (Table 11.4-1)	F_a = **1.0**
at 1 sec period (Table 11.4-2)	F_v = **1.5**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S$ = **2.139**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1$ = **1.116**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS}$ = **1.426**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1}$ = **0.744**

Seismic design category

Risk category (Table 1.5-1) II

Seismic design category based on short period response acceleration (Table 11.6-1)

D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

D

Seismic design category D

Approximate fundamental period

Height above base to highest level of building h_n = **10** ft

From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**

Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x$= **0.112** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.112** sec
Long-period transition period	T_L = **12** sec

Alternative site and spectral response parameters for seismic response coefficient (Sect. 12.8.1.3)

Mapped accelteration parameters

at short period S_{Salt} = 1.50

Site coefficients

at short period F_{aalt} = 1.00

Spectral response acceleration parameters
at short period S_{MSalt} = 1.50

Design spectral response acceleration parameters
at short period S_{DSalt} = 1.00

Seismic response coefficient

Seismic force-resisting system (Table 12.14-1) A. Bearing_Wall_Systems
15. Light-frame (wood) walls sheathed with wood structural panels

Response modification factor (Table 12.14-1) R = **6.5**

Seismic importance factor (Table 11.5-2) I_e = **1.000**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) $C_{s\_calc} = S_{DSalt} / (R / I_e)$= **0.154**

Maximum ((Eq 12.8-3)) $C_{s\_max} = S_{D1} / (T \times (R / I_e))$ = **1.018**

Minimum:

Eq 12.8-5 $C_{s\_min1} = \max(0.044 \times S_{DSalt} \times I_e, 0.01)$ = **0.044**

Eq 12.8-6 (where S_1 >= 0.6) $C_{s\_min2} = (0.5 \times S_1) / (R / I_e)$ = **0.057**

C_{s_min} = **0.057**

Seismic response coefficient C_s = **0.154**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **31.0** kips

Seismic response coefficient C_s = **0.154**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **4.8** kips

Calculations of Wall Stiffness for the Olsen Residence.

OLSEN HOUSE

First Level

Wall A1

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **22.167** ft
Window lengths	W1 = **16.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **6.167** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **48.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.271**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.278**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.507**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.255**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.001** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; $h / L * d_a$ = **0.045** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.642** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **10.353** in

k = v * 1ft / Δ_{s_Cop} = **0.116** kips/ft

k = **0.010** kips/in
k = **9.659** lbf/in

WALL B1

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **20.167** ft
Window lengths	W1 = **0.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **20.167** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **0.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.000**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **1.000**
Sheathing area ratio	r = 1 / (1 + α / β) = **1.000**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **1.000**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

$Ga = ((1293lbf/8ft) * 8\ ft / (0.08*8ft) + (862lbf/8ft) * 8\ ft / (0.02*8ft))/2 =$ **0.309** kips/in

$v_n = v/(2ft^{-1}) =$ **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018} =$ **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12 =$ **0.001** in

; $(v * h) / (Ga) =$ **2.592** in

; $0.75 * h * (e_n/1lbf)/12 =$ **0.004** in

; $h / L * d_a =$ **0.050** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a =$ **2.646** in

$\Delta_{s\_Cop} = \Delta_s / Cop =$ **2.646** in

$k = v * 1ft / \Delta_{s\_Cop} =$ **0.453** kips/ft

k = **0.038** kips/in
k = **37.788** lbf/in

WALL C1

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **7.000** ft
Window lengths	W1 = **0.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **1.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **4.000** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **21.000** ft^2

Ratio of area of openings to total wall area α = Ao / (L * H) = **0.375**
Ratio of full-ht sheathing to total wall sheathing $\beta = \Sigma$Li / L = **0.571**
Sheathing area ratio r = 1 / (1 + α / β) = **0.604**

Opening Adjustment Factor Cop = r / (3 – 2 * r) = **0.337**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.003** in

; (v * h) / (Ga) = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.143** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.742** in

$\Delta_{s\_Cop} = \Delta_s$ / Cop = **8.140** in

k = v * 1ft / Δ_{s_Cop} = **0.147** kips/ft

k = **0.012** kips/in
k = **12.285** lbf/in

WALL D1

Assumed window height winht = 3 ft
Assumed wall height H = 8 ft
Assumed door height dorht = 7 ft
Assumed door width dorwid = 3 ft
Total wall length L = **25.000** ft

Window lengths W1 = **4.167** ft
W2 = **5.167** ft
W3 = **0.000** ft

	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **1.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **12.667** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **49.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.245**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.507**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.674**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.408**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.001** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; h / L * d_a = **0.040** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.637** in

$\Delta_{s\_Cop} = \Delta_s$ / Cop = **6.461** in

k = v * 1ft / Δ_{s_Cop} = **0.186** kips/ft

k = **0.015** kips/in
k = **15.477** lbf/in

WALL E1

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **35.167** ft
Window lengths	W1 = **4.667** ft
	W2 = **8.333** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **22.167** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **39.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.139**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.630**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.820**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.602**

v = 100 plf   h = H   E = 1600000psi d_a = 0.125in   A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.001** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; h / L * d_a = **0.028** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.625** in

Δ_{s_Cop} = Δ_s / Cop = **4.356** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.275** kips/ft

k = **0.023** kips/in
k = **22.955** lbf/in

WALL F1

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **13.000** ft
Window lengths	W1 = **0.000** ft
	W2 = **5.250** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **7.750** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **15.750** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.151**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.596**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.797**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.568**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.002** in

; $(v * h) / (Ga)$ = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.077** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.674** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **4.712** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.255** kips/ft

k = **0.021** kips/in
k = **21.220** lbf/in

Wall G1

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **16.000** ft
Window lengths	W1 = **10.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **6.000** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **30.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.234**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.375**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.615**

Opening Adjustment Factor Cop = r / (3 – 2 * r) = **0.348**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.002** in

; (v * h) / (Ga) = **2.592** in

; $0.75 * h * (e_n/1lbf)/12$ = **0.004** in

; $h / L * d_a$ = **0.063** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.660** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **7.646** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.157** kips/ft

k = **0.013** kips/in
k = **13.078** lbf/in

Wall H1

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **7.000** ft
Window lengths	W1 = **0.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft

	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **7.000** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **0.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.000**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **1.000**
Sheathing area ratio	r = 1 / (1 + α / β) = **1.000**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **1.000**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.003** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; h / L * d_a = **0.143** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.742** in

Δ_{s_Cop} = Δ_s / Cop = **2.742** in

k = v * 1ft / Δ_{s_Cop} = **0.438** kips/ft

k = **0.036** kips/in
k = **36.471** lbf/in

Second Level

Wall A2

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **26.000** ft
Window lengths	W1 = **4.500** ft
	W2 = **4.500** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **17.000** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **27.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.130**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.654**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.834**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.627**

v = 100 plf   h = H   E = 1600000psi d_a = 0.125in   A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.001** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; h / L * d_a = **0.038** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.635** in

$\Delta_{s\_Cop} = \Delta_s$ / Cop = **4.204** in

k = v * 1ft / Δ_{s_Cop} = **0.285** kips/ft

k = **0.024** kips/in
k = **23.785** lbf/in

Wall B2

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **17.000** ft
Window lengths	W1 = **0.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **17.000** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **0.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.000**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **1.000**
Sheathing area ratio	r = 1 / (1 + α / β) = **1.000**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **1.000**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

$e_n = 1.2\text{lbf}*(v_n/616\text{lbf})^{3.018}$ = **0.001** lbf

; $(8 * v * h^3) / (E * A * L)/12$ = **0.001** in

; $(v * h) / (Ga)$ = **2.592** in

; $0.75 * h * (e_n/1\text{lbf})/12$ = **0.004** in

; $h / L * d_a$ = **0.059** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1\text{lbf})/12 + h / L * d_a$ = **2.656** in

$\Delta_{s\_Cop} = \Delta_s$ / Cop = **2.656** in

k = v * 1ft / Δ_{s_Cop} = **0.452** kips/ft

k = **0.038** kips/in
k = **37.653** lbf/in

Wall C2

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **7.000** ft
Window lengths	W1 = **0.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **7.000** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **0.000** ft^2

Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.000**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **1.000**
Sheathing area ratio	r = 1 / (1 + α / β) = **1.000**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **1.000**

v = 100 plf   h = H   E = 1600000psi d_a = 0.125in   A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

$v_n = v/(2ft^{-1})$ = **50.000** lbf

$e_n = 1.2lbf*(v_n/616lbf)^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.003** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; h / L * d_a = **0.143** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.742** in

$\Delta_{s\_Cop} = \Delta_s$ / Cop = **2.742** in

k = v * 1ft / Δ_{s_Cop} = **0.438** kips/ft

k = **0.036** kips/in

k = **36.471** lbf/in

Wall D2

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **22.250** ft

Window lengths	W1 = **0.000** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **22.250** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **0.000** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.000**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **1.000**
Sheathing area ratio	r = 1 / (1 + α / β) = **1.000**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **1.000**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.001** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; h / L * d_a = **0.045** in

Δ_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a = **2.642** in

Δ_{s_Cop} = Δ_s / Cop = **2.642** in

k = v * 1ft / Δ_{s_Cop} = **0.454** kips/ft

k = **0.038** kips/in
k = **37.856** lbf/in

Wall E2

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **36.500** ft
Window lengths	W1 = **4.250** ft
	W2 = **4.000** ft
	W3 = **8.250** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **20.000** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **49.500** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.170**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.548**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.764**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.519**

v = 100 plf h = H E = 1600000psi d_a = 0.125in A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.001** lbf

; (8 * v * h^3) / (E * A * L)/12 = **0.001** in

; (v * h) / (Ga) = **2.592** in

; 0.75 * h * (e_n/1lbf)/12 = **0.004** in

; $h / L * d_a$ = **0.027** in

$\Delta_s = (8 * v * h^3) / (E * A * L)/12 + (v * h) / (Ga) + 0.75 * h * (e_n/1lbf)/12 + h / L * d_a$ = **2.624** in

$\Delta_{s\_Cop} = \Delta_s / Cop$ = **5.059** in

$k = v * 1ft / \Delta_{s\_Cop}$ = **0.237** kips/ft

k = **0.020** kips/in
k = **19.768** lbf/in

Wall F2

Assumed window height	winht = 3 ft
Assumed wall height	H = 8 ft
Assumed door height	dorht = 7 ft
Assumed door width	dorwid = 3 ft
Total wall length	L = **26.000** ft
Window lengths	W1 = **6.500** ft
	W2 = **0.000** ft
	W3 = **0.000** ft
	W4 = **0.000** ft
	W5 = **0.000** ft
	W6 = **0.000** ft
Number of doors	ND = **0.000**
Length of full-ht sheathing	ΣLi = L – (W1 + W2 + W3 + W4 + W5 + W6) – ND*dorwid= **19.500** ft
Area of openings	Ao = (W1+W2+W3+W4+W5+W6)*winht + ND*dorht*dorwid = **19.500** ft^2
Ratio of area of openings to total wall area	α = Ao / (L * H) = **0.094**
Ratio of full-ht sheathing to total wall sheathing	β = ΣLi / L = **0.750**
Sheathing area ratio	r = 1 / (1 + α / β) = **0.889**
Opening Adjustment Factor	Cop = r / (3 – 2 * r) = **0.727**

v = 100 plf   h = H   E = 1600000psi d_a = 0.125in   A = 2 * (1.5in) * (3.5in) = **10.500** in^2

Ga = ((1293lbf/8ft) * 8 ft / (0.08*8ft) + (862lbf/8ft) * 8 ft / (0.02*8ft))/2 = **0.309** kips/in

v_n = v/(2ft^{-1}) = **50.000** lbf

e_n = 1.2lbf*(v_n/616lbf)$^{3.018}$ = **0.001** lbf

;   (8 * v * h^3) / (E * A * L)/12 = **0.001** in

;   (v * h) / (Ga) = **2.592** in

;   0.75 * h * (e_n/1lbf)/12 = **0.004** in

;   h / L * d_a = **0.038** in

Δ_s = (8 * v * h^3) / (E * A * L)/12   +   (v * h) / (Ga)   +   0.75 * h * (e_n/1lbf)/12   +   h / L * d_a = **2.635** in

Δ_{s_Cop} = Δ_s / Cop = **3.623** in

k = v * 1ft / Δ_{s_Cop} = **0.331** kips/ft

k = **0.028** kips/in

k = **27.601** lbf/in

Seismic Design Loads for Olsen Residence.

SEISMIC FORCES (ASCE 7-10)

Tedds calculation version 3.0.03

Site parameters

Site class D

Mapped acceleration parameters (Section 11.4.1)

at short period S_S = **2.139**

at 1 sec period S_1 = **0.744**

Site coefficientat short period (Table 11.4-1) F_a = **1.0**

at 1 sec period (Table 11.4-2) F_v = **1.5**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S$ = **2.139**

at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1$ = **1.116**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS}$ = **1.426**

at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1}$ = **0.744**

Seismic design category

Risk category (Table 1.5-1) II

Seismic design category based on short period response acceleration (Table 11.6-1)

D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

D

Seismic design category D

Approximate fundamental period

Height above base to highest level of building h_n = **19** ft

From Table 12.8-2:

Structure type All other systems

Building period parameter C_t C_t = **0.02**

Building period parameter x x = **0.75**

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$ = **0.182** sec

Building fundamental period (Sect 12.8.2) $T = T_a$ = **0.182** sec

Long-period transition period T_L = **12** sec

Alternative site and spectral response parameters for seismic response coefficient (Sect. 12.8.1.3)

Mapped accelteration parameters

at short period S_{Salt} = 1.50

Site coefficients

at short period F_{aalt} = 1.00

Spectral response acceleration parameters
at short period S_{MSalt} = 1.50

Design spectral response acceleration parameters
at short period S_{DSalt} = 1.00

Seismic response coefficient

Seismic force-resisting system (Table 12.14-1) A. Bearing_Wall_Systems
15. Light-frame (wood) walls sheathed with wood structural panels

Response modification factor (Table 12.14-1) R = **6.5**

Seismic importance factor (Table 11.5-2) I_e = **1.000**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) $C_{s\_calc} = S_{DSalt} / (R / I_e)$= **0.154**

Maximum ((Eq 12.8-3)) $C_{s\_max} = S_{D1} / (T \times (R / I_e))$ = **0.629**

Minimum:

Eq 12.8-5 $C_{s\_min1} = \max(0.044 \times S_{DSalt} \times I_e, 0.01)$ = **0.044**

Eq 12.8-6 (where S_1 >= 0.6) $C_{s\_min2} = (0.5 \times S_1) / (R / I_e)$ = **0.057**

C_{s_min} = **0.057**

Seismic response coefficient C_s = **0.154**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **63.0** kips

Seismic response coefficient C_s = **0.154**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **9.7** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12) $C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$

Lateral force induced at level i (Eq 12.8-11) $F_x = C_{vx} \times V$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x
1	9.0	33.0	1.00	0.343	3.3
2	19.0	30.0	1.00	0.657	6.4

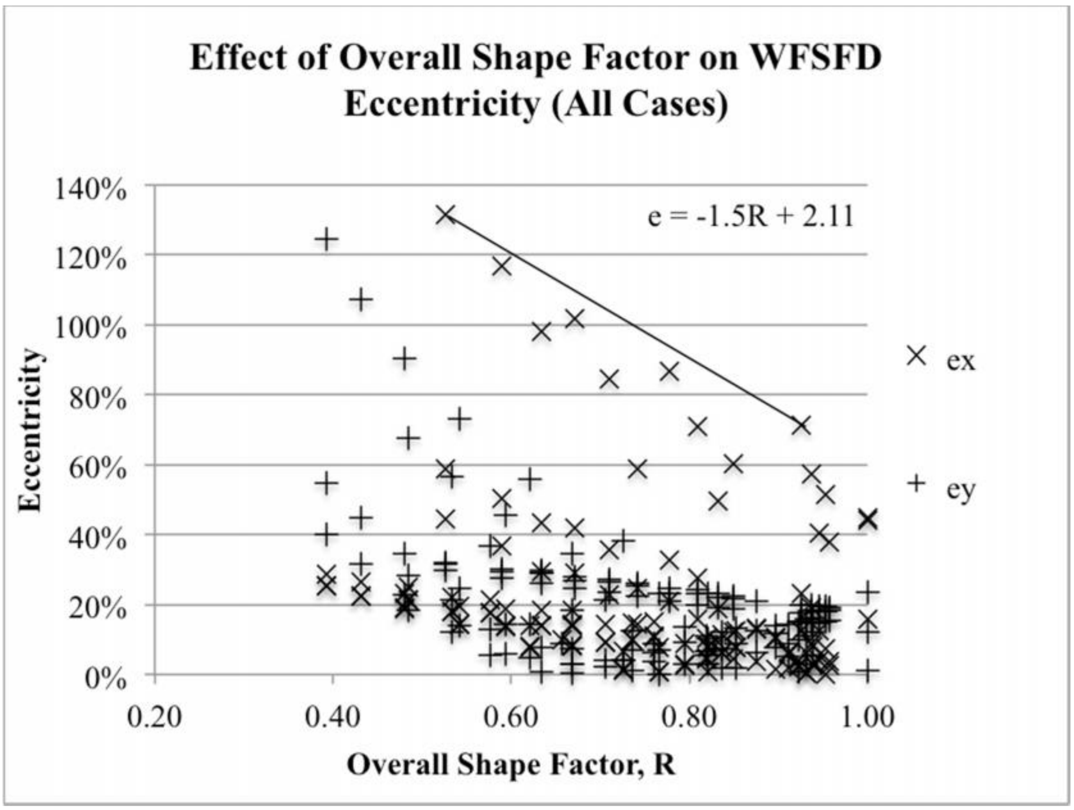


Figure B.1. Effect of Overall Shape Factor on WFSFD Eccentricity – All Cases.

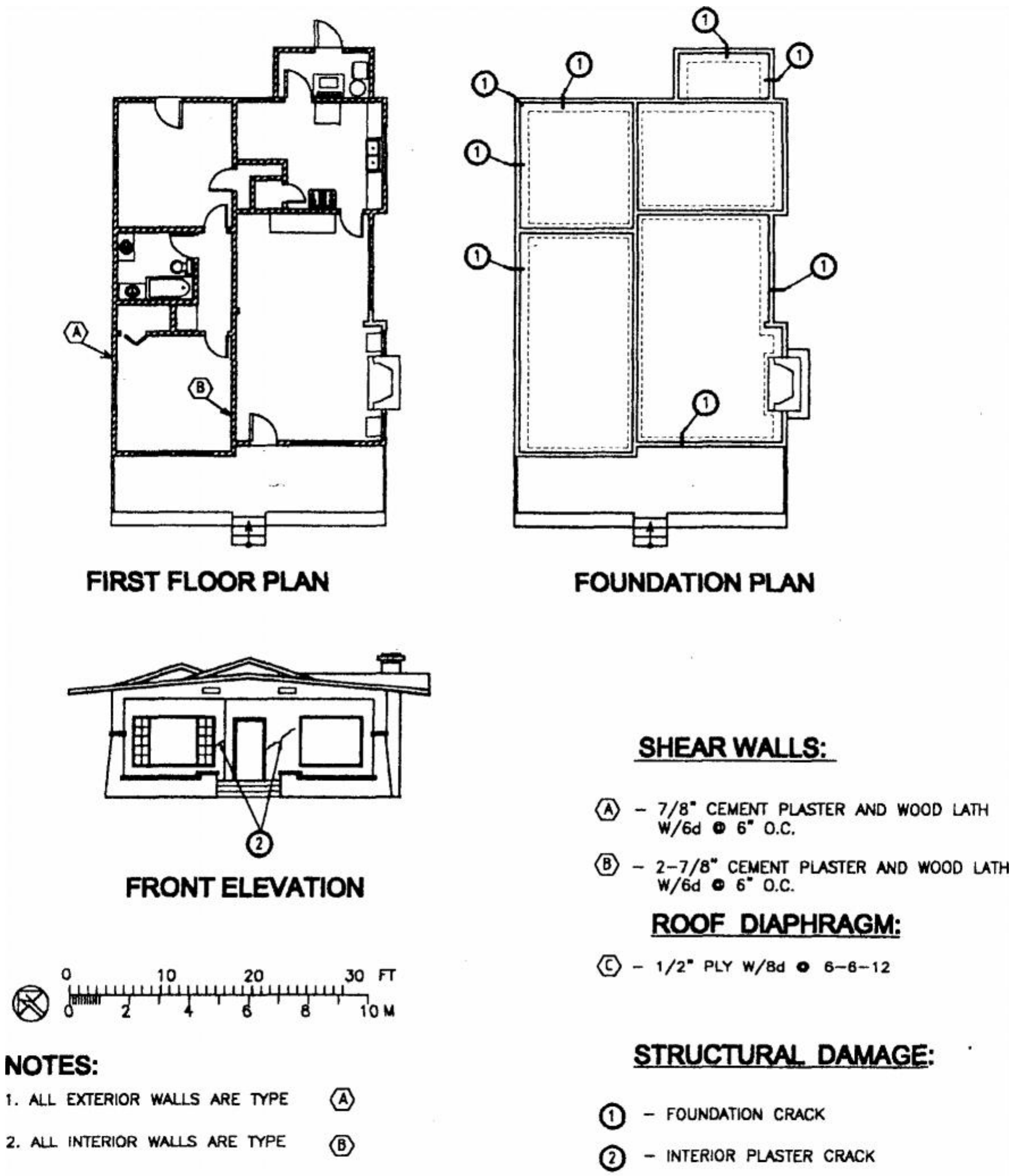


Figure B.2. Smith Residence (Schierle 2003).

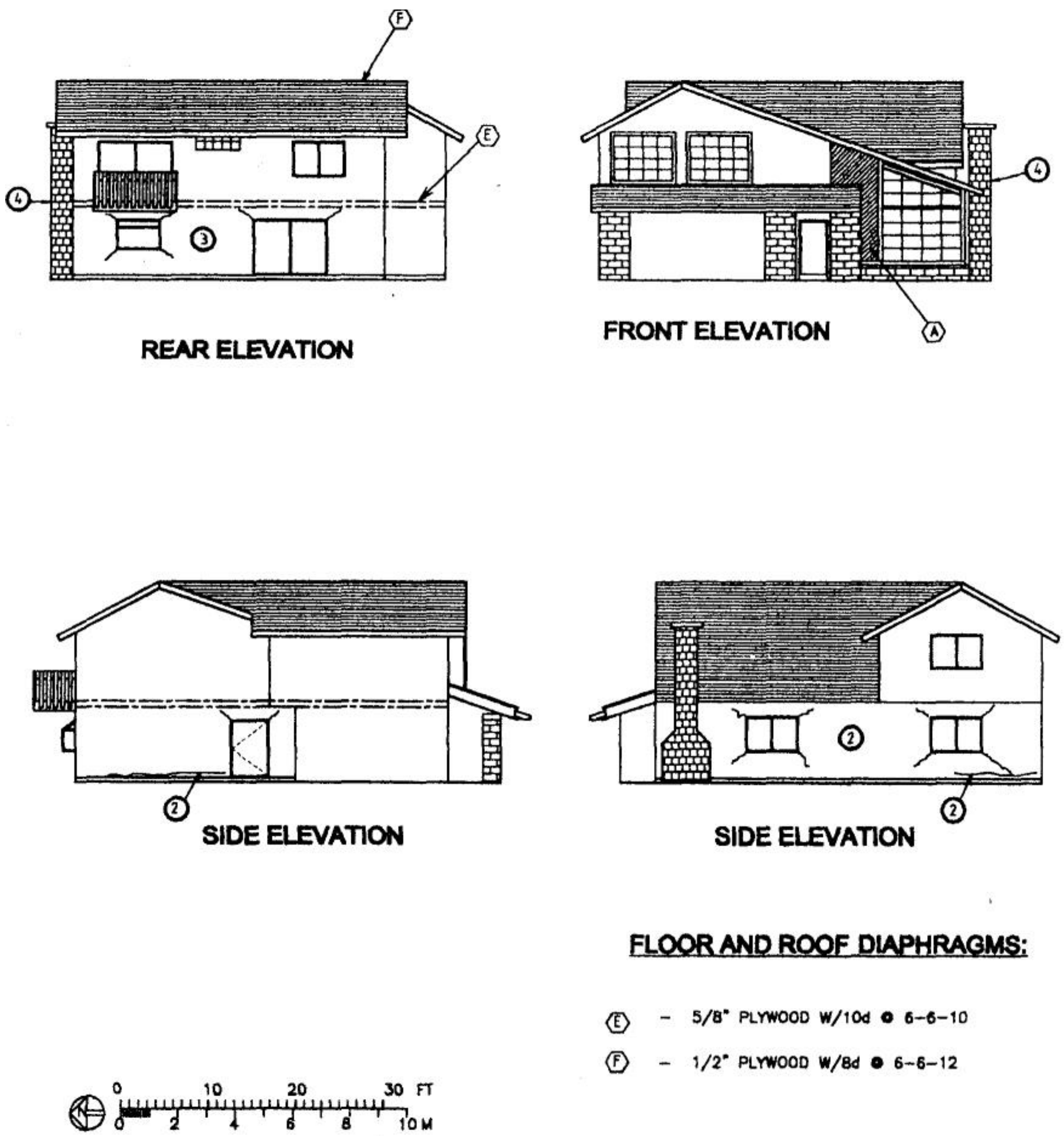


Figure B.3. Olsen Residence Elevations (Schierle 2003).

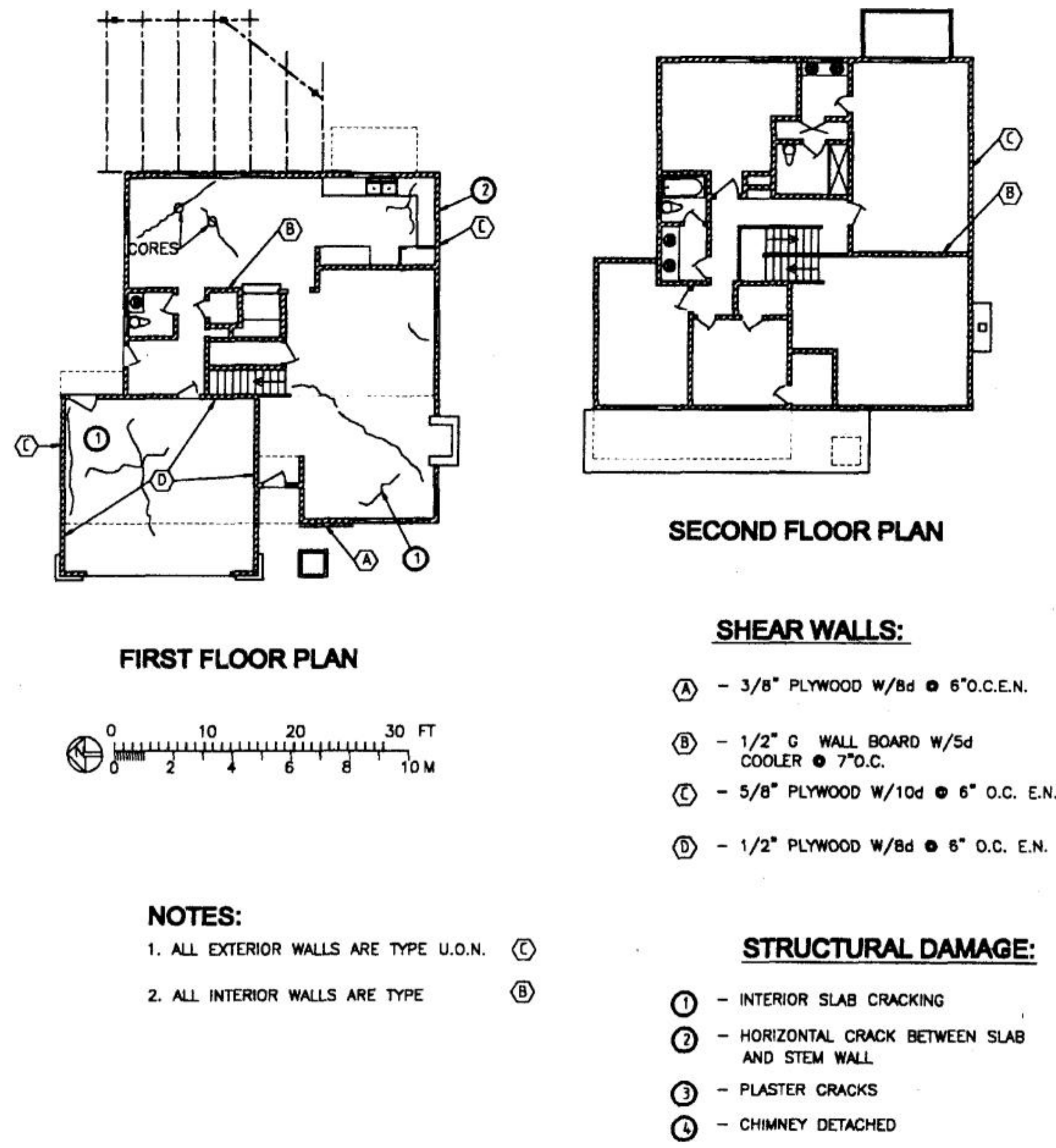


Figure B.4. Olsen Residence Floor Plans (Schierle 2003).

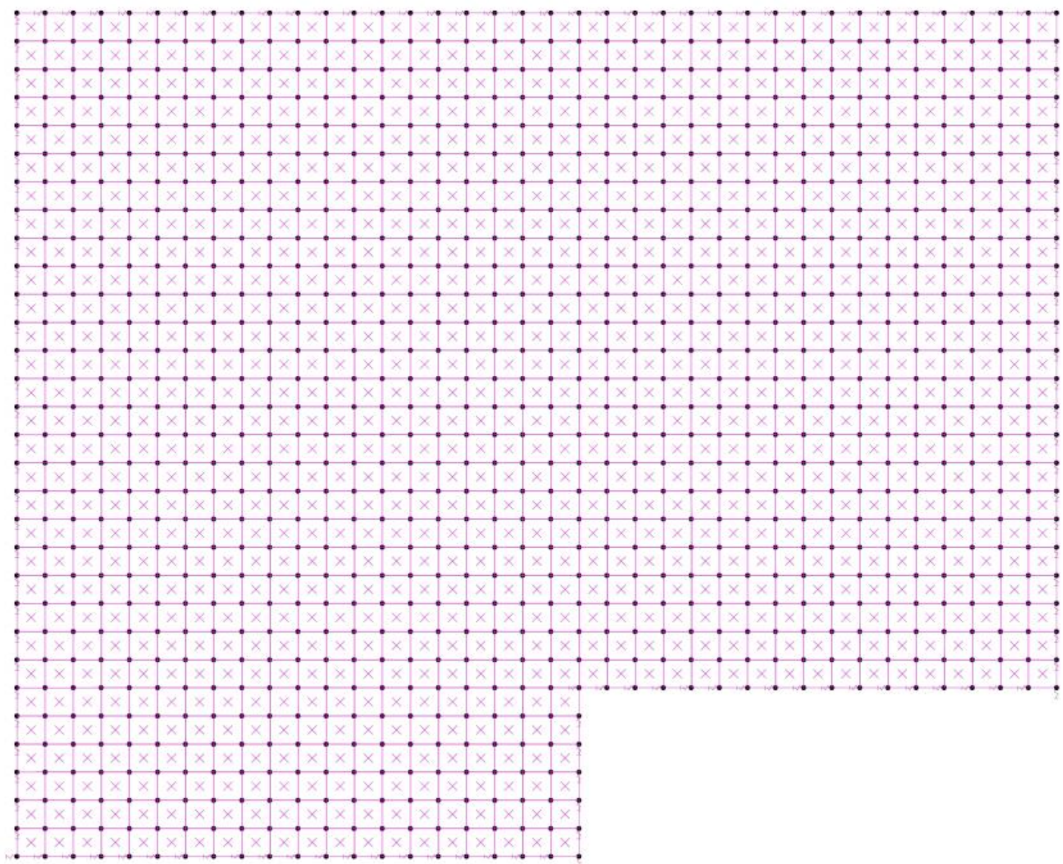


Figure B.5. Example of Paevere House (2003) RP FEM X-Axis Deflected Shape.

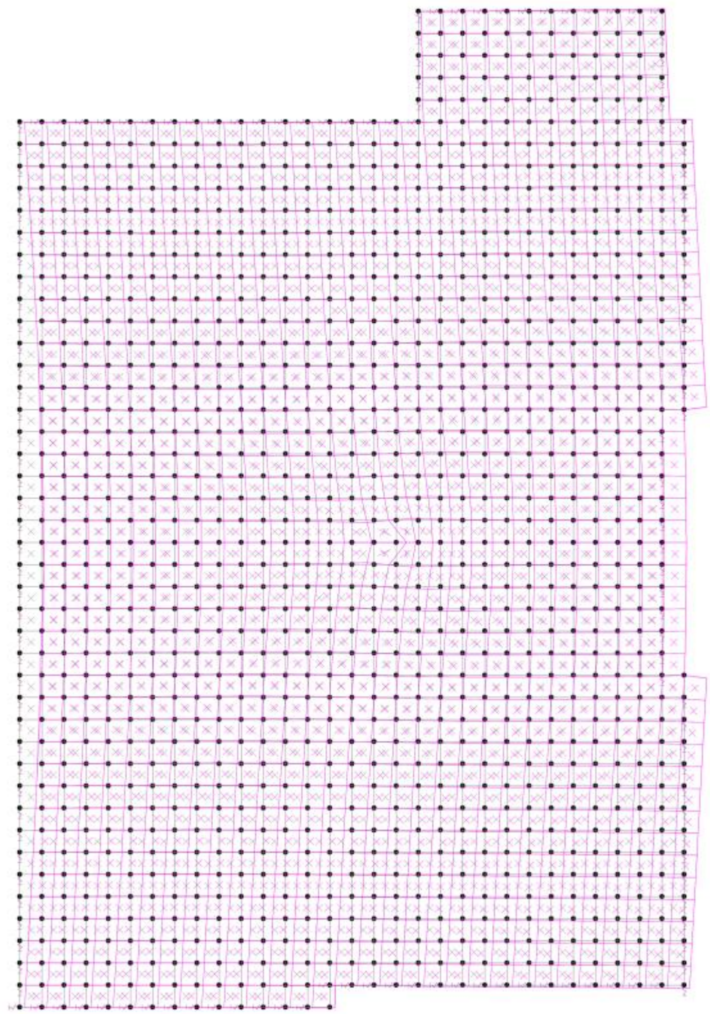
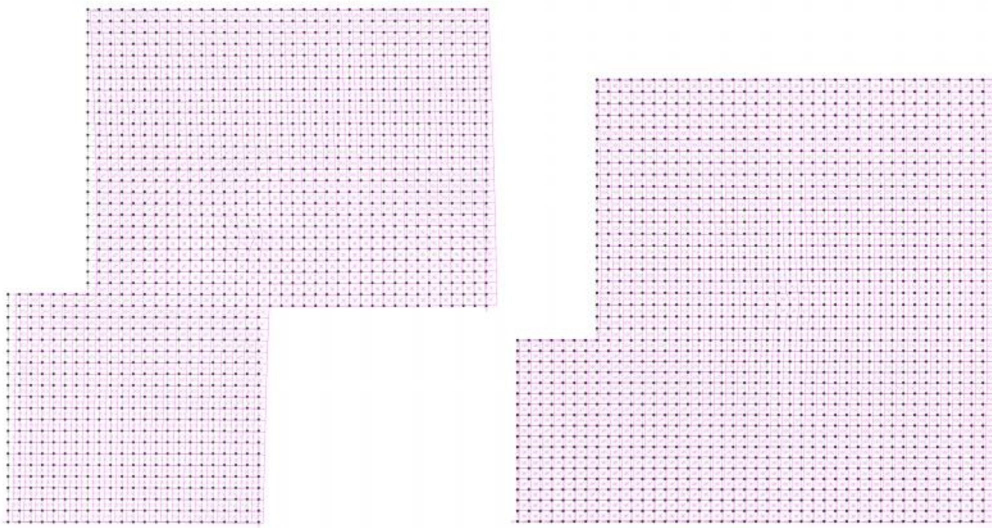


Figure B.6. Example of Smith Residence (Schierle 2003)
SP FEM Y-Axis Deflected Shape.



(a) Level 1. (b) Level 2.

Figure B.7. Example of Olsen Residence (Schierle 2003)
FP FEM X-Axis Deflected Shape.

Table B.1. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment										
		Wall a		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid	Wind. wid					b_{fh}/b		h_o/h	C_o
		(ft)	(ft)	a	b	r	Cop	Raw	Looku	Raw	
Case 1	P82_10	3	9.10	16.5%	0.670	0.80	0.58	0.670	0.7	0.833	0.63
	P100_0	3	8.53	18.2%	0.640	0.78	0.54	0.640	0.6	0.833	0.57
	P88_8	3	7.31	16.8%	0.678	0.80	0.57	0.678	0.7	0.833	0.63
	P93_16	3	7.40	16.9%	0.675	0.80	0.57	0.675	0.7	0.833	0.63
	P78_22	3	7.48	17.0%	0.673	0.80	0.57	0.673	0.7	0.833	0.63
	P67_26	3	7.54	17.0%	0.671	0.80	0.57	0.671	0.7	0.833	0.63
	P59_29	3	7.60	17.1%	0.669	0.80	0.57	0.669	0.7	0.833	0.63
	P53_32	3	7.65	17.2%	0.667	0.80	0.56	0.667	0.7	0.833	0.63
	P73_8	3	9.63	16.2%	0.673	0.81	0.58	0.673	0.7	0.833	0.63
	P90_15	3	9.93	16.5%	0.665	0.80	0.57	0.665	0.7	0.833	0.63
	P94_20	3	10.17	16.7%	0.658	0.80	0.57	0.658	0.7	0.833	0.63
	P81_24	3	10.39	16.9%	0.653	0.79	0.56	0.653	0.7	0.833	0.63
	P71_27	3	10.57	17.1%	0.648	0.79	0.56	0.648	0.6	0.833	0.57
	P63_29	3	10.72	17.2%	0.644	0.79	0.55	0.644	0.6	0.833	0.57
	P62_9	3	11.84	15.7%	0.671	0.81	0.59	0.671	0.7	0.833	0.63
	P77_18	3	12.11	15.9%	0.665	0.81	0.58	0.665	0.7	0.833	0.63
	P91_24	3	12.34	16.1%	0.660	0.80	0.58	0.660	0.7	0.833	0.63
	P95_28	3	12.54	16.2%	0.656	0.80	0.57	0.656	0.7	0.833	0.63
	P83_31	3	12.72	16.4%	0.652	0.80	0.57	0.652	0.7	0.833	0.63
	P74_34	3	12.87	16.5%	0.648	0.80	0.57	0.648	0.6	0.833	0.57
	P54_10	3	14.11	15.3%	0.669	0.81	0.59	0.669	0.7	0.833	0.63
	P67_20	3	14.36	15.5%	0.664	0.81	0.59	0.664	0.7	0.833	0.63
	P80_26	3	14.57	15.7%	0.660	0.81	0.58	0.660	0.7	0.833	0.63
	P92_31	3	14.76	15.8%	0.656	0.81	0.58	0.656	0.7	0.833	0.63
	P95_35	3	14.92	15.9%	0.653	0.80	0.58	0.653	0.7	0.833	0.63
	P85_37	3	15.06	16.0%	0.651	0.80	0.58	0.651	0.7	0.833	0.63
	P48_10	3	16.43	15.1%	0.666	0.82	0.60	0.666	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall a		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	b_{fh}/b Looku	h_o/h Raw	C_o
P59_21	3	16.65	15.2%	0.663	0.81	0.59	0.663	0.7	0.833	0.63
P71_28	3	16.84	15.4%	0.659	0.81	0.59	0.659	0.7	0.833	0.63
P82_33	3	17.01	15.5%	0.656	0.81	0.59	0.656	0.7	0.833	0.63
P93_37	3	17.16	15.6%	0.654	0.81	0.58	0.654	0.7	0.833	0.63
P96_40	3	17.29	15.6%	0.652	0.81	0.58	0.652	0.7	0.833	0.63
P43_11	3	18.78	14.9%	0.664	0.82	0.60	0.664	0.7	0.833	0.63
P53_22	3	18.98	15.0%	0.661	0.81	0.59	0.661	0.7	0.833	0.63
P63_30	3	19.15	15.1%	0.658	0.81	0.59	0.658	0.7	0.833	0.63
P74_35	3	19.31	15.2%	0.656	0.81	0.59	0.656	0.7	0.833	0.63
P84_39	3	19.44	15.3%	0.654	0.81	0.59	0.654	0.7	0.833	0.63
P94_42	3	19.56	15.4%	0.652	0.81	0.59	0.652	0.7	0.833	0.63
P39_12	3	21.17	14.8%	0.661	0.82	0.60	0.661	0.7	0.833	0.63
P48_23	3	21.34	14.9%	0.659	0.82	0.60	0.659	0.7	0.833	0.63
P58_31	3	21.49	15.0%	0.657	0.81	0.59	0.657	0.7	0.833	0.63
P67_36	3	21.63	15.0%	0.655	0.81	0.59	0.655	0.7	0.833	0.63
P76_41	3	21.75	15.1%	0.653	0.81	0.59	0.653	0.7	0.833	0.63
P85_44	3	21.86	15.2%	0.652	0.81	0.59	0.652	0.7	0.833	0.63
P82_10	3	15.20	16.5%	0.503	0.75	0.50	0.503	0.5	0.833	0.57
P100_0	3	17.07	18.2%	0.373	0.67	0.41	0.373	0.4	0.833	0.53
P88_8	3	12.80	16.8%	0.506	0.75	0.50	0.506	0.5	0.833	0.57
P93_16	3	13.13	16.9%	0.496	0.75	0.49	0.496	0.5	0.833	0.57
P78_22	3	13.40	17.0%	0.488	0.74	0.49	0.488	0.5	0.833	0.57
P67_26	3	13.63	17.0%	0.480	0.74	0.48	0.480	0.5	0.833	0.57
P59_29	3	13.83	17.1%	0.474	0.73	0.48	0.474	0.5	0.833	0.57
P53_32	3	14.01	17.2%	0.468	0.73	0.48	0.468	0.5	0.833	0.57
P73_8	3	15.70	16.2%	0.515	0.76	0.51	0.515	0.5	0.833	0.57
P90_15	3	16.46	16.5%	0.495	0.75	0.50	0.495	0.5	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall a		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Looku	h_o/h Raw	C_o
Case 2	P94_20	3	17.11	16.7%	0.478	0.74	0.49	0.478	0.5	0.833	0.57
	P81_24	3	17.68	16.9%	0.464	0.73	0.48	0.464	0.5	0.833	0.57
	P71_27	3	18.18	17.1%	0.451	0.73	0.47	0.451	0.5	0.833	0.57
	P63_29	3	18.63	17.2%	0.439	0.72	0.46	0.439	0.4	0.833	0.53
	P62_9	3	18.28	15.7%	0.528	0.77	0.53	0.528	0.5	0.833	0.57
	P77_18	3	19.06	15.9%	0.511	0.76	0.52	0.511	0.5	0.833	0.57
	P91_24	3	19.74	16.1%	0.496	0.76	0.51	0.496	0.5	0.833	0.57
	P95_28	3	20.35	16.2%	0.483	0.75	0.50	0.483	0.5	0.833	0.57
	P83_31	3	20.89	16.4%	0.471	0.74	0.49	0.471	0.5	0.833	0.57
	P74_34	3	21.37	16.5%	0.460	0.74	0.48	0.460	0.5	0.833	0.57
	P54_10	3	20.85	15.3%	0.539	0.78	0.54	0.539	0.5	0.833	0.57
	P67_20	3	21.65	15.5%	0.523	0.77	0.53	0.523	0.5	0.833	0.57
	P80_26	3	22.36	15.7%	0.509	0.76	0.52	0.509	0.5	0.833	0.57
	P92_31	3	22.99	15.8%	0.497	0.76	0.51	0.497	0.5	0.833	0.57
	P95_35	3	23.57	15.9%	0.486	0.75	0.50	0.486	0.5	0.833	0.57
	P85_37	3	24.09	16.0%	0.476	0.75	0.50	0.476	0.5	0.833	0.57
	P48_10	3	23.43	15.1%	0.546	0.78	0.55	0.546	0.5	0.833	0.57
	P59_21	3	24.23	15.2%	0.532	0.78	0.54	0.532	0.5	0.833	0.57
	P71_28	3	24.96	15.4%	0.520	0.77	0.53	0.520	0.5	0.833	0.57
	P82_33	3	25.62	15.5%	0.509	0.77	0.52	0.509	0.5	0.833	0.57
	P93_37	3	26.22	15.6%	0.498	0.76	0.52	0.498	0.5	0.833	0.57
	P96_40	3	26.77	15.6%	0.489	0.76	0.51	0.489	0.5	0.833	0.57
	P43_11	3	26.00	14.9%	0.553	0.79	0.55	0.553	0.6	0.833	0.57
	P53_22	3	26.82	15.0%	0.540	0.78	0.54	0.540	0.5	0.833	0.57
	P63_30	3	27.56	15.1%	0.528	0.78	0.54	0.528	0.5	0.833	0.57
	P74_35	3	28.24	15.2%	0.518	0.77	0.53	0.518	0.5	0.833	0.57
	P84_39	3	28.87	15.3%	0.508	0.77	0.53	0.508	0.5	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall a		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Looku	h_o/h Raw	C_o
P94_42	3	29.44	15.4%	0.499	0.76	0.52	0.499	0.5	0.833	0.57
P39_12	3	28.57	14.8%	0.558	0.79	0.56	0.558	0.6	0.833	0.57
P48_23	3	29.40	14.9%	0.546	0.79	0.55	0.546	0.5	0.833	0.57
P58_31	3	30.16	15.0%	0.535	0.78	0.54	0.535	0.5	0.833	0.57
P67_36	3	30.86	15.0%	0.526	0.78	0.54	0.526	0.5	0.833	0.57
P76_41	3	31.50	15.1%	0.517	0.77	0.53	0.517	0.5	0.833	0.57
P85_44	3	32.10	15.2%	0.508	0.77	0.53	0.508	0.5	0.833	0.57
P82_10	3	15.20	16.5%	0.503	0.75	0.50	0.503	0.5	0.833	0.57
P100_0	3	17.07	18.2%	0.373	0.67	0.41	0.373	0.4	0.833	0.53
P88_8	3	12.80	16.8%	0.506	0.75	0.50	0.506	0.5	0.833	0.57
P93_16	15	13.13	49.7%	0.121	0.20	0.08	0.121	0.1	0.833	0.43
P78_22	15	13.40	49.8%	0.113	0.18	0.07	0.113	0.1	0.833	0.43
P67_26	15	13.63	49.9%	0.105	0.17	0.07	0.105	0.1	0.833	0.43
P59_29	15	13.83	49.9%	0.099	0.17	0.06	0.099	0.1	0.833	0.43
P53_32	15	14.01	50.0%	0.093	0.16	0.06	0.093	0.1	0.833	0.43
P73_8	3	15.70	16.2%	0.515	0.76	0.51	0.515	0.5	0.833	0.57
P90_15	3	16.46	16.5%	0.495	0.75	0.50	0.495	0.5	0.833	0.57
P94_20	15	17.11	43.9%	0.167	0.28	0.11	0.167	0.2	0.833	0.45
P81_24	15	17.68	44.1%	0.152	0.26	0.10	0.152	0.2	0.833	0.45
P71_27	15	18.18	44.3%	0.139	0.24	0.09	0.139	0.1	0.833	0.43
P63_29	15	18.63	44.5%	0.128	0.22	0.09	0.128	0.1	0.833	0.43
P62_9	3	18.28	15.7%	0.528	0.77	0.53	0.528	0.5	0.833	0.57
P77_18	3	19.06	15.9%	0.511	0.76	0.52	0.511	0.5	0.833	0.57
P91_24	3	19.74	16.1%	0.496	0.76	0.51	0.496	0.5	0.833	0.57
P95_28	15	20.35	39.5%	0.217	0.35	0.15	0.217	0.2	0.833	0.45
P83_31	15	20.89	39.7%	0.205	0.34	0.15	0.205	0.2	0.833	0.45
P74_34	15	21.37	39.8%	0.194	0.33	0.14	0.194	0.2	0.833	0.45

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall a		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Looku	h_o/h Raw	C_o
Case 3	P54_10	3	20.85	15.3%	0.539	0.78	0.54	0.539	0.5	0.833	0.57
	P67_20	3	21.65	15.5%	0.523	0.77	0.53	0.523	0.5	0.833	0.57
	P80_26	3	22.36	15.7%	0.509	0.76	0.52	0.509	0.5	0.833	0.57
	P92_31	3	22.99	15.8%	0.497	0.76	0.51	0.497	0.5	0.833	0.57
	P95_35	27	23.57	56.5%	0.022	0.04	0.01	0.022	0	0.833	0
	P85_37	27	24.09	56.6%	0.012	0.02	0.01	0.012	0	0.833	0
	P48_10	3	23.43	15.1%	0.546	0.78	0.55	0.546	0.5	0.833	0.57
	P59_21	3	24.23	15.2%	0.532	0.78	0.54	0.532	0.5	0.833	0.57
	P71_28	3	24.96	15.4%	0.520	0.77	0.53	0.520	0.5	0.833	0.57
	P82_33	3	25.62	15.5%	0.509	0.77	0.52	0.509	0.5	0.833	0.57
	P93_37	3	26.22	15.6%	0.498	0.76	0.52	0.498	0.5	0.833	0.57
	P96_40	27	26.77	51.7%	0.077	0.13	0.05	0.077	0.1	0.833	0.43
	P43_11	3	26.00	14.9%	0.553	0.79	0.55	0.553	0.6	0.833	0.57
	P53_22	3	26.82	15.0%	0.540	0.78	0.54	0.540	0.5	0.833	0.57
	P63_30	3	27.56	15.1%	0.528	0.78	0.54	0.528	0.5	0.833	0.57
	P74_35	3	28.24	15.2%	0.518	0.77	0.53	0.518	0.5	0.833	0.57
	P84_39	3	28.87	15.3%	0.508	0.77	0.53	0.508	0.5	0.833	0.57
	P94_42	3	29.44	15.4%	0.499	0.76	0.52	0.499	0.5	0.833	0.57
	P39_12	3	28.57	14.8%	0.558	0.79	0.56	0.558	0.6	0.833	0.57
	P48_23	3	29.40	14.9%	0.546	0.79	0.55	0.546	0.5	0.833	0.57
	P58_31	3	30.16	15.0%	0.535	0.78	0.54	0.535	0.5	0.833	0.57
	P67_36	3	30.86	15.0%	0.526	0.78	0.54	0.526	0.5	0.833	0.57
	P76_41	3	31.50	15.1%	0.517	0.77	0.53	0.517	0.5	0.833	0.57
	P85_44	3	32.10	15.2%	0.508	0.77	0.53	0.508	0.5	0.833	0.57
	Smith	3	11.74	16.1%	0.663	0.80	0.58	0.663	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall a'		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 1	P82_10	3	5.00	22.3%	0.603	0.73	0.47	0.603	0.6	0.833	0.57
	P100_0	3	8.53	18.2%	0.640	0.78	0.54	0.640	0.6	0.833	0.57
	P88_8	3	3.54	25.5%	0.578	0.69	0.43	0.578	0.6	0.833	0.57
	P93_16	3	3.58	25.6%	0.575	0.69	0.43	0.575	0.6	0.833	0.57
	P78_22	3	3.62	25.7%	0.573	0.69	0.43	0.573	0.6	0.833	0.57
	P67_26	3	3.65	25.8%	0.571	0.69	0.42	0.571	0.6	0.833	0.57
	P59_29	3	3.68	25.8%	0.569	0.69	0.42	0.569	0.6	0.833	0.57
	P53_32	3	3.70	25.9%	0.568	0.69	0.42	0.568	0.6	0.833	0.57
	P73_8	3	5.03	22.4%	0.601	0.73	0.47	0.601	0.6	0.833	0.57
	P90_15	3	5.19	22.7%	0.594	0.72	0.47	0.594	0.6	0.833	0.57
	P94_20	3	5.32	22.9%	0.587	0.72	0.46	0.587	0.6	0.833	0.57
	P81_24	3	5.43	23.1%	0.582	0.72	0.46	0.582	0.6	0.833	0.57
	P71_27	3	5.52	23.3%	0.577	0.71	0.45	0.577	0.6	0.833	0.57
	P63_29	3	5.60	23.5%	0.573	0.71	0.45	0.573	0.6	0.833	0.57
	P62_9	3	5.29	22.9%	0.589	0.72	0.46	0.589	0.6	0.833	0.57
	P77_18	3	5.41	23.1%	0.583	0.72	0.46	0.583	0.6	0.833	0.57
	P91_24	3	5.51	23.3%	0.578	0.71	0.45	0.578	0.6	0.833	0.57
	P95_28	3	5.60	23.5%	0.573	0.71	0.45	0.573	0.6	0.833	0.57
	P83_31	3	5.68	23.6%	0.569	0.71	0.45	0.569	0.6	0.833	0.57
	P74_34	3	5.75	23.7%	0.566	0.70	0.44	0.566	0.6	0.833	0.57
	P54_10	3	5.50	23.3%	0.578	0.71	0.45	0.578	0.6	0.833	0.57
	P67_20	3	5.60	23.4%	0.573	0.71	0.45	0.573	0.6	0.833	0.57
	P80_26	3	5.68	23.6%	0.569	0.71	0.45	0.569	0.6	0.833	0.57
	P92_31	3	5.75	23.7%	0.566	0.70	0.44	0.566	0.6	0.833	0.57
	P95_35	3	5.82	23.8%	0.563	0.70	0.44	0.563	0.6	0.833	0.57
	P85_37	3	5.87	24.0%	0.560	0.70	0.44	0.560	0.6	0.833	0.57
	P48_10	3	5.68	23.6%	0.569	0.71	0.45	0.569	0.6	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment										
	Wall a'		per HUD/PATH, 6-30				Per Breyer, 10.26				
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	b_{fh}/b Lookup	h_o/h Raw	C_o
P59_21	3	5.76	23.7%	0.565	0.70	0.44	0.565	0.6	0.833	0.57
P71_28	3	5.83	23.9%	0.562	0.70	0.44	0.562	0.6	0.833	0.57
P82_33	3	5.89	24.0%	0.559	0.70	0.44	0.559	0.6	0.833	0.57
P93_37	3	5.94	24.1%	0.556	0.70	0.44	0.556	0.6	0.833	0.57
P96_40	3	5.98	24.2%	0.554	0.70	0.43	0.554	0.6	0.833	0.57
P43_11	3	5.84	23.9%	0.561	0.70	0.44	0.561	0.6	0.833	0.57
P53_22	3	5.90	24.0%	0.558	0.70	0.44	0.558	0.6	0.833	0.57
P63_30	3	5.96	24.1%	0.556	0.70	0.43	0.556	0.6	0.833	0.57
P74_35	3	6.00	24.2%	0.553	0.70	0.43	0.553	0.6	0.833	0.57
P84_39	3	6.05	24.3%	0.551	0.69	0.43	0.551	0.6	0.833	0.57
P94_42	3	6.08	24.3%	0.549	0.69	0.43	0.549	0.5	0.833	0.57
P39_12	3	5.98	24.1%	0.555	0.70	0.43	0.555	0.6	0.833	0.57
P48_23	3	6.03	24.2%	0.552	0.69	0.43	0.552	0.6	0.833	0.57
P58_31	3	6.07	24.3%	0.550	0.69	0.43	0.550	0.5	0.833	0.57
P67_36	3	6.11	24.4%	0.548	0.69	0.43	0.548	0.5	0.833	0.57
P76_41	3	6.14	24.5%	0.546	0.69	0.43	0.546	0.5	0.833	0.57
P85_44	3	6.17	24.5%	0.545	0.69	0.43	0.545	0.5	0.833	0.57
P82_10	3	8.36	22.3%	0.436	0.66	0.39	0.436	0.4	0.833	0.53
P100_0	3	17.07	18.2%	0.373	0.67	0.41	0.373	0.4	0.833	0.53
P88_8	3	6.20	25.5%	0.406	0.61	0.35	0.406	0.4	0.833	0.53
P93_16	3	6.36	25.6%	0.396	0.61	0.34	0.396	0.4	0.833	0.53
P78_22	3	6.49	25.7%	0.388	0.60	0.33	0.388	0.4	0.833	0.53
P67_26	3	6.60	25.8%	0.380	0.60	0.33	0.380	0.4	0.833	0.53
P59_29	3	6.70	25.8%	0.374	0.59	0.33	0.374	0.4	0.833	0.53
P53_32	3	6.79	25.9%	0.369	0.59	0.32	0.369	0.4	0.833	0.53
P73_8	3	8.21	22.4%	0.444	0.66	0.40	0.444	0.4	0.833	0.53
P90_15	3	8.60	22.7%	0.424	0.65	0.38	0.424	0.4	0.833	0.53

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment										
		Wall a'		per HUD/PATH, 6-30				Per Breyer, 10.26				
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 2	P94_20	3	8.94	22.9%	0.407	0.64	0.37	0.407	0.4	0.833	0.53
	P81_24	3	9.24	23.1%	0.393	0.63	0.36	0.393	0.4	0.833	0.53
	P71_27	3	9.50	23.3%	0.380	0.62	0.35	0.380	0.4	0.833	0.53
	P63_29	3	9.73	23.5%	0.368	0.61	0.34	0.368	0.4	0.833	0.53
	P62_9	3	8.16	22.9%	0.446	0.66	0.39	0.446	0.4	0.833	0.53
	P77_18	3	8.51	23.1%	0.429	0.65	0.38	0.429	0.4	0.833	0.53
	P91_24	3	8.82	23.3%	0.414	0.64	0.37	0.414	0.4	0.833	0.53
	P95_28	3	9.09	23.5%	0.400	0.63	0.36	0.400	0.4	0.833	0.53
	P83_31	3	9.33	23.6%	0.388	0.62	0.35	0.388	0.4	0.833	0.53
	P74_34	3	9.54	23.7%	0.377	0.61	0.35	0.377	0.4	0.833	0.53
	P54_10	3	8.13	23.3%	0.448	0.66	0.39	0.448	0.4	0.833	0.53
	P67_20	3	8.44	23.4%	0.432	0.65	0.38	0.432	0.4	0.833	0.53
	P80_26	3	8.72	23.6%	0.419	0.64	0.37	0.419	0.4	0.833	0.53
	P92_31	3	8.96	23.7%	0.406	0.63	0.36	0.406	0.4	0.833	0.53
	P95_35	3	9.19	23.8%	0.395	0.62	0.36	0.395	0.4	0.833	0.53
	P85_37	3	9.39	24.0%	0.385	0.62	0.35	0.385	0.4	0.833	0.53
	P48_10	3	8.10	23.6%	0.449	0.66	0.39	0.449	0.4	0.833	0.53
	P59_21	3	8.38	23.7%	0.435	0.65	0.38	0.435	0.4	0.833	0.53
	P71_28	3	8.64	23.9%	0.423	0.64	0.37	0.423	0.4	0.833	0.53
	P82_33	3	8.86	24.0%	0.411	0.63	0.36	0.411	0.4	0.833	0.53
	P93_37	3	9.07	24.1%	0.401	0.62	0.36	0.401	0.4	0.833	0.53
	P96_40	3	9.26	24.2%	0.391	0.62	0.35	0.391	0.4	0.833	0.53
	P43_11	3	8.08	23.9%	0.450	0.65	0.39	0.450	0.4	0.833	0.53
	P53_22	3	8.34	24.0%	0.437	0.65	0.38	0.437	0.4	0.833	0.53
	P63_30	3	8.57	24.1%	0.426	0.64	0.37	0.426	0.4	0.833	0.53
	P74_35	3	8.78	24.2%	0.415	0.63	0.36	0.415	0.4	0.833	0.53
	P84_39	3	8.98	24.3%	0.406	0.63	0.36	0.406	0.4	0.833	0.53

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment										
	Wall a'		per HUD/PATH, 6-30				Per Breyer, 10.26				
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
P94_42	3	9.16	24.3%	0.397	0.62	0.35	0.397	0.4	0.833	0.53
P39_12	3	8.07	24.1%	0.451	0.65	0.38	0.451	0.5	0.833	0.57
P48_23	3	8.30	24.2%	0.439	0.64	0.38	0.439	0.4	0.833	0.53
P58_31	3	8.51	24.3%	0.429	0.64	0.37	0.429	0.4	0.833	0.53
P67_36	3	8.71	24.4%	0.419	0.63	0.36	0.419	0.4	0.833	0.53
P76_41	3	8.89	24.5%	0.410	0.63	0.36	0.410	0.4	0.833	0.53
P85_44	3	9.06	24.5%	0.401	0.62	0.35	0.401	0.4	0.833	0.53
P82_10	3	8.36	22.3%	0.436	0.66	0.39	0.436	0.4	0.833	0.53
P100_0	3	17.07	18.2%	0.373	0.67	0.41	0.373	0.4	0.833	0.53
P88_8	3	6.20	25.5%	0.406	0.61	0.35	0.406	0.4	0.833	0.53
P93_16	3	6.36	25.6%	0.396	0.61	0.34	0.396	0.4	0.833	0.53
P78_22	3	6.49	25.7%	0.388	0.60	0.33	0.388	0.4	0.833	0.53
P67_26	3	6.60	25.8%	0.380	0.60	0.33	0.380	0.4	0.833	0.53
P59_29	3	6.70	25.8%	0.374	0.59	0.33	0.374	0.4	0.833	0.53
P53_32	3	6.79	25.9%	0.369	0.59	0.32	0.369	0.4	0.833	0.53
P73_8	3	8.21	22.4%	0.444	0.66	0.40	0.444	0.4	0.833	0.53
P90_15	3	8.60	22.7%	0.424	0.65	0.38	0.424	0.4	0.833	0.53
P94_20	3	8.94	22.9%	0.407	0.64	0.37	0.407	0.4	0.833	0.53
P81_24	3	9.24	23.1%	0.393	0.63	0.36	0.393	0.4	0.833	0.53
P71_27	3	9.50	23.3%	0.380	0.62	0.35	0.380	0.4	0.833	0.53
P63_29	3	9.73	23.5%	0.368	0.61	0.34	0.368	0.4	0.833	0.53
P62_9	3	8.16	22.9%	0.446	0.66	0.39	0.446	0.4	0.833	0.53
P77_18	3	8.51	23.1%	0.429	0.65	0.38	0.429	0.4	0.833	0.53
P91_24	3	8.82	23.3%	0.414	0.64	0.37	0.414	0.4	0.833	0.53
P95_28	3	9.09	23.5%	0.400	0.63	0.36	0.400	0.4	0.833	0.53
P83_31	3	9.33	23.6%	0.388	0.62	0.35	0.388	0.4	0.833	0.53
P74_34	3	9.54	23.7%	0.377	0.61	0.35	0.377	0.4	0.833	0.53

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall a'		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	b_{fh}/b Lookup	h_o/h Raw	C_o
Case 3	P54_10	3	8.13	23.3%	0.448	0.66	0.39	0.448	0.4	0.833	0.53
	P67_20	3	8.44	23.4%	0.432	0.65	0.38	0.432	0.4	0.833	0.53
	P80_26	3	8.72	23.6%	0.419	0.64	0.37	0.419	0.4	0.833	0.53
	P92_31	3	8.96	23.7%	0.406	0.63	0.36	0.406	0.4	0.833	0.53
	P95_35	3	9.19	23.8%	0.395	0.62	0.36	0.395	0.4	0.833	0.53
	P85_37	3	9.39	24.0%	0.385	0.62	0.35	0.385	0.4	0.833	0.53
	P48_10	3	8.10	23.6%	0.449	0.66	0.39	0.449	0.4	0.833	0.53
	P59_21	3	8.38	23.7%	0.435	0.65	0.38	0.435	0.4	0.833	0.53
	P71_28	3	8.64	23.9%	0.423	0.64	0.37	0.423	0.4	0.833	0.53
	P82_33	3	8.86	24.0%	0.411	0.63	0.36	0.411	0.4	0.833	0.53
	P93_37	3	9.07	24.1%	0.401	0.62	0.36	0.401	0.4	0.833	0.53
	P96_40	3	9.26	24.2%	0.391	0.62	0.35	0.391	0.4	0.833	0.53
	P43_11	3	8.08	23.9%	0.450	0.65	0.39	0.450	0.4	0.833	0.53
	P53_22	3	8.34	24.0%	0.437	0.65	0.38	0.437	0.4	0.833	0.53
	P63_30	3	8.57	24.1%	0.426	0.64	0.37	0.426	0.4	0.833	0.53
	P74_35	3	8.78	24.2%	0.415	0.63	0.36	0.415	0.4	0.833	0.53
	P84_39	3	8.98	24.3%	0.406	0.63	0.36	0.406	0.4	0.833	0.53
	P94_42	3	9.16	24.3%	0.397	0.62	0.35	0.397	0.4	0.833	0.53
	P39_12	3	8.07	24.1%	0.451	0.65	0.38	0.451	0.5	0.833	0.57
	P48_23	3	8.30	24.2%	0.439	0.64	0.38	0.439	0.4	0.833	0.53
	P58_31	3	8.51	24.3%	0.429	0.64	0.37	0.429	0.4	0.833	0.53
	P67_36	3	8.71	24.4%	0.419	0.63	0.36	0.419	0.4	0.833	0.53
	P76_41	3	8.89	24.5%	0.410	0.63	0.36	0.410	0.4	0.833	0.53
	P85_44	3	9.06	24.5%	0.401	0.62	0.35	0.401	0.4	0.833	0.53
	Smith	3	10.35	16.9%	0.654	0.79	0.56	0.654	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall b		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid	Wind. wid					b_{fh}/b		h_o/h	C_o
		(ft)	(ft)	a	b	r	Cop	Raw	Lookup	Raw	
Case 1	P82_10	0	7.47	9.3%	0.752	0.89	0.73	0.752	0.8	0.833	0.77
	P100_0	0	8.53	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
	P88_8	0	6.39	9%	0.772	0.90	0.75	0.772	0.8	0.833	0.77
	P93_16	0	7.99	9%	0.769	0.90	0.75	0.769	0.8	0.833	0.77
	P78_22	0	9.61	9%	0.766	0.90	0.74	0.766	0.8	0.833	0.77
	P67_26	0	11.24	9%	0.764	0.90	0.74	0.764	0.8	0.833	0.77
	P59_29	0	12.88	9%	0.763	0.90	0.74	0.763	0.8	0.833	0.77
	P53_32	0	14.53	9%	0.761	0.89	0.74	0.761	0.8	0.833	0.77
	P73_8	0	6.99	9%	0.750	0.89	0.73	0.750	0.8	0.833	0.77
	P90_15	0	8.90	10%	0.743	0.88	0.72	0.743	0.7	0.833	0.63
	P94_20	0	10.85	10%	0.736	0.88	0.71	0.736	0.7	0.833	0.63
	P81_24	0	12.84	10%	0.731	0.88	0.71	0.731	0.7	0.833	0.63
	P71_27	0	14.87	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P63_29	0	16.91	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P62_9	0	7.35	10%	0.738	0.88	0.71	0.738	0.7	0.833	0.63
	P77_18	0	9.28	10%	0.732	0.88	0.71	0.732	0.7	0.833	0.63
	P91_24	0	11.25	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P95_28	0	13.26	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P83_31	0	15.29	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P74_34	0	17.34	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P54_10	0	7.65	10%	0.727	0.88	0.70	0.727	0.7	0.833	0.63
	P67_20	0	9.60	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P80_26	0	11.59	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P92_31	0	13.61	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P95_35	0	15.66	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
	P85_37	0	17.72	11%	0.709	0.87	0.68	0.709	0.7	0.833	0.63
	P48_10	0	7.90	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

Case	Perforated Shearwall Stiffness and Strength Adjustment										
	Wall b		per HUD/PATH, 6-30				Per Breyer, 10.26				
	Door wid	Wind. wid					b_{fh}/b		h_o/h	C_o
	(ft)	(ft)	a	b	r	Cop	Raw	Lookup	Raw	
P59_21	0	9.88	11%	0.714	0.87	0.69	0.714	0.7	0.833	0.63
P71_28	0	11.89	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
P82_33	0	13.93	11%	0.708	0.87	0.68	0.708	0.7	0.833	0.63
P93_37	0	15.98	11%	0.705	0.86	0.68	0.705	0.7	0.833	0.63
P96_40	0	18.05	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P43_11	0	8.12	11%	0.710	0.87	0.69	0.710	0.7	0.833	0.63
P53_22	0	10.12	11%	0.707	0.87	0.68	0.707	0.7	0.833	0.63
P63_30	0	12.15	11%	0.704	0.86	0.68	0.704	0.7	0.833	0.63
P74_35	0	14.21	11%	0.702	0.86	0.68	0.702	0.7	0.833	0.63
P84_39	0	16.27	11%	0.700	0.86	0.67	0.700	0.7	0.833	0.63
P94_42	0	18.36	11%	0.698	0.86	0.67	0.698	0.7	0.833	0.63
P39_12	0	8.30	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P48_23	0	10.33	11%	0.701	0.86	0.68	0.701	0.7	0.833	0.63
P58_31	0	12.38	11%	0.699	0.86	0.67	0.699	0.7	0.833	0.63
P67_36	0	14.45	11%	0.697	0.86	0.67	0.697	0.7	0.833	0.63
P76_41	0	16.53	11%	0.695	0.86	0.67	0.695	0.7	0.833	0.63
P85_44	0	18.63	11%	0.694	0.86	0.67	0.694	0.7	0.833	0.63
P82_10	0	0.00	9%	1.000	0.91	0.78	1.000	1	0.833	1
P100_0	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
P88_8	0	0.00	9%	1.000	0.92	0.80	1.000	1	0.833	1
P93_16	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P78_22	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P67_26	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P59_29	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P53_32	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P73_8	0	0.00	9%	1.000	0.91	0.78	1.000	1	0.833	1
P90_15	0	0.00	10%	1.000	0.91	0.78	1.000	1	0.833	1

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall b		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid	Wind. wid					b_{fh}/b		h_o/h	C_o
		(ft)	(ft)	a	b	r	Cop	Raw	Lookup	Raw	
Case 2	P94_20	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P81_24	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P71_27	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P63_29	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P62_9	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P77_18	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P91_24	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P95_28	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P83_31	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P74_34	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P54_10	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P67_20	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P80_26	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P92_31	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P95_35	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P85_37	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P48_10	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P59_21	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P71_28	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P82_33	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P93_37	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P96_40	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P43_11	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P53_22	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P63_30	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P74_35	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P84_39	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment										
	Wall b		per HUD/PATH, 6-30				Per Breyer, 10.26				
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
P94_42	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P39_12	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P48_23	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P58_31	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P67_36	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P76_41	0	0.00	11%	1.000	0.90	0.74	1.000	1	0.833	1
P85_44	0	0.00	11%	1.000	0.90	0.74	1.000	1	0.833	1
P82_10	27	0.00	88%	0.103	0.10	0.04	0.103	0.1	0.833	0.43
P100_0	27	0.00	84%	0.156	0.16	0.06	0.156	0.2	0.833	0.45
P88_8	27	0.00	93%	0.036	0.04	0.01	0.036	0	0.833	0
P93_16	3	0.00	16%	0.913	0.85	0.65	0.913	0.9	0.833	0.87
P78_22	3	0.00	15%	0.927	0.86	0.67	0.927	0.9	0.833	0.87
P67_26	3	0.00	14%	0.937	0.87	0.69	0.937	0.9	0.833	0.87
P59_29	3	0.00	14%	0.945	0.87	0.70	0.945	0.9	0.833	0.87
P53_32	3	0.00	13%	0.951	0.88	0.70	0.951	1	0.833	1
P73_8	27	0.00	94%	0.036	0.04	0.01	0.036	0	0.833	0
P90_15	27	0.00	78%	0.219	0.22	0.09	0.219	0.2	0.833	0.45
P94_20	3	0.00	16%	0.927	0.85	0.65	0.927	0.9	0.833	0.87
P81_24	3	0.00	16%	0.937	0.86	0.67	0.937	0.9	0.833	0.87
P71_27	3	0.00	15%	0.945	0.86	0.68	0.945	0.9	0.833	0.87
P63_29	3	0.00	15%	0.951	0.87	0.68	0.951	1	0.833	1
P62_9	27	0.00	94%	0.036	0.04	0.01	0.036	0	0.833	0
P77_18	27	0.00	78%	0.219	0.22	0.09	0.219	0.2	0.833	0.45
P91_24	27	0.00	68%	0.343	0.34	0.14	0.343	0.3	0.833	0.45
P95_28	3	0.00	16%	0.937	0.85	0.66	0.937	0.9	0.833	0.87
P83_31	3	0.00	15%	0.945	0.86	0.67	0.945	0.9	0.833	0.87
P74_34	3	0.00	15%	0.951	0.86	0.68	0.951	1	0.833	1

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment										
		Wall b		per HUD/PATH, 6-30				Per Breyer, 10.26				
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 3	P54_10	27	0.00	95%	0.036	0.04	0.01	0.036	0	0.833	0
	P67_20	27	0.00	79%	0.219	0.22	0.08	0.219	0.2	0.833	0.45
	P80_26	27	0.00	68%	0.343	0.34	0.14	0.343	0.3	0.833	0.45
	P92_31	27	0.00	60%	0.434	0.42	0.19	0.434	0.4	0.833	0.53
	P95_35	3	0.00	16%	0.945	0.86	0.67	0.945	0.9	0.833	0.87
	P85_37	3	0.00	15%	0.951	0.86	0.68	0.951	1	0.833	1
	P48_10	27	0.00	95%	0.036	0.04	0.01	0.036	0	0.833	0
	P59_21	27	0.00	79%	0.219	0.22	0.08	0.219	0.2	0.833	0.45
	P71_28	27	0.00	68%	0.343	0.33	0.14	0.343	0.3	0.833	0.45
	P82_33	27	0.00	60%	0.434	0.42	0.19	0.434	0.4	0.833	0.53
	P93_37	27	0.00	55%	0.502	0.48	0.23	0.502	0.5	0.833	0.57
	P96_40	3	0.00	15%	0.951	0.86	0.67	0.951	1	0.833	1
	P43_11	27	0.00	95%	0.036	0.04	0.01	0.036	0	0.833	0
	P53_22	27	0.00	79%	0.219	0.22	0.08	0.219	0.2	0.833	0.45
	P63_30	27	0.00	69%	0.343	0.33	0.14	0.343	0.3	0.833	0.45
	P74_35	27	0.00	61%	0.434	0.42	0.19	0.434	0.4	0.833	0.53
	P84_39	27	0.00	55%	0.502	0.48	0.23	0.502	0.5	0.833	0.57
	P94_42	27	0.00	50%	0.556	0.53	0.27	0.556	0.6	0.833	0.57
	P39_12	27	0.00	95%	0.036	0.04	0.01	0.036	0	0.833	0
	P48_23	27	0.00	80%	0.219	0.22	0.08	0.219	0.2	0.833	0.45
	P58_31	27	0.00	69%	0.343	0.33	0.14	0.343	0.3	0.833	0.45
	P67_36	27	0.00	61%	0.434	0.42	0.19	0.434	0.4	0.833	0.53
	P76_41	27	0.00	55%	0.502	0.48	0.23	0.502	0.5	0.833	0.57
	P85_44	27	0.00	50%	0.556	0.52	0.27	0.556	0.6	0.833	0.57
	Smith	0	7.72	10%	0.732	0.88	0.71	0.732	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall b'		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 1	P82_10	0	5.84	9.3%	0.752	0.89	0.73	0.752	0.8	0.833	0.77
	P100_0	0	8.53	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
	P88_8	0	5.37	9%	0.772	0.90	0.75	0.772	0.8	0.833	0.77
	P93_16	0	5.44	9%	0.769	0.90	0.75	0.769	0.8	0.833	0.77
	P78_22	0	5.49	9%	0.766	0.90	0.74	0.766	0.8	0.833	0.77
	P67_26	0	5.54	9%	0.764	0.90	0.74	0.764	0.8	0.833	0.77
	P59_29	0	5.58	9%	0.763	0.90	0.74	0.763	0.8	0.833	0.77
	P53_32	0	5.62	9%	0.761	0.89	0.74	0.761	0.8	0.833	0.77
	P73_8	0	5.87	9%	0.750	0.89	0.73	0.750	0.8	0.833	0.77
	P90_15	0	6.05	10%	0.743	0.88	0.72	0.743	0.7	0.833	0.63
	P94_20	0	6.20	10%	0.736	0.88	0.71	0.736	0.7	0.833	0.63
	P81_24	0	6.33	10%	0.731	0.88	0.71	0.731	0.7	0.833	0.63
	P71_27	0	6.44	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P63_29	0	6.54	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P62_9	0	6.17	10%	0.738	0.88	0.71	0.738	0.7	0.833	0.63
	P77_18	0	6.31	10%	0.732	0.88	0.71	0.732	0.7	0.833	0.63
	P91_24	0	6.43	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P95_28	0	6.53	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P83_31	0	6.62	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P74_34	0	6.70	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P54_10	0	6.42	10%	0.727	0.88	0.70	0.727	0.7	0.833	0.63
	P67_20	0	6.53	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P80_26	0	6.63	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P92_31	0	6.71	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P95_35	0	6.78	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
	P85_37	0	6.85	11%	0.709	0.87	0.68	0.709	0.7	0.833	0.63
	P48_10	0	6.63	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall b'		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
P59_21	0	6.72	11%	0.714	0.87	0.69	0.714	0.7	0.833	0.63
P71_28	0	6.80	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
P82_33	0	6.87	11%	0.708	0.87	0.68	0.708	0.7	0.833	0.63
P93_37	0	6.93	11%	0.705	0.86	0.68	0.705	0.7	0.833	0.63
P96_40	0	6.98	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P43_11	0	6.81	11%	0.710	0.87	0.69	0.710	0.7	0.833	0.63
P53_22	0	6.88	11%	0.707	0.87	0.68	0.707	0.7	0.833	0.63
P63_30	0	6.95	11%	0.704	0.86	0.68	0.704	0.7	0.833	0.63
P74_35	0	7.00	11%	0.702	0.86	0.68	0.702	0.7	0.833	0.63
P84_39	0	7.05	11%	0.700	0.86	0.67	0.700	0.7	0.833	0.63
P94_42	0	7.10	11%	0.698	0.86	0.67	0.698	0.7	0.833	0.63
P39_12	0	6.97	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P48_23	0	7.03	11%	0.701	0.86	0.68	0.701	0.7	0.833	0.63
P58_31	0	7.08	11%	0.699	0.86	0.67	0.699	0.7	0.833	0.63
P67_36	0	7.12	11%	0.697	0.86	0.67	0.697	0.7	0.833	0.63
P76_41	0	7.16	11%	0.695	0.86	0.67	0.695	0.7	0.833	0.63
P85_44	0	7.20	11%	0.694	0.86	0.67	0.694	0.7	0.833	0.63
P82_10	0	0.00	9%	1.000	0.91	0.78	1.000	1	0.833	1
P100_0	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
P88_8	0	0.00	9%	1.000	0.92	0.80	1.000	1	0.833	1
P93_16	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P78_22	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P67_26	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P59_29	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P53_32	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P73_8	0	0.00	9%	1.000	0.91	0.78	1.000	1	0.833	1
P90_15	0	0.00	10%	1.000	0.91	0.78	1.000	1	0.833	1

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall b'		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	Raw	b_{fh}/b Lookup	h_o/h Raw	C_o
Case 2	P94_20	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P81_24	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P71_27	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P63_29	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P62_9	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P77_18	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P91_24	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P95_28	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P83_31	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P74_34	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P54_10	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P67_20	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P80_26	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P92_31	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P95_35	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P85_37	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P48_10	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P59_21	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P71_28	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P82_33	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P93_37	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P96_40	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P43_11	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P53_22	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P63_30	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P74_35	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P84_39	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall b'		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
P94_42	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P39_12	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P48_23	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P58_31	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P67_36	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
P76_41	0	0.00	11%	1.000	0.90	0.74	1.000	1	0.833	1
P85_44	0	0.00	11%	1.000	0.90	0.74	1.000	1	0.833	1
P82_10	0	0.00	9%	1.000	0.91	0.78	1.000	1	0.833	1
P100_0	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
P88_8	0	0.00	9%	1.000	0.92	0.80	1.000	1	0.833	1
P93_16	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P78_22	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P67_26	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P59_29	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P53_32	0	0.00	9%	1.000	0.92	0.79	1.000	1	0.833	1
P73_8	0	0.00	9%	1.000	0.91	0.78	1.000	1	0.833	1
P90_15	0	0.00	10%	1.000	0.91	0.78	1.000	1	0.833	1
P94_20	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
P81_24	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
P71_27	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
P63_29	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
P62_9	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
P77_18	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
P91_24	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
P95_28	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
P83_31	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
P74_34	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall b'		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 3	P54_10	0	0.00	10%	1.000	0.91	0.77	1.000	1	0.833	1
	P67_20	0	0.00	10%	1.000	0.91	0.76	1.000	1	0.833	1
	P80_26	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P92_31	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P95_35	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P85_37	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P48_10	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P59_21	0	0.00	11%	1.000	0.90	0.76	1.000	1	0.833	1
	P71_28	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P82_33	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P93_37	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P96_40	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P43_11	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P53_22	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P63_30	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P74_35	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P84_39	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P94_42	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P39_12	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P48_23	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P58_31	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P67_36	0	0.00	11%	1.000	0.90	0.75	1.000	1	0.833	1
	P76_41	0	0.00	11%	1.000	0.90	0.74	1.000	1	0.833	1
	P85_44	0	0.00	11%	1.000	0.90	0.74	1.000	1	0.833	1

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall c		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 1	P82_10	0	4.10	9.3%	0.752	0.89	0.73	0.752	0.8	0.833	0.77
	P100_0	0	0.00	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
	P88_8	0	3.77	9%	0.772	0.90	0.75	0.772	0.8	0.833	0.77
	P93_16	0	3.82	9%	0.769	0.90	0.75	0.769	0.8	0.833	0.77
	P78_22	0	3.86	9%	0.766	0.90	0.74	0.766	0.8	0.833	0.77
	P67_26	0	3.89	9%	0.764	0.90	0.74	0.764	0.8	0.833	0.77
	P59_29	0	3.92	9%	0.763	0.90	0.74	0.763	0.8	0.833	0.77
	P53_32	0	3.94	9%	0.761	0.89	0.74	0.761	0.8	0.833	0.77
	P73_8	0	4.60	9%	0.750	0.89	0.73	0.750	0.8	0.833	0.77
	P90_15	0	4.74	10%	0.743	0.88	0.72	0.743	0.7	0.833	0.63
	P94_20	0	4.86	10%	0.736	0.88	0.71	0.736	0.7	0.833	0.63
	P81_24	0	4.96	10%	0.731	0.88	0.71	0.731	0.7	0.833	0.63
	P71_27	0	5.04	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P63_29	0	5.12	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P62_9	0	6.55	10%	0.738	0.88	0.71	0.738	0.7	0.833	0.63
	P77_18	0	6.70	10%	0.732	0.88	0.71	0.732	0.7	0.833	0.63
	P91_24	0	6.83	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P95_28	0	6.94	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P83_31	0	7.04	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P74_34	0	7.12	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P54_10	0	8.61	10%	0.727	0.88	0.70	0.727	0.7	0.833	0.63
	P67_20	0	8.76	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P80_26	0	8.89	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P92_31	0	9.00	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P95_35	0	9.10	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
	P85_37	0	9.19	11%	0.709	0.87	0.68	0.709	0.7	0.833	0.63
	P48_10	0	10.75	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall c		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	b_{fh}/b Lookup	h_o/h Raw	C_o
P59_21	0	10.89	11%	0.714	0.87	0.69	0.714	0.7	0.833	0.63
P71_28	0	11.02	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
P82_33	0	11.13	11%	0.708	0.87	0.68	0.708	0.7	0.833	0.63
P93_37	0	11.22	11%	0.705	0.86	0.68	0.705	0.7	0.833	0.63
P96_40	0	11.31	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P43_11	0	12.94	11%	0.710	0.87	0.69	0.710	0.7	0.833	0.63
P53_22	0	13.08	11%	0.707	0.87	0.68	0.707	0.7	0.833	0.63
P63_30	0	13.20	11%	0.704	0.86	0.68	0.704	0.7	0.833	0.63
P74_35	0	13.30	11%	0.702	0.86	0.68	0.702	0.7	0.833	0.63
P84_39	0	13.40	11%	0.700	0.86	0.67	0.700	0.7	0.833	0.63
P94_42	0	13.48	11%	0.698	0.86	0.67	0.698	0.7	0.833	0.63
P39_12	0	15.19	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P48_23	0	15.31	11%	0.701	0.86	0.68	0.701	0.7	0.833	0.63
P58_31	0	15.42	11%	0.699	0.86	0.67	0.699	0.7	0.833	0.63
P67_36	0	15.52	11%	0.697	0.86	0.67	0.697	0.7	0.833	0.63
P76_41	0	15.61	11%	0.695	0.86	0.67	0.695	0.7	0.833	0.63
P85_44	0	15.69	11%	0.694	0.86	0.67	0.694	0.7	0.833	0.63
P82_10	0	6.84	9%	0.585	0.86	0.68	0.585	0.6	0.833	0.57
P100_0	0	0.00	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
P88_8	0	6.60	9%	0.600	0.88	0.70	0.600	0.6	0.833	0.57
P93_16	0	6.77	9%	0.590	0.87	0.69	0.590	0.6	0.833	0.57
P78_22	0	6.91	9%	0.581	0.87	0.69	0.581	0.6	0.833	0.57
P67_26	0	7.03	9%	0.574	0.87	0.68	0.574	0.6	0.833	0.57
P59_29	0	7.13	9%	0.568	0.86	0.68	0.568	0.6	0.833	0.57
P53_32	0	7.22	9%	0.562	0.86	0.68	0.562	0.6	0.833	0.57
P73_8	0	7.50	9%	0.593	0.86	0.68	0.593	0.6	0.833	0.57
P90_15	0	7.86	10%	0.573	0.86	0.66	0.573	0.6	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall c		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	b_{fh}/b Lookup	h_o/h Raw	C_o
Case 2	P94_20	0	8.17	10%	0.556	0.85	0.65	0.556	0.6	0.833	0.57
	P81_24	0	8.44	10%	0.541	0.84	0.64	0.541	0.5	0.833	0.57
	P71_27	0	8.68	10%	0.528	0.84	0.63	0.528	0.5	0.833	0.57
	P63_29	0	8.89	10%	0.517	0.83	0.62	0.517	0.5	0.833	0.57
	P62_9	0	10.11	10%	0.595	0.86	0.67	0.595	0.6	0.833	0.57
	P77_18	0	10.55	10%	0.578	0.85	0.66	0.578	0.6	0.833	0.57
	P91_24	0	10.93	10%	0.563	0.85	0.65	0.563	0.6	0.833	0.57
	P95_28	0	11.26	10%	0.549	0.84	0.64	0.549	0.5	0.833	0.57
	P83_31	0	11.56	11%	0.537	0.84	0.63	0.537	0.5	0.833	0.57
	P74_34	0	11.83	11%	0.526	0.83	0.62	0.526	0.5	0.833	0.57
	P54_10	0	12.72	10%	0.597	0.85	0.66	0.597	0.6	0.833	0.57
	P67_20	0	13.21	10%	0.581	0.85	0.65	0.581	0.6	0.833	0.57
	P80_26	0	13.64	11%	0.567	0.84	0.64	0.567	0.6	0.833	0.57
	P92_31	0	14.03	11%	0.555	0.84	0.63	0.555	0.6	0.833	0.57
	P95_35	0	14.38	11%	0.544	0.83	0.63	0.544	0.5	0.833	0.57
	P85_37	0	14.69	11%	0.534	0.83	0.62	0.534	0.5	0.833	0.57
	P48_10	0	15.32	11%	0.598	0.85	0.65	0.598	0.6	0.833	0.57
	P59_21	0	15.85	11%	0.584	0.84	0.64	0.584	0.6	0.833	0.57
	P71_28	0	16.33	11%	0.571	0.84	0.64	0.571	0.6	0.833	0.57
	P82_33	0	16.76	11%	0.560	0.84	0.63	0.560	0.6	0.833	0.57
	P93_37	0	17.15	11%	0.550	0.83	0.62	0.550	0.5	0.833	0.57
	P96_40	0	17.51	11%	0.540	0.83	0.62	0.540	0.5	0.833	0.57
	P43_11	0	17.91	11%	0.599	0.85	0.65	0.599	0.6	0.833	0.57
	P53_22	0	18.48	11%	0.586	0.84	0.64	0.586	0.6	0.833	0.57
	P63_30	0	18.99	11%	0.575	0.84	0.63	0.575	0.6	0.833	0.57
	P74_35	0	19.46	11%	0.564	0.83	0.63	0.564	0.6	0.833	0.57
	P84_39	0	19.89	11%	0.555	0.83	0.62	0.555	0.6	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall c		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
P94_42	0	20.29	11%	0.546	0.83	0.62	0.546	0.5	0.833	0.57
P39_12	0	20.50	11%	0.600	0.84	0.64	0.600	0.6	0.833	0.57
P48_23	0	21.10	11%	0.588	0.84	0.64	0.588	0.6	0.833	0.57
P58_31	0	21.64	11%	0.577	0.84	0.63	0.577	0.6	0.833	0.57
P67_36	0	22.14	11%	0.568	0.83	0.62	0.568	0.6	0.833	0.57
P76_41	0	22.61	11%	0.559	0.83	0.62	0.559	0.6	0.833	0.57
P85_44	0	23.04	11%	0.550	0.83	0.61	0.550	0.6	0.833	0.57
P82_10	0	6.84	9%	0.585	0.86	0.68	0.585	0.6	0.833	0.57
P100_0	0	0.00	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
P88_8	0	6.60	9%	0.600	0.88	0.70	0.600	0.6	0.833	0.57
P93_16	0	6.77	9%	0.590	0.87	0.69	0.590	0.6	0.833	0.57
P78_22	0	6.91	9%	0.581	0.87	0.69	0.581	0.6	0.833	0.57
P67_26	0	7.03	9%	0.574	0.87	0.68	0.574	0.6	0.833	0.57
P59_29	0	7.13	9%	0.568	0.86	0.68	0.568	0.6	0.833	0.57
P53_32	0	7.22	9%	0.562	0.86	0.68	0.562	0.6	0.833	0.57
P73_8	0	7.50	9%	0.593	0.86	0.68	0.593	0.6	0.833	0.57
P90_15	0	7.86	10%	0.573	0.86	0.66	0.573	0.6	0.833	0.57
P94_20	0	8.17	10%	0.556	0.85	0.65	0.556	0.6	0.833	0.57
P81_24	0	8.44	10%	0.541	0.84	0.64	0.541	0.5	0.833	0.57
P71_27	0	8.68	10%	0.528	0.84	0.63	0.528	0.5	0.833	0.57
P63_29	0	8.89	10%	0.517	0.83	0.62	0.517	0.5	0.833	0.57
P62_9	0	10.11	10%	0.595	0.86	0.67	0.595	0.6	0.833	0.57
P77_18	0	10.55	10%	0.578	0.85	0.66	0.578	0.6	0.833	0.57
P91_24	0	10.93	10%	0.563	0.85	0.65	0.563	0.6	0.833	0.57
P95_28	0	11.26	10%	0.549	0.84	0.64	0.549	0.5	0.833	0.57
P83_31	0	11.56	11%	0.537	0.84	0.63	0.537	0.5	0.833	0.57
P74_34	0	11.83	11%	0.526	0.83	0.62	0.526	0.5	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall c		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid	Wind. wid					b_{fh}/b		h_o/h	C_o
		(ft)	(ft)	a	b	r	Cop	Raw	Lookup	Raw	
Case 3	P54_10	0	12.72	10%	0.597	0.85	0.66	0.597	0.6	0.833	0.57
	P67_20	0	13.21	10%	0.581	0.85	0.65	0.581	0.6	0.833	0.57
	P80_26	0	13.64	11%	0.567	0.84	0.64	0.567	0.6	0.833	0.57
	P92_31	0	14.03	11%	0.555	0.84	0.63	0.555	0.6	0.833	0.57
	P95_35	0	14.38	11%	0.544	0.83	0.63	0.544	0.5	0.833	0.57
	P85_37	0	14.69	11%	0.534	0.83	0.62	0.534	0.5	0.833	0.57
	P48_10	0	15.32	11%	0.598	0.85	0.65	0.598	0.6	0.833	0.57
	P59_21	0	15.85	11%	0.584	0.84	0.64	0.584	0.6	0.833	0.57
	P71_28	0	16.33	11%	0.571	0.84	0.64	0.571	0.6	0.833	0.57
	P82_33	0	16.76	11%	0.560	0.84	0.63	0.560	0.6	0.833	0.57
	P93_37	0	17.15	11%	0.550	0.83	0.62	0.550	0.5	0.833	0.57
	P96_40	0	17.51	11%	0.540	0.83	0.62	0.540	0.5	0.833	0.57
	P43_11	0	17.91	11%	0.599	0.85	0.65	0.599	0.6	0.833	0.57
	P53_22	0	18.48	11%	0.586	0.84	0.64	0.586	0.6	0.833	0.57
	P63_30	0	18.99	11%	0.575	0.84	0.63	0.575	0.6	0.833	0.57
	P74_35	0	19.46	11%	0.564	0.83	0.63	0.564	0.6	0.833	0.57
	P84_39	0	19.89	11%	0.555	0.83	0.62	0.555	0.6	0.833	0.57
	P94_42	0	20.29	11%	0.546	0.83	0.62	0.546	0.5	0.833	0.57
	P39_12	0	20.50	11%	0.600	0.84	0.64	0.600	0.6	0.833	0.57
	P48_23	0	21.10	11%	0.588	0.84	0.64	0.588	0.6	0.833	0.57
	P58_31	0	21.64	11%	0.577	0.84	0.63	0.577	0.6	0.833	0.57
	P67_36	0	22.14	11%	0.568	0.83	0.62	0.568	0.6	0.833	0.57
	P76_41	0	22.61	11%	0.559	0.83	0.62	0.559	0.6	0.833	0.57
	P85_44	0	23.04	11%	0.550	0.83	0.61	0.550	0.6	0.833	0.57
	Smith	0	1.39	10%	0.732	0.88	0.71	0.732	0.7	0.833	0.63

Table B.1 cont'd. Comarison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall d		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 1	P82_10	0	1.63	9.3%	0.752	0.89	0.73	0.752	0.8	0.833	0.77
	P100_0	0	0.00	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
	P88_8	0	1.03	9%	0.772	0.90	0.75	0.772	0.8	0.833	0.77
	P93_16	0	2.56	9%	0.769	0.90	0.75	0.769	0.8	0.833	0.77
	P78_22	0	4.12	9%	0.766	0.90	0.74	0.766	0.8	0.833	0.77
	P67_26	0	5.70	9%	0.764	0.90	0.74	0.764	0.8	0.833	0.77
	P59_29	0	7.30	9%	0.763	0.90	0.74	0.763	0.8	0.833	0.77
	P53_32	0	8.91	9%	0.761	0.89	0.74	0.761	0.8	0.833	0.77
	P73_8	0	1.12	9%	0.750	0.89	0.73	0.750	0.8	0.833	0.77
	P90_15	0	2.85	10%	0.743	0.88	0.72	0.743	0.7	0.833	0.63
	P94_20	0	4.65	10%	0.736	0.88	0.71	0.736	0.7	0.833	0.63
	P81_24	0	6.51	10%	0.731	0.88	0.71	0.731	0.7	0.833	0.63
	P71_27	0	8.42	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P63_29	0	10.37	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P62_9	0	1.18	10%	0.738	0.88	0.71	0.738	0.7	0.833	0.63
	P77_18	0	2.97	10%	0.732	0.88	0.71	0.732	0.7	0.833	0.63
	P91_24	0	4.82	10%	0.726	0.88	0.70	0.726	0.7	0.833	0.63
	P95_28	0	6.72	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P83_31	0	8.66	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P74_34	0	10.64	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P54_10	0	1.23	10%	0.727	0.88	0.70	0.727	0.7	0.833	0.63
	P67_20	0	3.07	10%	0.722	0.87	0.70	0.722	0.7	0.833	0.63
	P80_26	0	4.97	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63
	P92_31	0	6.90	11%	0.715	0.87	0.69	0.715	0.7	0.833	0.63
	P95_35	0	8.87	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
	P85_37	0	10.87	11%	0.709	0.87	0.68	0.709	0.7	0.833	0.63
	P48_10	0	1.27	11%	0.718	0.87	0.69	0.718	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall d		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid	Wind. wid					b_{fh}/b		h_o/h	C_o
	(ft)	(ft)	a	b	r	Cop	Raw	Lookup	Raw	
P59_21	0	3.16	11%	0.714	0.87	0.69	0.714	0.7	0.833	0.63
P71_28	0	5.09	11%	0.711	0.87	0.69	0.711	0.7	0.833	0.63
P82_33	0	7.06	11%	0.708	0.87	0.68	0.708	0.7	0.833	0.63
P93_37	0	9.06	11%	0.705	0.86	0.68	0.705	0.7	0.833	0.63
P96_40	0	11.08	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P43_11	0	1.30	11%	0.710	0.87	0.69	0.710	0.7	0.833	0.63
P53_22	0	3.24	11%	0.707	0.87	0.68	0.707	0.7	0.833	0.63
P63_30	0	5.21	11%	0.704	0.86	0.68	0.704	0.7	0.833	0.63
P74_35	0	7.20	11%	0.702	0.86	0.68	0.702	0.7	0.833	0.63
P84_39	0	9.22	11%	0.700	0.86	0.67	0.700	0.7	0.833	0.63
P94_42	0	11.26	11%	0.698	0.86	0.67	0.698	0.7	0.833	0.63
P39_12	0	1.33	11%	0.703	0.86	0.68	0.703	0.7	0.833	0.63
P48_23	0	3.31	11%	0.701	0.86	0.68	0.701	0.7	0.833	0.63
P58_31	0	5.31	11%	0.699	0.86	0.67	0.699	0.7	0.833	0.63
P67_36	0	7.33	11%	0.697	0.86	0.67	0.697	0.7	0.833	0.63
P76_41	0	9.37	11%	0.695	0.86	0.67	0.695	0.7	0.833	0.63
P85_44	0	11.43	11%	0.694	0.86	0.67	0.694	0.7	0.833	0.63
P82_10	0	2.73	9%	0.585	0.86	0.68	0.585	0.6	0.833	0.57
P100_0	0	0.00	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
P88_8	0	1.80	9%	0.600	0.88	0.70	0.600	0.6	0.833	0.57
P93_16	0	4.53	9%	0.590	0.87	0.69	0.590	0.6	0.833	0.57
P78_22	0	7.38	9%	0.581	0.87	0.69	0.581	0.6	0.833	0.57
P67_26	0	10.30	9%	0.574	0.87	0.68	0.574	0.6	0.833	0.57
P59_29	0	13.29	9%	0.568	0.86	0.68	0.568	0.6	0.833	0.57
P53_32	0	16.33	9%	0.562	0.86	0.68	0.562	0.6	0.833	0.57
P73_8	0	1.83	9%	0.593	0.86	0.68	0.593	0.6	0.833	0.57
P90_15	0	4.72	10%	0.573	0.86	0.66	0.573	0.6	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment									
		Wall d		per HUD/PATH, 6-30				Per Breyer, 10.26			
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	b_{fh}/b Lookup	h_o/h Raw	C_o
Case 2	P94_20	0	7.82	10%	0.556	0.85	0.65	0.556	0.6	0.833	0.57
	P81_24	0	11.09	10%	0.541	0.84	0.64	0.541	0.5	0.833	0.57
	P71_27	0	14.50	10%	0.528	0.84	0.63	0.528	0.5	0.833	0.57
	P63_29	0	18.02	10%	0.517	0.83	0.62	0.517	0.5	0.833	0.57
	P62_9	0	1.82	10%	0.595	0.86	0.67	0.595	0.6	0.833	0.57
	P77_18	0	4.67	10%	0.578	0.85	0.66	0.578	0.6	0.833	0.57
	P91_24	0	7.71	10%	0.563	0.85	0.65	0.563	0.6	0.833	0.57
	P95_28	0	10.90	10%	0.549	0.84	0.64	0.549	0.5	0.833	0.57
	P83_31	0	14.23	11%	0.537	0.84	0.63	0.537	0.5	0.833	0.57
	P74_34	0	17.67	11%	0.526	0.83	0.62	0.526	0.5	0.833	0.57
	P54_10	0	1.81	10%	0.597	0.85	0.66	0.597	0.6	0.833	0.57
	P67_20	0	4.63	10%	0.581	0.85	0.65	0.581	0.6	0.833	0.57
	P80_26	0	7.62	11%	0.567	0.84	0.64	0.567	0.6	0.833	0.57
	P92_31	0	10.76	11%	0.555	0.84	0.63	0.555	0.6	0.833	0.57
	P95_35	0	14.02	11%	0.544	0.83	0.63	0.544	0.5	0.833	0.57
	P85_37	0	17.38	11%	0.534	0.83	0.62	0.534	0.5	0.833	0.57
	P48_10	0	1.81	11%	0.598	0.85	0.65	0.598	0.6	0.833	0.57
	P59_21	0	4.60	11%	0.584	0.84	0.64	0.584	0.6	0.833	0.57
	P71_28	0	7.55	11%	0.571	0.84	0.64	0.571	0.6	0.833	0.57
	P82_33	0	10.64	11%	0.560	0.84	0.63	0.560	0.6	0.833	0.57
	P93_37	0	13.84	11%	0.550	0.83	0.62	0.550	0.5	0.833	0.57
	P96_40	0	17.15	11%	0.540	0.83	0.62	0.540	0.5	0.833	0.57
	P43_11	0	1.80	11%	0.599	0.85	0.65	0.599	0.6	0.833	0.57
	P53_22	0	4.57	11%	0.586	0.84	0.64	0.586	0.6	0.833	0.57
	P63_30	0	7.49	11%	0.575	0.84	0.63	0.575	0.6	0.833	0.57
	P74_35	0	10.54	11%	0.564	0.83	0.63	0.564	0.6	0.833	0.57
	P84_39	0	13.69	11%	0.555	0.83	0.62	0.555	0.6	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

	Perforated Shearwall Stiffness and Strength Adjustment									
	Wall d		per HUD/PATH, 6-30				Per Breyer, 10.26			
Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
P94_42	0	16.95	11%	0.546	0.83	0.62	0.546	0.5	0.833	0.57
P39_12	0	1.80	11%	0.600	0.84	0.64	0.600	0.6	0.833	0.57
P48_23	0	4.55	11%	0.588	0.84	0.64	0.588	0.6	0.833	0.57
P58_31	0	7.44	11%	0.577	0.84	0.63	0.577	0.6	0.833	0.57
P67_36	0	10.45	11%	0.568	0.83	0.62	0.568	0.6	0.833	0.57
P76_41	0	13.57	11%	0.559	0.83	0.62	0.559	0.6	0.833	0.57
P85_44	0	16.78	11%	0.550	0.83	0.61	0.550	0.6	0.833	0.57
P82_10	0	2.73	9%	0.585	0.86	0.68	0.585	0.6	0.833	0.57
P100_0	0	0.00	10%	0.733	0.88	0.71	0.733	0.7	0.833	0.63
P88_8	0	1.80	9%	0.600	0.88	0.70	0.600	0.6	0.833	0.57
P93_16	0	4.53	9%	0.590	0.87	0.69	0.590	0.6	0.833	0.57
P78_22	0	7.38	9%	0.581	0.87	0.69	0.581	0.6	0.833	0.57
P67_26	0	10.30	9%	0.574	0.87	0.68	0.574	0.6	0.833	0.57
P59_29	0	13.29	9%	0.568	0.86	0.68	0.568	0.6	0.833	0.57
P53_32	0	16.33	9%	0.562	0.86	0.68	0.562	0.6	0.833	0.57
P73_8	0	1.83	9%	0.593	0.86	0.68	0.593	0.6	0.833	0.57
P90_15	0	4.72	10%	0.573	0.86	0.66	0.573	0.6	0.833	0.57
P94_20	0	7.82	10%	0.556	0.85	0.65	0.556	0.6	0.833	0.57
P81_24	0	11.09	10%	0.541	0.84	0.64	0.541	0.5	0.833	0.57
P71_27	0	14.50	10%	0.528	0.84	0.63	0.528	0.5	0.833	0.57
P63_29	0	18.02	10%	0.517	0.83	0.62	0.517	0.5	0.833	0.57
P62_9	0	1.82	10%	0.595	0.86	0.67	0.595	0.6	0.833	0.57
P77_18	0	4.67	10%	0.578	0.85	0.66	0.578	0.6	0.833	0.57
P91_24	0	7.71	10%	0.563	0.85	0.65	0.563	0.6	0.833	0.57
P95_28	0	10.90	10%	0.549	0.84	0.64	0.549	0.5	0.833	0.57
P83_31	0	14.23	11%	0.537	0.84	0.63	0.537	0.5	0.833	0.57
P74_34	0	17.67	11%	0.526	0.83	0.62	0.526	0.5	0.833	0.57

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

		Perforated Shearwall Stiffness and Strength Adjustment										
		Wall d		per HUD/PATH, 6-30				Per Breyer, 10.26				
	Case	Door wid (ft)	Wind. wid (ft)	a	b	r	Cop	b_{fh}/b Raw	Lookup	h_o/h Raw	C_o
Case 3	P54_10	0	1.81	10%	0.597	0.85	0.66	0.597	0.6	0.833	0.57
	P67_20	0	4.63	10%	0.581	0.85	0.65	0.581	0.6	0.833	0.57
	P80_26	0	7.62	11%	0.567	0.84	0.64	0.567	0.6	0.833	0.57
	P92_31	0	10.76	11%	0.555	0.84	0.63	0.555	0.6	0.833	0.57
	P95_35	0	14.02	11%	0.544	0.83	0.63	0.544	0.5	0.833	0.57
	P85_37	0	17.38	11%	0.534	0.83	0.62	0.534	0.5	0.833	0.57
	P48_10	0	1.81	11%	0.598	0.85	0.65	0.598	0.6	0.833	0.57
	P59_21	0	4.60	11%	0.584	0.84	0.64	0.584	0.6	0.833	0.57
	P71_28	0	7.55	11%	0.571	0.84	0.64	0.571	0.6	0.833	0.57
	P82_33	0	10.64	11%	0.560	0.84	0.63	0.560	0.6	0.833	0.57
	P93_37	0	13.84	11%	0.550	0.83	0.62	0.550	0.5	0.833	0.57
	P96_40	0	17.15	11%	0.540	0.83	0.62	0.540	0.5	0.833	0.57
	P43_11	0	1.80	11%	0.599	0.85	0.65	0.599	0.6	0.833	0.57
	P53_22	0	4.57	11%	0.586	0.84	0.64	0.586	0.6	0.833	0.57
	P63_30	0	7.49	11%	0.575	0.84	0.63	0.575	0.6	0.833	0.57
	P74_35	0	10.54	11%	0.564	0.83	0.63	0.564	0.6	0.833	0.57
	P84_39	0	13.69	11%	0.555	0.83	0.62	0.555	0.6	0.833	0.57
	P94_42	0	16.95	11%	0.546	0.83	0.62	0.546	0.5	0.833	0.57
	P39_12	0	1.80	11%	0.600	0.84	0.64	0.600	0.6	0.833	0.57
	P48_23	0	4.55	11%	0.588	0.84	0.64	0.588	0.6	0.833	0.57
	P58_31	0	7.44	11%	0.577	0.84	0.63	0.577	0.6	0.833	0.57
	P67_36	0	10.45	11%	0.568	0.83	0.62	0.568	0.6	0.833	0.57
	P76_41	0	13.57	11%	0.559	0.83	0.62	0.559	0.6	0.833	0.57
	P85_44	0	16.78	11%	0.550	0.83	0.61	0.550	0.6	0.833	0.57
	Smith	0	4.70	10%	0.732	0.88	0.71	0.732	0.7	0.833	0.63

Table B.1 cont'd. Comparison of Perforated Shearwall Methods.

Maximum Difference	0.39
Average Difference	0.18

Table B.2. Seismic Mass, Load and Calculated Eccentricity.

	Case	Wall Area (ft^2)	Mass (lbm)	Seismic Load (ASD) (lbf)	Eccentricity ex (ft)	ex (m)	(%)	ey (ft)	ey (m)	(%)
			Cs*0.7	0.08						
Case 1	P82_10	1068	24571	1966	0.34	0.10	1%	0.94	0.29	3%
	P100_0	1024	24750	1980	-5.05	-1.54	16%	3.84	1.17	12%
	P88_8	960	20878	1670	-1.15	-0.35	4%	1.73	0.53	6%
	P93_16	1065	23361	1869	-4.00	-1.22	12%	5.26	1.60	15%
	P78_22	1170	25843	2067	-6.69	-2.04	21%	8.57	2.61	21%
	P67_26	1275	28325	2266	-9.28	-2.83	29%	11.78	3.59	25%
	P59_29	1380	30808	2465	-11.77	-3.59	37%	14.96	4.56	28%
	P53_32	1485	33290	2663	-14.21	-4.33	44%	18.11	5.52	30%
	P73_8	1065	24597	1968	2.51	0.77	7%	-1.12	-0.34	4%
	P90_15	1170	27593	2207	-0.54	-0.16	1%	2.63	0.80	8%
	P94_20	1275	30589	2447	-3.42	-1.04	9%	6.10	1.86	15%
	P81_24	1380	33585	2687	-6.16	-1.88	16%	9.44	2.88	20%
	P71_27	1485	36581	2926	-8.81	-2.69	23%	12.71	3.88	23%
	P63_29	1590	39577	3166	-11.39	-3.47	30%	15.95	4.86	26%
	P62_9	1170	27963	2237	6.34	1.93	14%	-4.01	-1.22	14%
	P77_18	1275	30959	2477	3.45	1.05	8%	0.02	0.01	0%
	P91_24	1380	33955	2716	0.76	0.23	2%	3.64	1.11	9%
	P95_28	1485	36951	2956	-1.78	-0.54	4%	7.05	2.15	15%
	P83_31	1590	39947	3196	-4.21	-1.28	9%	10.35	3.16	19%
	P74_34	1695	42943	3435	-6.54	-1.99	15%	13.60	4.14	22%
	P54_10	1275	31330	2506	10.00	3.05	19%	-6.87	-2.10	25%
	P67_20	1380	34326	2746	7.21	2.20	14%	-2.50	-0.76	7%
	P80_26	1485	37322	2986	4.63	1.41	9%	1.29	0.39	3%
	P92_31	1590	40318	3225	2.20	0.67	4%	4.80	1.46	10%
	P95_35	1695	43314	3465	-0.10	-0.03	0%	8.14	2.48	15%
	P85_37	1800	46310	3705	-2.30	-0.70	4%	11.39	3.47	19%
	P48_10	1380	34696	2776	13.56	4.13	23%	-9.71	-2.96	35%

Table B.2 cont'd. Seismic Mass, Load and Calculated Eccentricity.

	Cs*0.7	0.08							
Case	**Wall Area**	**Mass**	**Seismic Load (ASD)**	**Eccentricity**					
	(ft^2)	**(lbm)**	**(lbf)**	**ex (ft)**	**ex (m)**	**(%)**	**ey (ft)**	**ey (m)**	**(%)**
P59_21	1485	37692	3015	10.83	3.30	19%	-4.98	-1.52	14%
P71_28	1590	40688	3255	8.30	2.53	14%	-0.98	-0.30	2%
P82_33	1695	43684	3495	5.94	1.81	10%	2.64	0.80	6%
P93_37	1800	46680	3734	3.72	1.13	6%	6.04	1.84	11%
P96_40	1905	49676	3974	1.60	0.49	3%	9.32	2.84	15%
P43_11	1485	38063	3045	17.05	5.20	26%	-12.54	-3.82	45%
P53_22	1590	41059	3285	14.36	4.38	22%	-7.41	-2.26	21%
P63_30	1695	44055	3524	11.86	3.62	18%	-3.19	-0.97	8%
P74_35	1800	47051	3764	9.54	2.91	15%	0.56	0.17	1%
P84_39	1905	50047	4004	7.36	2.24	11%	4.03	1.23	7%
P94_42	2010	53043	4243	5.29	1.61	8%	7.35	2.24	12%
P39_12	1590	41429	3314	20.50	6.25	29%	-15.35	-4.68	55%
P48_23	1695	44425	3554	17.82	5.43	25%	-9.81	-2.99	28%
P58_31	1800	47421	3794	15.35	4.68	22%	-5.35	-1.63	13%
P67_36	1905	50417	4033	13.05	3.98	18%	-1.46	-0.45	3%
P76_41	2010	53413	4273	10.88	3.32	15%	2.10	0.64	4%
P85_44	2115	56409	4513	8.84	2.69	12%	5.46	1.66	9%
P82_10	1068	24571	1966	-2.97	-0.91	8%	3.11	0.95	10%
P100_0	1024	24750	1980	-14.33	-4.37	45%	7.56	2.30	24%
P88_8	960	20878	1670	-4.29	-1.31	13%	3.54	1.08	13%
P93_16	1065	23361	1869	-7.46	-2.28	23%	6.91	2.11	20%
P78_22	1170	25843	2067	-10.47	-3.19	33%	10.13	3.09	25%
P67_26	1275	28325	2266	-13.35	-4.07	42%	13.29	4.05	28%
P59_29	1380	30808	2465	-16.14	-4.92	50%	16.43	5.01	30%
P53_32	1485	33290	2663	-18.86	-5.75	59%	19.56	5.96	32%
P73_8	1065	24597	1968	-0.54	-0.16	1%	1.17	0.36	4%
P90_15	1170	27593	2207	-4.06	-1.24	11%	4.82	1.47	14%

Table B.2 cont'd. Seismic Mass, Load and Calculated Eccentricity.

		Cs*0.7	0.08							
	Case	**Wall Area**	**Mass**	**Seismic Load (ASD)**	**Eccentricity**					
		(ft^2)	**(lbm)**	**(lbf)**	**ex (ft)**	**ex (m)**	**(%)**	**ey (ft)**	**ey (m)**	**(%)**
Case 2	P94_20	1275	30589	2447	-7.41	-2.26	19%	8.25	2.51	20%
	P81_24	1380	33585	2687	-10.63	-3.24	28%	11.57	3.53	24%
	P71_27	1485	36581	2926	-13.75	-4.19	36%	14.85	4.53	27%
	P63_29	1590	39577	3166	-16.79	-5.12	44%	18.10	5.52	30%
	P62_9	1170	27963	2237	3.52	1.07	8%	-1.39	-0.42	5%
	P77_18	1275	30959	2477	0.28	0.09	1%	2.45	0.75	7%
	P91_24	1380	33955	2716	-2.76	-0.84	6%	5.97	1.82	15%
	P95_28	1485	36951	2956	-5.65	-1.72	13%	9.32	2.84	20%
	P83_31	1590	39947	3196	-8.43	-2.57	19%	12.59	3.84	23%
	P74_34	1695	42943	3435	-11.12	-3.39	25%	15.82	4.82	26%
	P54_10	1275	31330	2506	7.33	2.23	14%	-3.90	-1.19	14%
	P67_20	1380	34326	2746	4.26	1.30	8%	0.18	0.05	1%
	P80_26	1485	37322	2986	1.39	0.42	3%	3.80	1.16	9%
	P92_31	1590	40318	3225	-1.31	-0.40	3%	7.21	2.20	15%
	P95_35	1695	43314	3465	-3.89	-1.18	8%	10.49	3.20	19%
	P85_37	1800	46310	3705	-6.36	-1.94	12%	13.70	4.18	23%
	P48_10	1380	34696	2776	10.99	3.35	19%	-6.37	-1.94	23%
	P59_21	1485	37692	3015	8.02	2.44	14%	-2.04	-0.62	6%
	P71_28	1590	40688	3255	5.26	1.60	9%	1.73	0.53	4%
	P82_33	1695	43684	3495	2.67	0.81	5%	5.20	1.59	11%
	P93_37	1800	46680	3734	0.22	0.07	0%	8.51	2.59	16%
	P96_40	1905	49676	3974	-2.12	-0.65	4%	11.73	3.57	19%
	P43_11	1485	38063	3045	14.55	4.43	22%	-8.83	-2.69	32%
	P53_22	1590	41059	3285	11.65	3.55	18%	-4.20	-1.28	12%
	P63_30	1695	44055	3524	8.96	2.73	14%	-0.27	-0.08	1%
	P74_35	1800	47051	3764	6.44	1.96	10%	3.28	1.00	7%
	P84_39	1905	50047	4004	4.07	1.24	6%	6.63	2.02	12%

Table B.2 cont'd. Seismic Mass, Load and Calculated Eccentricity.

	Cs*0.7	0.08							
Case	**Wall Area**	**Mass**	**Seismic Load (ASD)**	**Eccentricity**					
	(ft^2)	**(lbm)**	**(lbf)**	**ex (ft)**	**ex (m)**	**(%)**	**ey (ft)**	**ey (m)**	**(%)**
P94_42	2010	53043	4243	1.81	0.55	3%	9.86	3.00	16%
P39_12	1590	41429	3314	18.04	5.50	25%	-11.26	-3.43	40%
P48_23	1695	44425	3554	15.19	4.63	21%	-6.33	-1.93	18%
P58_31	1800	47421	3794	12.55	3.82	18%	-2.23	-0.68	5%
P67_36	1905	50417	4033	10.07	3.07	14%	1.43	0.44	3%
P76_41	2010	53413	4273	7.74	2.36	11%	4.83	1.47	9%
P85_44	2115	56409	4513	5.54	1.69	8%	8.08	2.46	13%
P82_10	1068	24571	1966	-2.83	-0.86	8%	-7.01	-2.14	23%
P100_0	1024	24750	1980	-14.11	-4.30	44%	0.31	0.10	1%
P88_8	960	20878	1670	-4.13	-1.26	13%	-5.90	-1.80	21%
P93_16	1065	23361	1869	-22.84	-6.96	71%	6.14	1.87	18%
P78_22	1170	25843	2067	-27.78	-8.47	87%	9.55	2.91	23%
P67_26	1275	28325	2266	-32.62	-9.94	102%	12.83	3.91	27%
P59_29	1380	30808	2465	-37.37	-11.39	117%	16.04	4.89	30%
P53_32	1485	33290	2663	-42.05	-12.82	131%	19.24	5.86	32%
P73_8	1065	24597	1968	-0.40	-0.12	1%	-10.67	-3.25	38%
P90_15	1170	27593	2207	-3.91	-1.19	10%	-4.04	-1.23	12%
P94_20	1275	30589	2447	-22.12	-6.74	57%	7.50	2.29	18%
P81_24	1380	33585	2687	-27.41	-8.35	71%	10.97	3.35	23%
P71_27	1485	36581	2926	-32.66	-9.96	85%	14.36	4.38	26%
P63_29	1590	39577	3166	-37.90	-11.55	98%	17.68	5.39	29%
P62_9	1170	27963	2237	3.63	1.11	8%	-15.60	-4.75	56%
P77_18	1275	30959	2477	0.41	0.12	1%	-8.04	-2.45	23%
P91_24	1380	33955	2716	-2.62	-0.80	6%	-2.20	-0.67	5%
P95_28	1485	36951	2956	-18.18	-5.54	40%	8.62	2.63	18%
P83_31	1590	39947	3196	-22.35	-6.81	50%	12.02	3.66	22%
P74_34	1695	42943	3435	-26.46	-8.06	59%	15.34	4.67	25%

Table B.2 cont'd. Seismic Mass, Load and Calculated Eccentricity.

		Cs*0.7	0.08							
	Case	**Wall Area** (ft^2)	**Mass** (lbm)	**Seismic Load (ASD)** (lbf)	**Eccentricity** ex (ft)	ex (m)	(%)	ey (ft)	ey (m)	(%)
Case 3	P54_10	1275	31330	2506	7.42	2.26	14%	-20.47	-6.24	73%
	P67_20	1380	34326	2746	4.36	1.33	8%	-11.95	-3.64	35%
	P80_26	1485	37322	2986	1.51	0.46	3%	-5.55	-1.69	14%
	P92_31	1590	40318	3225	-1.19	-0.36	2%	-0.30	-0.09	1%
	P95_35	1695	43314	3465	-26.58	-8.10	51%	9.87	3.01	18%
	P85_37	1800	46310	3705	-31.23	-9.52	60%	13.19	4.02	22%
	P48_10	1380	34696	2776	11.07	3.37	19%	-25.30	-7.71	90%
	P59_21	1485	37692	3015	8.11	2.47	14%	-15.79	-4.81	46%
	P71_28	1590	40688	3255	5.36	1.63	9%	-8.82	-2.69	21%
	P82_33	1695	43684	3495	2.78	0.85	5%	-3.21	-0.98	7%
	P93_37	1800	46680	3734	0.34	0.10	1%	1.60	0.49	3%
	P96_40	1905	49676	3974	-22.08	-6.73	38%	11.15	3.40	18%
	P43_11	1485	38063	3045	14.62	4.46	23%	-30.09	-9.17	107%
	P53_22	1590	41059	3285	11.73	3.57	18%	-19.57	-5.97	57%
	P63_30	1695	44055	3524	9.05	2.76	14%	-12.00	-3.66	29%
	P74_35	1800	47051	3764	6.54	1.99	10%	-6.03	-1.84	13%
	P84_39	1905	50047	4004	4.17	1.27	6%	-0.99	-0.30	2%
	P94_42	2010	53043	4243	1.91	0.58	3%	3.47	1.06	6%
	P39_12	1590	41429	3314	18.11	5.52	25%	-34.87	-10.63	125%
	P48_23	1695	44425	3554	15.26	4.65	21%	-23.31	-7.11	67%
	P58_31	1800	47421	3794	12.63	3.85	18%	-15.13	-4.61	37%
	P67_36	1905	50417	4033	10.16	3.10	14%	-8.78	-2.68	18%
	P76_41	2010	53413	4273	7.83	2.39	11%	-3.50	-1.07	6%
	P85_44	2115	56409	4513	5.63	1.72	8%	1.12	0.34	2%
	Smith	1160	28162	4800	4.38	1.34	10%	-2.50	-0.76	9%

Table B.2 cont'd. Seismic Mass, Load and Calculated Eccentricity.

		Cs*0.7	0.08						
Case	**Wall Area**	**Mass**	**Seismic Load (ASD)**	**Eccentricity**					
	(ft^2)	**(lbm)**	**(lbf)**	**ex (ft)**	**ex (m)**	**(%)**	**ey (ft)**	**ey (m)**	**(%)**
					Min.	0%		Min.	0%
					Max.	131%		Max.	125%

Table B.3. WFSFD Model Parameters.

	h=	8 ft	Door ht., ft=	6.67			Roof wt=	16.83	psf		Wall wt=		7.3 psf		
Case	**a**		**b**		**c**		**d**		**R**	**Cp**	**Pitch**	**Net Floor Area**		
	(inches)	**(ft)**	**(inches)**	**(ft)**	**inches**	**(ft)**	**(inches)**	**(ft)**			**(n:12)**	**(inches^2)**	**(m^2)**	**(ft^2)**
P82_10	439.82	36.652	361.08	30.090	198	16.500	79	6.583	0.82	10%	0/12	143168	92	994
P100_0	384	32.000	384.00	32.000	0	0.000	0.00	0.000	1.00	0%	0/12	147456	95	1024
P88_8	384	32.000	336.00	28.000	198	16.500	53.92	4.493	0.88	8%	0/12	118348	76	822
P93_16	384	32.000	414.74	34.562	198	16.500	132.66	11.055	0.93	16%	0/12	132994	86	924
P78_22	384	32.000	493.48	41.123	198	16.500	211.40	17.617	0.78	22%	0/12	147639	95	1025
P67_26	384	32.000	572.22	47.685	198	16.500	290.14	24.178	0.67	26%	0/12	162285	105	1127
P59_29	384	32.000	650.96	54.247	198	16.500	368.88	30.740	0.59	29%	0/12	176931	114	1229
P53_32	384	32.000	729.70	60.808	198	16.500	447.62	37.302	0.53	32%	0/12	191576	124	1330
P73_8	462.74	38.562	336.00	28.000	221	18.410	53.92	4.493	0.73	8%	0/12	143569	93	997
P90_15	462.74	38.562	414.74	34.562	221	18.410	132.66	11.055	0.90	15%	0/12	162610	105	1129
P94_20	462.74	38.562	493.48	41.123	221	18.410	211.4	17.617	0.94	20%	0/12	181651	117	1261
P81_24	462.74	38.562	572.22	47.685	221	18.410	290.14	24.178	0.81	24%	0/12	200692	129	1394
P71_27	462.74	38.562	650.96	54.247	221	18.410	368.88	30.740	0.71	27%	0/12	219732	142	1526
P63_29	462.74	38.562	729.70	60.808	221	18.410	447.62	37.302	0.63	29%	0/12	238773	154	1658
P62_9	541.48	45.123	336.00	28.000	300	24.972	53.92	4.493	0.62	9%	0/12	165780	107	1151
P77_18	541.48	45.123	414.74	34.562	300	24.972	132.66	11.055	0.77	18%	0/12	184821	119	1283

Table B.3 cont'd. WFSFD Model Parameters.

		h=	8 **ft**	**Door ht., ft=**	**6.67**			**Roof wt=**	16.83	psf		**Wall wt=**		7.3 psf	
	Case	**a**		**b**		**c**		**d**		**R**	**Cp**	**Pitch**	**Net Floor Area**		
		(inches)	**(ft)**	**(inches)**	**(ft)**	**inches**	**(ft)**	**(inches)**	**(ft)**			**(n:12)**	**(inches^2)**	**(m^2)**	**(ft^2)**
Case 1	P91_24	541.48	45.123	493.48	41.123	300	24.972	211.4	17.617	0.91	24%	0/12	203862	132	1416
	P95_28	541.48	45.123	572.22	47.685	300	24.972	290.14	24.178	0.95	28%	0/12	222903	144	1548
	P83_31	541.48	45.123	650.96	54.247	300	24.972	368.88	30.740	0.83	31%	0/12	241943	156	1680
	P74_34	541.48	45.123	729.70	60.808	300	24.972	447.62	37.302	0.74	34%	0/12	260984	168	1812
	P54_10	620.22	51.685	336.00	28.000	378	31.533	53.92	4.493	0.54	10%	0/12	187991	121	1305
	P67_20	620.22	51.685	414.74	34.562	378	31.533	132.66	11.055	0.67	20%	0/12	207032	134	1438
	P80_26	620.22	51.685	493.48	41.123	378	31.533	211.4	17.617	0.80	26%	0/12	226073	146	1570
	P92_31	620.22	51.685	572.22	47.685	378	31.533	290.14	24.178	0.92	31%	0/12	245114	158	1702
	P95_35	620.22	51.685	650.96	54.247	378	31.533	368.88	30.740	0.95	35%	0/12	264155	170	1834
	P85_37	620.22	51.685	729.70	60.808	378	31.533	447.62	37.302	0.85	37%	0/12	283195	183	1967
	P48_10	698.96	58.247	336.00	28.000	457	38.095	53.92	4.493	0.48	10%	0/12	210202	136	1460
	P59_21	698.96	58.247	414.74	34.562	457	38.095	132.66	11.055	0.59	21%	0/12	229243	148	1592
	P71_28	698.96	58.247	493.48	41.123	457	38.095	211.4	17.617	0.71	28%	0/12	248284	160	1724
	P82_33	698.96	58.247	572.22	47.685	457	38.095	290.14	24.178	0.82	33%	0/12	267325	172	1856
	P93_37	698.96	58.247	650.96	54.247	457	38.095	368.88	30.740	0.93	37%	0/12	286366	185	1989

Table B.3 cont'd. WFSFD Model Parameters.

h= 8 ft | Door ht., ft= 6.67 | Roof wt= 16.83 psf | Wall wt= 7.3 psf

Case	a		b		c		d		R	Cp	Pitch	Net Floor Area		
	(inches)	(ft)	(inches)	(ft)	inches	(ft)	(inches)	(ft)			(n:12)	(inches^2)	(m^2)	(ft^2)
P96_40	698.96	58.247	729.70	60.808	457	38.095	447.62	37.302	0.96	40%	0/12	305406	197	*2121*
P43_11	777.70	64.808	336.00	28.000	536	44.657	53.92	4.493	0.43	11%	0/12	232413	150	*1614*
P53_22	777.70	64.808	414.74	34.562	536	44.657	132.66	11.055	0.53	22%	0/12	251454	162	*1746*
P63_30	777.70	64.808	493.48	41.123	536	44.657	211.4	17.617	0.63	30%	0/12	270495	175	*1878*
P74_35	777.70	64.808	572.22	47.685	536	44.657	290.14	24.178	0.74	35%	0/12	289536	187	*2011*
P84_39	777.70	64.808	650.96	54.247	536	44.657	368.88	30.740	0.84	39%	0/12	308577	199	2143
P94_42	777.70	64.808	729.70	60.808	536	44.657	447.62	37.302	0.94	42%	0/12	327617	211	*2275*
P39_12	856.44	71.370	336.00	28.000	615	51.218	53.92	4.493	0.39	12%	0/12	254624	164	*1768*
P48_23	856.44	71.370	414.74	34.562	615	51.218	132.66	11.055	0.48	23%	0/12	273665	177	*1900*
P58_31	856.44	71.370	493.48	41.123	615	51.218	211.4	17.617	0.58	31%	0/12	292706	189	*2033*
P67_36	856.44	71.370	572.22	47.685	615	51.218	290.14	24.178	0.67	36%	0/12	311747	201	*2165*
P76_41	856.44	71.370	650.96	54.247	615	51.218	368.88	30.740	0.76	41%	0/12	330788	213	*2297*
P85_44	856.44	71.370	729.70	60.808	615	51.218	447.62	37.302	0.85	44%	0/12	349829	226	*2429*
P82_10	439.82	36.652	361.08	30.090	198	16.500	79	6.583	0.82	10%	0/12	143168	92	994
P100_0	384	32.000	384.00	32.000	0	0.000	0.00	0.000	1.00	0%	0/12	147456	95	*1024*

Table B.3 cont'd. WFSFD Model Parameters.

	h=	8 ft	Door ht., ft=	6.67	Roof wt=	16.83	psf			Wall wt=	7.3 psf		
Case	**a**		**b**		**c**		**d**		**R**	**Cp**	**Pitch**	**Net Floor Area**		
	(inches)	**(ft)**	**(inches)**	**(ft)**	**inches**	**(ft)**	**(inches)**	**(ft)**			**(n:12)**	**(inches^2)**	**(m^2)**	**(ft^2)**
P88_8	384	32.000	336.00	28.000	198	16.500	53.92	4.493	0.88	8%	0/12	118348	76	822
P93_16	384	32.000	414.74	34.562	198	16.500	132.66	11.055	0.93	16%	0/12	132994	86	924
P78_22	384	32.000	493.48	41.123	198	16.500	211.40	17.617	0.78	22%	0/12	147639	95	1025
P67_26	384	32.000	572.22	47.685	198	16.500	290.14	24.178	0.67	26%	0/12	162285	105	1127
P59_29	384	32.000	650.96	54.247	198	16.500	368.88	30.740	0.59	29%	0/12	176931	114	1229
P53_32	384	32.000	729.70	60.808	198	16.500	447.62	37.302	0.53	32%	0/12	191576	124	1330
P73_8	462.74	38.562	336.00	28.000	221	18.410	53.92	4.493	0.73	8%	0/12	143569	93	997
P90_15	462.74	38.562	414.74	34.562	221	18.410	132.66	11.055	0.90	15%	0/12	162610	105	1129
P94_20	462.74	38.562	493.48	41.123	221	18.410	211.4	17.617	0.94	20%	0/12	181651	117	1261
P81_24	462.74	38.562	572.22	47.685	221	18.410	290.14	24.178	0.81	24%	0/12	200692	129	1394
P71_27	462.74	38.562	650.96	54.247	221	18.410	368.88	30.740	0.71	27%	0/12	219732	142	1526
P63_29	462.74	38.562	729.70	60.808	221	18.410	447.62	37.302	0.63	29%	0/12	238773	154	1658
P62_9	541.48	45.123	336.00	28.000	300	24.972	53.92	4.493	0.62	9%	0/12	165780	107	1151
P77_18	541.48	45.123	414.74	34.562	300	24.972	132.66	11.055	0.77	18%	0/12	184821	119	1283
P91_24	541.48	45.123	493.48	41.123	300	24.972	211.4	17.617	0.91	24%	0/12	203862	132	1416

Table B.3 cont'd. WFSFD Model Parameters.

	h=	8 ft	Door ht., ft=		6.67		Roof wt=		16.83	psf	Wall wt=			7.3 psf
	Case	**a**		**b**		**c**		**d**		**R**	**Cp**	**Pitch**	**Net Floor Area**		
		(inches)	(ft)	(inches)	(ft)	inches	(ft)	(inches)	(ft)			(n:12)	($inches^2$)	(m^2)	(ft^2)
Case 2	P95_28	541.48	45.123	572.22	47.685	300	24.972	290.14	24.178	0.95	28%	0/12	222903	144	1548
	P83_31	541.48	45.123	650.96	54.247	300	24.972	368.88	30.740	0.83	31%	0/12	241943	156	1680
	P74_34	541.48	45.123	729.70	60.808	300	24.972	447.62	37.302	0.74	34%	0/12	260984	168	1812
	P54_10	620.22	51.685	336.00	28.000	378	31.533	53.92	4.493	0.54	10%	0/12	187991	121	1305
	P67_20	620.22	51.685	414.74	34.562	378	31.533	132.66	11.055	0.67	20%	0/12	207032	134	1438
	P80_26	620.22	51.685	493.48	41.123	378	31.533	211.4	17.617	0.80	26%	0/12	226073	146	1570
	P92_31	620.22	51.685	572.22	47.685	378	31.533	290.14	24.178	0.92	31%	0/12	245114	158	1702
	P95_35	620.22	51.685	650.96	54.247	378	31.533	368.88	30.740	0.95	35%	0/12	264155	170	1834
	P85_37	620.22	51.685	729.70	60.808	378	31.533	447.62	37.302	0.85	37%	0/12	283195	183	1967
	P48_10	698.96	58.247	336.00	28.000	457	38.095	53.92	4.493	0.48	10%	0/12	210202	136	1460
	P59_21	698.96	58.247	414.74	34.562	457	38.095	132.66	11.055	0.59	21%	0/12	229243	148	1592
	P71_28	698.96	58.247	493.48	41.123	457	38.095	211.4	17.617	0.71	28%	0/12	248284	160	1724
	P82_33	698.96	58.247	572.22	47.685	457	38.095	290.14	24.178	0.82	33%	0/12	267325	172	1856
	P93_37	698.96	58.247	650.96	54.247	457	38.095	368.88	30.740	0.93	37%	0/12	286366	185	1989
	P96_40	698.96	58.247	729.70	60.808	457	38.095	447.62	37.302	0.96	40%	0/12	305406	197	2121

Table B.3 cont'd. WFSFD Model Parameters.

	h=	8 ft	Door ht., ft=			6.67	Roof wt=		16.83	psf	Wall wt=			7.3 psf
Case	**a**		**b**		**c**		**d**		**R**	**Cp**	**Pitch**	**Net Floor Area**		
	(inches)	**(ft)**	**(inches)**	**(ft)**	**inches**	**(ft)**	**(inches)**	**(ft)**			**(n:12)**	**(inches^2)**	**(m^2)**	**(ft^2)**
P43_11	777.70	64.808	336.00	28.000	536	44.657	53.92	4.493	0.43	11%	0/12	232413	150	1614
P53_22	777.70	64.808	414.74	34.562	536	44.657	132.66	11.055	0.53	22%	0/12	251454	162	1746
P63_30	777.70	64.808	493.48	41.123	536	44.657	211.4	17.617	0.63	30%	0/12	270495	175	1878
P74_35	777.70	64.808	572.22	47.685	536	44.657	290.14	24.178	0.74	35%	0/12	289536	187	2011
P84_39	777.70	64.808	650.96	54.247	536	44.657	368.88	30.740	0.84	39%	0/12	308577	199	2143
P94_42	777.70	64.808	729.70	60.808	536	44.657	447.62	37.302	0.94	42%	0/12	327617	211	2275
P39_12	856.44	71.370	336.00	28.000	615	51.218	53.92	4.493	0.39	12%	0/12	254624	164	1768
P48_23	856.44	71.370	414.74	34.562	615	51.218	132.66	11.055	0.48	23%	0/12	273665	177	1900
P58_31	856.44	71.370	493.48	41.123	615	51.218	211.4	17.617	0.58	31%	0/12	292706	189	2033
P67_36	856.44	71.370	572.22	47.685	615	51.218	290.14	24.178	0.67	36%	0/12	311747	201	2165
P76_41	856.44	71.370	650.96	54.247	615	51.218	368.88	30.740	0.76	41%	0/12	330788	213	2297
P85_44	856.44	71.370	729.70	60.808	615	51.218	447.62	37.302	0.85	44%	0/12	349829	226	2429
P82_10	439.82	36.652	361.08	30.090	198	16.500	79	6.583	0.82	10%	0/12	143168	92	994
P100_0	384	32.000	384.00	32.000	0	0.000	0.00	0.000	1.00	0%	0/12	147456	95	1024
P88_8	384	32.000	336.00	28.000	198	16.500	53.92	4.493	0.88	8%	0/12	118348	76	822

Table B.3 cont'd. WFSFD Model Parameters.

	h=	8 ft		Door ht., ft=		6.67	Roof wt=		16.83	psf	Wall wt=		7.3	psf
Case	a		b		c		d		R	Cp	Pitch	Net Floor Area		
	(inches)	(ft)	(inches)	(ft)	inches	(ft)	(inches)	(ft)			(n:12)	($inches^2$)	(m^2)	(ft^2)
P93_16	384	32.000	414.74	34.562	198	16.500	132.66	11.055	0.93	16%	0/12	132994	86	924
P78_22	384	32.000	493.48	41.123	198	16.500	211.40	17.617	0.78	22%	0/12	147639	95	1025
P67_26	384	32.000	572.22	47.685	198	16.500	290.14	24.178	0.67	26%	0/12	162285	105	1127
P59_29	384	32.000	650.96	54.247	198	16.500	368.88	30.740	0.59	29%	0/12	176931	114	1229
P53_32	384	32.000	729.70	60.808	198	16.500	447.62	37.302	0.53	32%	0/12	191576	124	1330
P73_8	462.74	38.562	336.00	28.000	221	18.410	53.92	4.493	0.73	8%	0/12	143569	93	997
P90_15	462.74	38.562	414.74	34.562	221	18.410	132.66	11.055	0.90	15%	0/12	162610	105	1129
P94_20	462.74	38.562	493.48	41.123	221	18.410	211.4	17.617	0.94	20%	0/12	181651	117	1261
P81_24	462.74	38.562	572.22	47.685	221	18.410	290.14	24.178	0.81	24%	0/12	200692	129	1394
P71_27	462.74	38.562	650.96	54.247	221	18.410	368.88	30.740	0.71	27%	0/12	219732	142	1526
P63_29	462.74	38.562	729.70	60.808	221	18.410	447.62	37.302	0.63	29%	0/12	238773	154	1658
P62_9	541.48	45.123	336.00	28.000	300	24.972	53.92	4.493	0.62	9%	0/12	165780	107	1151
P77_18	541.48	45.123	414.74	34.562	300	24.972	132.66	11.055	0.77	18%	0/12	184821	119	1283
P91_24	541.48	45.123	493.48	41.123	300	24.972	211.4	17.617	0.91	24%	0/12	203862	132	1416
P95_28	541.48	45.123	572.22	47.685	300	24.972	290.14	24.178	0.95	28%	0/12	222903	144	1548

Table B.3 cont'd. WFSFD Model Parameters.

		h=	8 ft	Door ht., ft=	6.67			Roof wt=	16.83	psf		Wall wt=		7.3 psf	
	Case	**a**		**b**		**c**		**d**		**R**	**Cp**	**Pitch**	**Net Floor Area**		
		(inches)	**(ft)**	**(inches)**	**(ft)**	**inches**	**(ft)**	**(inches)**	**(ft)**			**(n:12)**	**(inches^2)**	**(m^2)**	**(ft^2)**
Case 3	P83_31	541.48	45.123	650.96	54.247	300	24.972	368.88	30.740	0.83	31%	0/12	241943	156	1680
	P74_34	541.48	45.123	729.70	60.808	300	24.972	447.62	37.302	0.74	34%	0/12	260984	168	1812
	P54_10	620.22	51.685	336.00	28.000	378	31.533	53.92	4.493	0.54	10%	0/12	187991	121	1305
	P67_20	620.22	51.685	414.74	34.562	378	31.533	132.66	11.055	0.67	20%	0/12	207032	134	1438
	P80_26	620.22	51.685	493.48	41.123	378	31.533	211.4	17.617	0.80	26%	0/12	226073	146	1570
	P92_31	620.22	51.685	572.22	47.685	378	31.533	290.14	24.178	0.92	31%	0/12	245114	158	1702
	P95_35	620.22	51.685	650.96	54.247	378	31.533	368.88	30.740	0.95	35%	0/12	264155	170	1834
	P85_37	620.22	51.685	729.70	60.808	378	31.533	447.62	37.302	0.85	37%	0/12	283195	183	1967
	P48_10	698.96	58.247	336.00	28.000	457	38.095	53.92	4.493	0.48	10%	0/12	210202	136	1460
	P59_21	698.96	58.247	414.74	34.562	457	38.095	132.66	11.055	0.59	21%	0/12	229243	148	1592
	P71_28	698.96	58.247	493.48	41.123	457	38.095	211.4	17.617	0.71	28%	0/12	248284	160	1724
	P82_33	698.96	58.247	572.22	47.685	457	38.095	290.14	24.178	0.82	33%	0/12	267325	172	1856
	P93_37	698.96	58.247	650.96	54.247	457	38.095	368.88	30.740	0.93	37%	0/12	286366	185	1989
	P96_40	698.96	58.247	729.70	60.808	457	38.095	447.62	37.302	0.96	40%	0/12	305406	197	2121
	P43_11	777.70	64.808	336.00	28.000	536	44.657	53.92	4.493	0.43	11%	0/12	232413	150	1614

Table B.3 cont'd. WFSFD Model Parameters.

	h=	8 **ft**	**Door ht., ft=**	**6.67**			**Roof wt=**	16.83	psf		**Wall wt=**		7.3	psf
Case	**a**		**b**		**c**		**d**		**R**	**Cp**	**Pitch**	**Net Floor Area**		
	(inches)	**(ft)**	**(inches)**	**(ft)**	**inches**	**(ft)**	**(inches)**	**(ft)**			**(n:12)**	**(inches^2)**	**(m^2)**	**(ft^2)**
P53_22	777.70	64.808	414.74	34.562	536	44.657	132.66	11.055	0.53	22%	0/12	251454	162	1746
P63_30	777.70	64.808	493.48	41.123	536	44.657	211.4	17.617	0.63	30%	0/12	270495	175	1878
P74_35	777.70	64.808	572.22	47.685	536	44.657	290.14	24.178	0.74	35%	0/12	289536	187	2011
P84_39	777.70	64.808	650.96	54.247	536	44.657	368.88	30.740	0.84	39%	0/12	308577	199	2143
P94_42	777.70	64.808	729.70	60.808	536	44.657	447.62	37.302	0.94	42%	0/12	327617	211	2275
P39_12	856.44	71.370	336.00	28.000	615	51.218	53.92	4.493	0.39	12%	0/12	254624	164	1768
P48_23	856.44	71.370	414.74	34.562	615	51.218	132.66	11.055	0.48	23%	0/12	273665	177	1900
P58_31	856.44	71.370	493.48	41.123	615	51.218	211.4	17.617	0.58	31%	0/12	292706	189	2033
P67_36	856.44	71.370	572.22	47.685	615	51.218	290.14	24.178	0.67	36%	0/12	311747	201	2165
P76_41	856.44	71.370	650.96	54.247	615	51.218	368.88	30.740	0.76	41%	0/12	330788	213	2297
P85_44	856.44	71.370	729.70	60.808	615	51.218	447.62	37.302	0.85	44%	0/12	349829	226	2429
Smith	525.00	43.750	345.00	28.750	62	5.167	210	17.500	0.66	7%	0/12	168105	108	1167
	Min.	32.000		28.000					0.39	0%				822
	Max.	71.370		60.808					1.00	44%				2429

Table B.4. Determination of Horizontal Structural Irregularities.

		ASCE 7 Horizontal Structural Irregularities					
		Torsional Irregularity			**Re-entrant Corn**		**Diaph.**
			Type 1a	**Type 1b**	**Type 2**		**Discont**
	Case	**Max. Amount Either Axis**	**Y/N**	**Y/N**	**Min. Amount Either Axis**	**Y/N**	**Type 3 Y/N**
Case 1	P82_10	1.26	Yes	No	0.22	Yes	No
	P100_0	1.02	No	No	0.00	No	No
	P88_8	1.35	Yes	No	0.16	Yes	No
	P93_16	1.35	Yes	No	0.32	Yes	No
	P78_22	1.35	Yes	No	0.43	Yes	No
	P67_26	1.35	Yes	No	0.51	Yes	No
	P59_29	1.36	Yes	No	0.52	Yes	No
	P53_32	1.36	Yes	No	0.52	Yes	No
	P73_8	1.28	Yes	No	0.16	Yes	No
	P90_15	1.27	Yes	No	0.32	Yes	No
	P94_20	1.27	Yes	No	0.43	Yes	No
	P81_24	1.28	Yes	No	0.48	Yes	No
	P71_27	1.29	Yes	No	0.48	Yes	No
	P63_29	1.30	Yes	No	0.48	Yes	No
	P62_9	1.32	Yes	No	0.16	Yes	No
	P77_18	1.33	Yes	No	0.32	Yes	No
	P91_24	1.32	Yes	No	0.43	Yes	No
	P95_28	1.32	Yes	No	0.51	Yes	No
	P83_31	1.33	Yes	No	0.55	Yes	No
	P74_34	1.34	Yes	No	0.55	Yes	No
	P54_10	1.35	Yes	No	0.16	Yes	No
	P67_20	1.36	Yes	No	0.32	Yes	No
	P80_26	1.36	Yes	No	0.43	Yes	No
	P92_31	1.36	Yes	No	0.51	Yes	No
	P95_35	1.36	Yes	No	0.57	Yes	No
	P85_37	1.37	Yes	No	0.61	Yes	No
	P48_10	1.36	Yes	No	0.16	Yes	No

Table B.4 cont'd. Determination of Horizontal Structural Irregularities.

	ASCE 7 Horizontal Structural Irregularities					
	Torsional Irregularity			Re-entrant Corn		Diaph.
		Type 1a	Type 1b	Type 2		Discont
Case	Max. Amount Either Axis	Y/N	Y/N	Min. Amount Either Axis	Y/N	Type 3 Y/N
P59_21	1.39	Yes	No	0.32	Yes	No
P71_28	1.39	Yes	No	0.43	Yes	No
P82_33	1.39	Yes	No	0.51	Yes	No
P93_37	1.38	Yes	No	0.57	Yes	No
P96_40	1.39	Yes	No	0.61	Yes	No
P43_11	1.38	Yes	No	0.16	Yes	No
P53_22	1.40	Yes	Yes	0.32	Yes	No
P63_30	1.42	Yes	Yes	0.43	Yes	No
P74_35	1.42	Yes	Yes	0.51	Yes	No
P84_39	1.41	Yes	Yes	0.57	Yes	No
P94_42	1.41	Yes	Yes	0.61	Yes	No
P39_12	1.39	Yes	No	0.16	Yes	No
P48_23	1.42	Yes	Yes	0.32	Yes	No
P58_31	1.43	Yes	Yes	0.43	Yes	No
P67_36	1.44	Yes	Yes	0.51	Yes	No
P76_41	1.43	Yes	Yes	0.57	Yes	No
P85_44	1.43	Yes	Yes	0.61	Yes	No
P82_10	1.28	Yes	No	0.22	Yes	No
P100_0	1.13	No	No	0.00	No	No
P88_8	1.38	Yes	No	0.16	Yes	No
P93_16	1.38	Yes	No	0.32	Yes	No
P78_22	1.38	Yes	No	0.43	Yes	No
P67_26	1.38	Yes	No	0.51	Yes	No
P59_29	1.38	Yes	No	0.52	Yes	No
P53_32	1.38	Yes	No	0.52	Yes	No
P73_8	1.30	Yes	No	0.16	Yes	No
P90_15	1.30	Yes	No	0.32	Yes	No

Table B.4 cont'd. Determination of Horizontal Structural Irregularities.

		ASCE 7 Horizontal Structural Irregularities					
		Torsional Irregularity			Re-entrant Corn		Diaph.
			Type 1a	Type 1b	Type 2		Discont
	Case	Max. Amount Either Axis	Y/N	Y/N	Min. Amount Either Axis	Y/N	Type 3 Y/N
Case 2	*P94_20*	1.30	Yes	No	0.43	Yes	No
	P81_24	1.31	Yes	No	0.48	Yes	No
	P71_27	1.31	Yes	No	0.48	Yes	No
	P63_29	1.31	Yes	No	0.48	Yes	No
	P62_9	1.35	Yes	No	0.16	Yes	No
	P77_18	1.35	Yes	No	0.32	Yes	No
	P91_24	1.35	Yes	No	0.43	Yes	No
	P95_28	1.35	Yes	No	0.51	Yes	No
	P83_31	1.36	Yes	No	0.55	Yes	No
	P74_34	1.36	Yes	No	0.55	Yes	No
	P54_10	1.39	Yes	No	0.16	Yes	No
	P67_20	1.39	Yes	No	0.32	Yes	No
	P80_26	1.38	Yes	No	0.43	Yes	No
	P92_31	1.38	Yes	No	0.51	Yes	No
	P95_35	1.39	Yes	No	0.57	Yes	No
	P85_37	1.39	Yes	No	0.61	Yes	No
	P48_10	1.41	Yes	Yes	0.16	Yes	No
	P59_21	1.42	Yes	Yes	0.32	Yes	No
	P71_28	1.42	Yes	Yes	0.43	Yes	No
	P82_33	1.41	Yes	Yes	0.51	Yes	No
	P93_37	1.41	Yes	Yes	0.57	Yes	No
	P96_40	1.42	Yes	Yes	0.61	Yes	No
	P43_11	1.42	Yes	Yes	0.16	Yes	No
	P53_22	1.44	Yes	Yes	0.32	Yes	No
	P63_30	1.45	Yes	Yes	0.43	Yes	No
	P74_35	1.44	Yes	Yes	0.51	Yes	No
	P84_39	1.44	Yes	Yes	0.57	Yes	No

Table B.4 cont'd. Determination of Horizontal Structural Irregularities.

	ASCE 7 Horizontal Structural Irregularities					
	Torsional Irregularity			Re-entrant Corn		Diaph.
		Type 1a	Type 1b	Type 2		Discont
Case	**Max. Amount Either Axis**	**Y/N**	**Y/N**	**Min. Amount Either Axis**	**Y/N**	**Type 3 Y/N**
P94_42	1.44	Yes	Yes	0.61	Yes	No
P39_12	1.43	Yes	Yes	0.16	Yes	No
P48_23	1.46	Yes	Yes	0.32	Yes	No
P58_31	1.47	Yes	Yes	0.43	Yes	No
P67_36	1.46	Yes	Yes	0.51	Yes	No
P76_41	1.46	Yes	Yes	0.57	Yes	No
P85_44	1.46	Yes	Yes	0.61	Yes	No
P82_10	1.77	Yes	Yes	0.22	Yes	No
P100_0	1.73	Yes	Yes	0.00	No	No
P88_8	1.92	Yes	Yes	0.16	Yes	No
P93_16	1.29	Yes	No	0.32	Yes	No
P78_22	1.30	Yes	No	0.43	Yes	No
P67_26	1.31	Yes	No	0.51	Yes	No
P59_29	1.32	Yes	No	0.52	Yes	No
P53_32	1.33	Yes	No	0.52	Yes	No
P73_8	1.92	Yes	Yes	0.16	Yes	No
P90_15	1.54	Yes	Yes	0.32	Yes	No
P94_20	1.17	No	No	0.43	Yes	No
P81_24	1.18	No	No	0.48	Yes	No
P71_27	1.19	No	No	0.48	Yes	No
P63_29	1.20	Yes	No	0.48	Yes	No
P62_9	1.91	Yes	Yes	0.16	Yes	No
P77_18	1.53	Yes	Yes	0.32	Yes	No
P91_24	1.33	Yes	No	0.43	Yes	No
P95_28	1.03	No	No	0.51	Yes	No
P83_31	1.03	No	No	0.55	Yes	No
P74_34	1.04	No	No	0.55	Yes	No

Table B.4 cont'd. Determination of Horizontal Structural Irregularities.

		ASCE 7 Horizontal Structural Irregularities					
		Torsional Irregularity			Re-entrant Corn		Diaph.
			Type 1a	Type 1b	Type 2		Discont
	Case	Max. Amount Either Axis	Y/N	Y/N	Min. Amount Either Axis	Y/N	Type 3 Y/N
Case 3	*P54_10*	1.90	Yes	Yes	0.16	Yes	No
	P67_20	1.50	Yes	Yes	0.32	Yes	No
	P80_26	1.36	Yes	No	0.43	Yes	No
	P92_31	1.38	Yes	No	0.51	Yes	No
	P95_35	1.74	Yes	Yes	0.57	Yes	No
	P85_37	1.84	Yes	Yes	0.61	Yes	No
	P48_10	1.89	Yes	Yes	0.16	Yes	No
	P59_21	1.47	Yes	Yes	0.32	Yes	No
	P71_28	1.37	Yes	No	0.43	Yes	No
	P82_33	1.40	Yes	Yes	0.51	Yes	No
	P93_37	1.40	Yes	Yes	0.57	Yes	No
	P96_40	1.30	Yes	No	0.61	Yes	No
	P43_11	1.88	Yes	Yes	0.16	Yes	No
	P53_22	1.44	Yes	Yes	0.32	Yes	No
	P63_30	1.39	Yes	No	0.43	Yes	No
	P74_35	1.42	Yes	Yes	0.51	Yes	No
	P84_39	1.43	Yes	Yes	0.57	Yes	No
	P94_42	1.42	Yes	Yes	0.61	Yes	No
	P39_12	1.87	Yes	Yes	0.16	Yes	No
	P48_23	1.41	Yes	Yes	0.32	Yes	No
	P58_31	1.39	Yes	No	0.43	Yes	No
	P67_36	1.43	Yes	Yes	0.51	Yes	No
	P76_41	1.45	Yes	Yes	0.57	Yes	No
	P85_44	1.45	Yes	Yes	0.61	Yes	No
	Smith	1.22	Yes	No	0.12	No	No

Table B.4 cont'd. Determination of Horizontal Structural Irregularities.

	ASCE 7 Horizontal Structural Irregularities					
	Torsional Irregularity			**Re-entrant Corn**		**Diaph.**
		Type 1a	**Type 1b**	**Type 2**		**Discont**
	No	8	79	No	3	132
	Yes	124	53	Yes	129	0
	%	94%	40%	%	98%	0%

Table B.5. Conflicts in Calculated Flexibility.

		Flat roof & Hip roof		**Gable roofs 4:12 Pitch**	
	Case	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**
Case 1	P82_10	0	1	0	3
	P100_0	0	2	0	2
	P88_8	0	1	0	1
	P93_16	0	1	0	3
	P78_22	0	2	0	3
	P67_26	0	2	0	3
	P59_29	0	3	0	3
	P53_32	0	3	0	3
	P73_8	0	1	0	2
	P90_15	0	1	0	4
	P94_20	0	2	0	4
	P81_24	0	2	0	3
	P71_27	0	3	0	2
	P63_29	0	3	0	2
	P62_9	0	1	0	4
	P77_18	0	1	0	3
	P91_24	0	3	0	3
	P95_28	0	4	0	2
	P83_31	0	3	0	2
	P74_34	0	3	0	1
	P54_10	0	1	0	4
	P67_20	0	2	0	3
	P80_26	0	3	0	3
	P92_31	0	3	0	2
	P95_35	0	3	0	2
	P85_37	0	2	0	1
	P48_10	0	0	0	4

Table B.5 cont'd. Conflicts in Calculated Flexibility.

	Flat roof & Hip roof		Gable roofs 4:12 Pitch	
Case	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**
P59_21	0	3	0	3
P71_28	0	3	0	3
P82_33	0	3	0	2
P93_37	0	3	0	2
P96_40	0	2	0	2
P43_11	0	0	0	4
P53_22	0	2	0	3
P63_30	0	1	0	3
P74_35	0	3	0	2
P84_39	0	2	0	2
P94_42	0	2	0	2
P39_12	0	0	0	4
P48_23	0	2	0	3
P58_31	0	1	0	3
P67_36	0	2	0	2
P76_41	0	2	0	2
P85_44	0	2	0	2
P82_10	0	1	0	3
P100_0	0	2	0	2
P88_8	0	1	0	1
P93_16	0	2	0	3
P78_22	0	2	0	3
P67_26	0	2	0	3
P59_29	0	3	0	3
P53_32	0	3	0	3
P73_8	0	1	0	2
P90_15	0	1	0	4

Table B.5 cont'd. Conflicts in Calculated Flexibility.

		Flat roof & Hip roof		Gable roofs 4:12 Pitch	
	Case	**Rigid says semi-rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**	**Rigid says semi-rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**
Case 2	*P94_20*	0	2	0	4
	P81_24	0	2	0	3
	P71_27	0	3	0	3
	P63_29	0	3	0	2
	P62_9	0	1	0	4
	P77_18	0	1	0	3
	P91_24	0	3	0	3
	P95_28	0	3	0	2
	P83_31	0	2	0	2
	P74_34	0	2	0	1
	P54_10	0	1	0	4
	P67_20	0	2	0	3
	P80_26	0	3	0	3
	P92_31	0	2	0	2
	P95_35	0	2	0	2
	P85_37	0	2	0	2
	P48_10	0	1	0	4
	P59_21	0	3	0	3
	P71_28	0	3	0	3
	P82_33	0	3	0	2
	P93_37	0	3	0	2
	P96_40	0	3	0	2
	P43_11	0	1	0	4
	P53_22	0	3	0	3
	P63_30	0	2	0	3
	P74_35	0	3	0	2
	P84_39	0	3	0	2

Table B.5 cont'd. Conflicts in Calculated Flexibility.

		Flat roof & Hip roof		Gable roofs 4:12 Pitch	
	Case	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**
	P94_42	0	3	0	2
	P39_12	0	0	0	4
	P48_23	0	3	0	3
	P58_31	0	2	0	3
	P67_36	0	3	0	2
	P76_41	0	3	0	2
	P85_44	0	3	0	2
	P82_10	0	1	0	3
	P100_0	0	2	0	2
	P88_8	0	1	0	2
	P93_16	0	1	0	2
	P78_22	0	1	0	2
	P67_26	0	2	0	2
	P59_29	0	2	0	2
	P53_32	0	2	0	2
	P73_8	0	1	0	3
	P90_15	0	1	0	3
	P94_20	0	1	0	3
	P81_24	0	2	0	2
	P71_27	0	2	0	2
	P63_29	0	2	0	1
	P62_9	0	1	0	3
	P77_18	0	2	0	2
	P91_24	0	2	0	2
	P95_28	0	2	0	1
	P83_31	0	1	0	1
	P74_34	0	1	0	0

Table B.5 cont'd. Conflicts in Calculated Flexibility.

		Flat roof & Hip roof		Gable roofs 4:12 Pitch	
	Case	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**
Case 3	*P54_10*	0	1	0	3
	P67_20	0	2	0	2
	P80_26	0	2	0	2
	P92_31	0	1	0	2
	P95_35	0	2	0	1
	P85_37	0	2	0	1
	P48_10	0	1	0	3
	P59_21	0	2	0	2
	P71_28	0	2	0	3
	P82_33	0	2	0	2
	P93_37	0	3	0	2
	P96_40	0	2	0	1
	P43_11	0	1	0	3
	P53_22	0	2	0	2
	P63_30	0	1	0	3
	P74_35	0	2	0	2
	P84_39	0	3	0	2
	P94_42	0	3	0	2
	P39_12	0	1	0	3
	P48_23	0	2	0	2
	P58_31	0	1	0	3
	P67_36	0	2	0	2
	P76_41	0	3	0	2
	P85_44	0	3	0	2
	Smith	0	1	0	2

Table B.5 cont'd. Conflicts in Calculated Flexibility.

	Flat roof & Hip roof		Gable roofs 4:12 Pitch	
Case	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**	**Rigid says semi rigid, flex says flex**	**Rigid says flex, flex says semi-rigid**
Case 1				
Totals	0	89	0	116
Percent	0%	17%	0%	22%
Case 1-3				
Totals	0	263	0	328
Percent	0%	50%	0%	62%

Total number of diaphragms 528

Table B.6. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

	Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
Case	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition
P82_10	0.14	0.17	0.84	Semi-	0.17	0.16	1.08	Semi-	0.12	0.13	0.91	Semi-	0.29	0.10	2.87	Flexib
P100_0	####	####	#####	Flexib	0.29	####	0.00	Semi-	####	####	#####	Flexib	0.27	####	0.00	Semi-
P88_8	0.13	0.21	0.61	Semi-	0.15	0.20	0.73	Semi-	0.10	0.15	0.65	Semi-	0.28	0.10	2.73	Flexib
P93_16	0.14	0.12	1.17	Semi-	0.20	0.11	1.83	Semi-	0.12	0.16	0.75	Semi-	0.31	0.11	2.85	Flexib
P78_22	0.15	0.09	1.56	Semi-	0.26	0.08	3.04	Flexib	0.10	0.17	0.57	Semi-	0.34	0.12	2.94	Flexib
P67_26	0.16	0.08	1.85	Semi-	0.31	0.07	4.34	Flexib	0.31	0.18	1.76	Semi-	0.36	0.12	3.01	Flexib
P59_29	0.16	0.08	2.11	Flexib	0.37	0.07	5.72	Flexib	0.33	0.19	1.76	Semi-	0.39	0.13	3.06	Flexib
P53_32	0.17	0.07	2.36	Flexib	0.43	0.06	7.17	Flexib	0.34	0.19	1.77	Semi-	0.41	0.13	3.11	Flexib
P73_8	0.16	0.22	0.71	Semi-	0.18	0.22	0.84	Semi-	0.12	0.12	0.97	Semi-	0.29	0.09	3.10	Flexib
P90_15	0.15	0.13	1.18	Semi-	0.21	0.12	1.77	Semi-	0.14	0.13	1.04	Semi-	0.32	0.10	3.22	Flexib
P94_20	0.17	0.10	1.61	Semi-	0.26	0.09	2.92	Flexib	0.17	0.14	1.18	Semi-	0.36	0.11	3.35	Flexib
P81_24	0.18	0.09	1.94	Semi-	0.32	0.08	4.17	Flexib	0.29	0.15	1.90	Semi-	0.39	0.11	3.45	Flexib
P71_27	0.19	0.09	2.22	Flexib	0.39	0.07	5.49	Flexib	0.31	0.16	1.92	Semi-	0.42	0.12	3.52	Flexib
P63_29	0.20	0.08	2.46	Flexib	0.45	0.07	6.88	Flexib	0.33	0.17	1.93	Semi-	0.45	0.12	3.58	Flexib
P62_9	0.27	0.23	1.15	Semi-	0.22	0.23	0.95	Semi-	0.13	0.11	1.17	Semi-	0.33	0.08	4.09	Flexib
P77_18	0.26	0.13	1.91	Semi-	0.25	0.12	1.99	Semi-	0.13	0.12	1.14	Semi-	0.33	0.08	4.02	Flexib

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

		Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
	Case	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition
		(in)	(in)			(in)	(in)			(in)	(in)			(in)	(in)		
Case 1	*P91_24*	0.25	0.11	2.35	Flexib	0.27	0.09	2.87	Flexib	0.16	0.13	1.28	Semi-	0.37	0.09	4.24	Flexib
	P95_28	0.26	0.09	2.72	Flexib	0.31	0.08	3.95	Flexib	0.28	0.13	2.11	Flexib	0.41	0.09	4.40	Flexib
	P83_31	0.27	0.09	3.02	Flexib	0.37	0.07	5.20	Flexib	0.31	0.14	2.13	Flexib	0.45	0.10	4.51	Flexib
	P74_34	0.28	0.08	3.28	Flexib	0.43	0.07	6.54	Flexib	0.33	0.15	2.15	Flexib	0.48	0.10	4.60	Flexib
	P54_10	0.30	0.25	1.21	Semi-	0.25	0.24	1.03	Semi-	0.15	0.11	1.33	Semi-	0.37	0.07	5.16	Flexib
	P67_20	0.29	0.15	1.99	Semi-	0.29	0.13	2.17	Flexib	0.15	0.11	1.32	Semi-	0.37	0.07	5.04	Flexib
	P80_26	0.28	0.12	2.42	Flexib	0.32	0.10	3.17	Flexib	0.16	0.12	1.37	Semi-	0.39	0.08	5.09	Flexib
	P92_31	0.27	0.10	2.73	Flexib	0.34	0.08	4.11	Flexib	0.28	0.12	2.27	Flexib	0.44	0.08	5.32	Flexib
	P95_35	0.27	0.09	2.99	Flexib	0.36	0.07	5.00	Flexib	0.30	0.13	2.30	Flexib	0.48	0.09	5.50	Flexib
	P85_37	0.28	0.09	3.26	Flexib	0.41	0.07	6.27	Flexib	0.33	0.14	2.32	Flexib	0.52	0.09	5.63	Flexib
	P48_10	0.33	0.26	1.26	Semi-	0.27	0.25	1.10	Semi-	0.15	0.10	1.46	Semi-	0.42	0.07	6.29	Flexib
	P59_21	0.32	0.16	2.05	Flexib	0.32	0.14	2.30	Flexib	0.16	0.11	1.47	Semi-	0.42	0.07	6.13	Flexib
	P71_28	0.31	0.12	2.49	Flexib	0.36	0.11	3.39	Flexib	0.17	0.11	1.49	Semi-	0.42	0.07	6.03	Flexib
	P82_33	0.30	0.11	2.80	Flexib	0.39	0.09	4.42	Flexib	0.28	0.12	2.41	Flexib	0.45	0.07	6.23	Flexib
	P93_37	0.29	0.10	3.05	Flexib	0.41	0.08	5.40	Flexib	0.30	0.12	2.44	Flexib	0.50	0.08	6.47	Flexib

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

	Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
Case	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition
	(in)	(in)			(in)	(in)			(in)	(in)			(in)	(in)		
P96_40	0.29	0.09	3.27	Flexib	0.43	0.07	6.35	Flexib	0.32	0.13	2.46	Flexib	0.54	0.08	6.65	Flexib
P43_11	0.35	0.27	1.30	Semi-	0.30	0.26	1.16	Semi-	0.16	0.10	1.61	Semi-	0.47	0.06	7.48	Flexib
P53_22	0.35	0.17	2.11	Flexib	0.35	0.15	2.40	Flexib	0.17	0.11	1.60	Semi-	0.46	0.06	7.29	Flexib
P63_30	0.34	0.13	2.56	Flexib	0.40	0.11	3.54	Flexib	0.18	0.11	1.65	Semi-	0.46	0.06	7.15	Flexib
P74_35	0.33	0.12	2.87	Flexib	0.43	0.09	4.64	Flexib	0.28	0.11	2.54	Flexib	0.47	0.07	7.13	Flexib
P84_39	0.32	0.10	3.12	Flexib	0.47	0.08	5.72	Flexib	0.30	0.12	2.56	Flexib	0.52	0.07	7.43	Flexib
P94_42	0.32	0.09	3.33	Flexib	0.49	0.07	6.76	Flexib	0.32	0.12	2.59	Flexib	0.57	0.07	7.67	Flexib
P39_12	0.38	0.28	1.34	Semi-	0.32	0.26	1.21	Semi-	0.18	0.10	1.80	Semi-	0.53	0.06	8.72	Flexib
P48_23	0.38	0.18	2.15	Flexib	0.38	0.15	2.49	Flexib	0.18	0.10	1.72	Semi-	0.51	0.06	8.50	Flexib
P58_31	0.37	0.14	2.61	Flexib	0.43	0.12	3.66	Flexib	0.19	0.11	1.78	Semi-	0.51	0.06	8.32	Flexib
P67_36	0.37	0.12	2.93	Flexib	0.47	0.10	4.81	Flexib	0.29	0.11	2.66	Flexib	0.51	0.06	8.21	Flexib
P76_41	0.36	0.11	3.19	Flexib	0.51	0.09	5.95	Flexib	0.30	0.11	2.68	Flexib	0.54	0.06	8.39	Flexib
P85_44	0.35	0.10	3.40	Flexib	0.54	0.08	7.08	Flexib	0.32	0.12	2.71	Flexib	0.59	0.07	8.69	Flexib
P82_10	0.15	0.17	0.87	Semi-	0.19	0.17	1.16	Semi-	0.12	0.15	0.83	Semi-	0.30	0.11	2.62	Flexib
P100_0	####	####	#####	Flexib	0.37	####	0.00	Semi-	####	####	#####	Flexib	0.26	####	0.00	Semi-

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

	Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
Case	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition
	(in)	(in)			(in)	(in)			(in)	(in)			(in)	(in)		
P88_8	0.14	0.22	0.63	Semi-	0.17	0.21	0.81	Semi-	0.10	0.18	0.57	Semi-	0.29	0.11	2.48	Flexib
P93_16	0.14	0.12	1.18	Semi-	0.24	0.12	2.03	Flexib	0.12	0.19	0.63	Semi-	0.31	0.12	2.55	Flexib
P78_22	0.15	0.10	1.54	Semi-	0.30	0.09	3.34	Flexib	0.09	0.20	0.46	Semi-	0.34	0.13	2.60	Flexib
P67_26	0.15	0.08	1.82	Semi-	0.36	0.08	4.73	Flexib	0.31	0.21	1.45	Semi-	0.36	0.14	2.64	Flexib
P59_29	0.16	0.08	2.06	Flexib	0.43	0.07	6.18	Flexib	0.32	0.22	1.44	Semi-	0.38	0.14	2.67	Flexib
P53_32	0.17	0.07	2.30	Flexib	0.49	0.06	7.71	Flexib	0.33	0.23	1.44	Semi-	0.40	0.15	2.71	Flexib
P73_8	0.15	0.23	0.64	Semi-	0.17	0.22	0.77	Semi-	0.12	0.14	0.84	Semi-	0.29	0.10	2.81	Flexib
P90_15	0.16	0.13	1.26	Semi-	0.24	0.12	1.95	Semi-	0.14	0.15	0.92	Semi-	0.33	0.11	2.91	Flexib
P94_20	0.17	0.10	1.67	Semi-	0.31	0.09	3.24	Flexib	0.17	0.17	1.00	Semi-	0.36	0.12	2.96	Flexib
P81_24	0.18	0.09	1.97	Semi-	0.38	0.08	4.63	Flexib	0.28	0.18	1.59	Semi-	0.39	0.13	2.99	Flexib
P71_27	0.19	0.09	2.22	Flexib	0.46	0.08	6.07	Flexib	0.30	0.19	1.57	Semi-	0.41	0.14	3.01	Flexib
P63_29	0.20	0.08	2.44	Flexib	0.53	0.07	7.56	Flexib	0.31	0.20	1.55	Semi-	0.44	0.14	3.02	Flexib
P62_9	0.26	0.24	1.09	Semi-	0.21	0.23	0.90	Semi-	0.13	0.13	0.99	Semi-	0.31	0.09	3.63	Flexib
P77_18	0.24	0.13	1.84	Semi-	0.23	0.12	1.87	Semi-	0.14	0.14	1.02	Semi-	0.34	0.09	3.71	Flexib
P91_24	0.25	0.11	2.38	Flexib	0.29	0.09	3.03	Flexib	0.17	0.15	1.12	Semi-	0.38	0.10	3.82	Flexib

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

		Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
	Case	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition
		(in)	(in)			(in)	(in)			(in)	(in)			(in)	(in)		
Case 2	*P95_28*	0.26	0.10	2.78	Flexib	0.36	0.08	4.35	Flexib	0.29	0.16	1.79	Semi-	0.42	0.11	3.88	Flexib
	P83_31	0.27	0.09	3.08	Flexib	0.43	0.07	5.76	Flexib	0.30	0.17	1.77	Semi-	0.45	0.11	3.92	Flexib
	P74_34	0.28	0.08	3.33	Flexib	0.50	0.07	7.25	Flexib	0.32	0.18	1.76	Semi-	0.48	0.12	3.95	Flexib
	P54_10	0.29	0.25	1.16	Semi-	0.24	0.24	1.02	Semi-	0.14	0.12	1.16	Semi-	0.35	0.08	4.58	Flexib
	P67_20	0.27	0.14	1.95	Semi-	0.28	0.13	2.13	Flexib	0.14	0.13	1.12	Semi-	0.36	0.08	4.50	Flexib
	P80_26	0.26	0.11	2.39	Flexib	0.30	0.10	3.08	Flexib	0.17	0.14	1.22	Semi-	0.40	0.09	4.67	Flexib
	P92_31	0.26	0.10	2.75	Flexib	0.34	0.08	4.13	Flexib	0.28	0.15	1.95	Semi-	0.44	0.09	4.79	Flexib
	P95_35	0.28	0.09	3.08	Flexib	0.40	0.07	5.47	Flexib	0.30	0.16	1.94	Semi-	0.48	0.10	4.86	Flexib
	P85_37	0.29	0.09	3.35	Flexib	0.47	0.07	6.90	Flexib	0.32	0.17	1.93	Semi-	0.51	0.10	4.91	Flexib
	P48_10	0.32	0.26	1.24	Semi-	0.28	0.25	1.10	Semi-	0.15	0.12	1.31	Semi-	0.39	0.07	5.60	Flexib
	P59_21	0.31	0.15	2.04	Flexib	0.32	0.14	2.33	Flexib	0.15	0.12	1.27	Semi-	0.40	0.07	5.47	Flexib
	P71_28	0.29	0.12	2.49	Flexib	0.35	0.10	3.40	Flexib	0.17	0.13	1.31	Semi-	0.42	0.08	5.53	Flexib
	P82_33	0.28	0.10	2.80	Flexib	0.38	0.09	4.38	Flexib	0.28	0.14	2.09	Flexib	0.47	0.08	5.69	Flexib
	P93_37	0.28	0.09	3.05	Flexib	0.39	0.07	5.30	Flexib	0.30	0.15	2.08	Flexib	0.51	0.09	5.81	Flexib
	P96_40	0.29	0.09	3.34	Flexib	0.45	0.07	6.61	Flexib	0.32	0.16	2.08	Flexib	0.55	0.09	5.90	Flexib

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

	Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
Case	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**
	(in)	**(in)**			**(in)**	**(in)**			**(in)**	**(in)**			**(in)**	**(in)**		
P43_11	0.35	0.27	1.30	Semi-	0.30	0.26	1.18	Semi-	0.16	0.11	1.44	Semi-	0.44	0.07	6.68	Flexib
P53_22	0.34	0.16	2.13	Flexib	0.35	0.14	2.47	Flexib	0.17	0.12	1.41	Semi-	0.44	0.07	6.51	Flexib
P63_30	0.33	0.13	2.59	Flexib	0.40	0.11	3.63	Flexib	0.17	0.12	1.41	Semi-	0.44	0.07	6.40	Flexib
P74_35	0.32	0.11	2.91	Flexib	0.43	0.09	4.73	Flexib	0.28	0.13	2.21	Flexib	0.49	0.07	6.60	Flexib
P84_39	0.31	0.10	3.15	Flexib	0.46	0.08	5.77	Flexib	0.30	0.14	2.21	Flexib	0.53	0.08	6.77	Flexib
P94_42	0.30	0.09	3.37	Flexib	0.47	0.07	6.76	Flexib	0.32	0.15	2.21	Flexib	0.57	0.08	6.89	Flexib
P39_12	0.38	0.28	1.36	Semi-	0.33	0.26	1.24	Semi-	0.17	0.11	1.57	Semi-	0.49	0.06	7.83	Flexib
P48_23	0.37	0.17	2.21	Flexib	0.39	0.15	2.59	Flexib	0.18	0.12	1.53	Semi-	0.48	0.06	7.61	Flexib
P58_31	0.36	0.13	2.68	Flexib	0.43	0.11	3.81	Flexib	0.19	0.12	1.55	Semi-	0.48	0.06	7.47	Flexib
P67_36	0.35	0.12	3.00	Flexib	0.47	0.10	4.99	Flexib	0.29	0.12	2.33	Flexib	0.51	0.07	7.52	Flexib
P76_41	0.34	0.10	3.25	Flexib	0.51	0.08	6.13	Flexib	0.30	0.13	2.33	Flexib	0.56	0.07	7.73	Flexib
P85_44	0.33	0.09	3.47	Flexib	0.54	0.07	7.24	Flexib	0.32	0.14	2.34	Flexib	0.60	0.08	7.89	Flexib
P82_10	0.29	0.20	1.42	Semi-	0.04	0.62	0.07	Semi-	0.13	0.15	0.84	Semi-	0.35	0.12	2.91	Flexib
P100_0	####	####	#####	Flexib	0.15	####	0.00	Semi-	####	####	#####	Flexib	0.26	####	0.00	Semi-
P88_8	0.27	0.25	1.10	Semi-	0.02	1.63	0.01	Semi-	0.10	0.18	0.58	Semi-	0.33	0.12	2.70	Flexib

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

	Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
Case	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition	Dia. Defl.	Ave. SW Defl.	Ratio D/S	Condition
	(in)	(in)			(in)	(in)			(in)	(in)			(in)	(in)		
P93_16	0.17	0.13	1.27	Semi-	0.32	0.14	2.28	Flexib	0.18	0.22	0.84	Semi-	0.23	0.33	0.69	Semi-
P78_22	0.19	0.11	1.75	Semi-	0.38	0.11	3.52	Flexib	0.14	0.23	0.59	Semi-	0.24	0.35	0.68	Semi-
P67_26	0.20	0.09	2.16	Flexib	0.44	0.09	4.83	Flexib	0.35	0.24	1.45	Semi-	0.25	0.38	0.67	Semi-
P59_29	0.22	0.09	2.52	Flexib	0.51	0.08	6.20	Flexib	0.37	0.26	1.44	Semi-	0.26	0.40	0.66	Semi-
P53_32	0.24	0.08	2.86	Flexib	0.57	0.07	7.65	Flexib	0.38	0.27	1.43	Semi-	0.28	0.43	0.65	Semi-
P73_8	0.29	0.26	1.11	Semi-	0.03	1.66	0.02	Semi-	0.12	0.14	0.86	Semi-	0.37	0.11	3.31	Flexib
P90_15	0.30	0.16	1.89	Semi-	0.09	0.30	0.29	Semi-	0.14	0.15	0.90	Semi-	0.34	0.11	3.03	Flexib
P94_20	0.18	0.11	1.65	Semi-	0.41	0.12	3.57	Flexib	0.24	0.19	1.24	Semi-	0.25	0.24	1.08	Semi-
P81_24	0.21	0.10	2.08	Flexib	0.49	0.10	4.89	Flexib	0.33	0.21	1.61	Semi-	0.27	0.26	1.04	Semi-
P71_27	0.23	0.09	2.46	Flexib	0.57	0.09	6.24	Flexib	0.35	0.22	1.59	Semi-	0.28	0.28	1.00	Semi-
P63_29	0.25	0.09	2.81	Flexib	0.65	0.09	7.65	Flexib	0.37	0.24	1.57	Semi-	0.30	0.31	0.97	Semi-
P62_9	0.37	0.28	1.31	Semi-	0.03	1.68	0.02	Semi-	0.13	0.13	1.03	Semi-	0.43	0.10	4.36	Flexib
P77_18	0.34	0.16	2.07	Flexib	0.09	0.30	0.30	Semi-	0.14	0.14	1.04	Semi-	0.40	0.10	4.10	Flexib
P91_24	0.34	0.13	2.58	Flexib	0.15	0.19	0.79	Semi-	0.15	0.14	1.05	Semi-	0.37	0.10	3.82	Flexib
P95_28	0.28	0.10	2.76	Flexib	0.48	0.10	4.77	Flexib	0.33	0.18	1.83	Semi-	0.31	0.18	1.73	Semi-

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

		Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
	Case	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition	Dia. Defl. (in)	Ave. SW Defl. (in)	Ratio D/S	Condition
Case 3	P83_31	0.29	0.09	3.04	Flexib	0.56	0.09	6.17	Flexib	0.35	0.19	1.81	Semi-	0.33	0.19	1.71	Semi-
	P74_34	0.29	0.09	3.28	Flexib	0.65	0.08	7.63	Flexib	0.37	0.21	1.79	Semi-	0.35	0.21	1.69	Semi-
	P54_10	0.40	0.30	1.37	Semi-	0.03	1.70	0.02	Semi-	0.16	0.13	1.28	Semi-	0.49	0.09	5.45	Flexib
	P67_20	0.38	0.18	2.14	Flexib	0.10	0.32	0.33	Semi-	0.15	0.13	1.18	Semi-	0.46	0.09	5.20	Flexib
	P80_26	0.36	0.14	2.57	Flexib	0.16	0.19	0.82	Semi-	0.16	0.14	1.21	Semi-	0.43	0.09	4.89	Flexib
	P92_31	0.35	0.12	2.91	Flexib	0.21	0.14	1.44	Semi-	0.27	0.14	1.93	Semi-	0.41	0.09	4.65	Flexib
	P95_35	0.30	0.10	3.05	Flexib	0.64	0.10	6.47	Flexib	0.40	0.20	2.03	Flexib	0.28	0.89	0.31	Semi-
	P85_37	0.31	0.09	3.30	Flexib	0.73	0.09	7.94	Flexib	0.42	0.21	2.02	Flexib	0.29	1.56	0.18	Semi-
	P48_10	0.44	0.31	1.41	Semi-	0.04	1.72	0.02	Semi-	0.19	0.12	1.54	Semi-	0.55	0.08	6.58	Flexib
	P59_21	0.42	0.19	2.20	Flexib	0.11	0.33	0.35	Semi-	0.17	0.12	1.33	Semi-	0.52	0.08	6.33	Flexib
	P71_28	0.40	0.15	2.65	Flexib	0.18	0.20	0.87	Semi-	0.17	0.13	1.34	Semi-	0.49	0.08	6.02	Flexib
	P82_33	0.38	0.13	2.96	Flexib	0.23	0.15	1.51	Semi-	0.28	0.13	2.08	Flexib	0.46	0.08	5.73	Flexib
	P93_37	0.36	0.11	3.21	Flexib	0.27	0.12	2.21	Flexib	0.28	0.14	2.07	Flexib	0.48	0.08	5.70	Flexib
	P96_40	0.33	0.10	3.39	Flexib	0.69	0.09	7.77	Flexib	0.42	0.19	2.20	Flexib	0.34	0.30	1.14	Semi-
	P43_11	0.48	0.33	1.45	Semi-	0.04	1.73	0.02	Semi-	0.22	0.12	1.81	Semi-	0.62	0.08	7.75	Flexib

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

	Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
Case	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**
	(in)	**(in)**			**(in)**	**(in)**			**(in)**	**(in)**			**(in)**	**(in)**		
P53_22	0.45	0.20	2.24	Flexib	0.12	0.34	0.37	Semi-	0.18	0.12	1.50	Semi-	0.59	0.08	7.51	Flexib
P63_30	0.43	0.16	2.70	Flexib	0.19	0.21	0.92	Semi-	0.19	0.13	1.49	Semi-	0.55	0.08	7.19	Flexib
P74_35	0.42	0.14	3.03	Flexib	0.25	0.16	1.59	Semi-	0.28	0.13	2.21	Flexib	0.52	0.08	6.88	Flexib
P84_39	0.40	0.12	3.29	Flexib	0.30	0.13	2.34	Flexib	0.29	0.13	2.20	Flexib	0.50	0.08	6.62	Flexib
P94_42	0.38	0.11	3.52	Flexib	0.34	0.11	3.14	Flexib	0.30	0.14	2.20	Flexib	0.55	0.08	6.82	Flexib
P39_12	0.51	0.35	1.48	Semi-	0.05	1.75	0.03	Semi-	0.26	0.12	2.10	Flexib	0.69	0.08	8.97	Flexib
P48_23	0.49	0.22	2.27	Flexib	0.13	0.35	0.39	Semi-	0.20	0.12	1.69	Semi-	0.66	0.08	8.73	Flexib
P58_31	0.47	0.17	2.74	Flexib	0.21	0.22	0.96	Semi-	0.20	0.12	1.64	Semi-	0.61	0.07	8.41	Flexib
P67_36	0.45	0.15	3.09	Flexib	0.27	0.16	1.66	Semi-	0.29	0.13	2.33	Flexib	0.58	0.07	8.07	Flexib
P76_41	0.44	0.13	3.37	Flexib	0.33	0.13	2.45	Flexib	0.30	0.13	2.32	Flexib	0.56	0.07	7.78	Flexib
P85_44	0.42	0.12	3.60	Flexib	0.38	0.11	3.29	Flexib	0.30	0.13	2.32	Flexib	0.56	0.07	7.70	Flexib
Smith	0.39	0.21	1.81	Semi-	0.49	0.17	2.88	Flexib	0.38	0.24	1.62	Semi-	0.48	0.23	2.06	Flexib

Table B.6 cont'd. Calculated Diaphragm Flexibility for Flat or Hip Roofs.

	Diaphragm A				Diaphragm B				Diaphragm C				Diaphragm D			
Case	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**	**Dia. Defl.**	**Ave. SW Defl.**	**Ratio D/S**	**Condition**
	(in)	**(in)**			**(in)**	**(in)**			**(in)**	**(in)**			**(in)**	**(in)**		
Ind. Flexib				88				85				39				114
Ind. Semi-Rigid				44				47				93				18

All	**Flexible**	**29**
	Semi-rigid	**1**

Table B.7. Calculation of Torsional Shear Percentage.

								Percentage torsion of direct shear					
	Case	Total Shear a	b	c	d	a'	b'	Total Shear a	b	c	d	a'	b'
		(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
	P82_10	996	995	566	215	454	779	3%	1%	2%	0%	3%	1%
	P100_0	1063	1210	0	0	1110	1103	7%	22%	12%	11%	12%	11%
	P88_8	858	877	580	135	318	734	5%	5%	5%	0%	7%	5%
	P93_16	1007	1114	700	309	414	724	10%	19%	13%	3%	24%	14%
	P78_22	1126	1368	804	480	512	696	11%	32%	17%	8%	39%	18%
	P67_26	1230	1622	892	656	604	657	11%	43%	19%	14%	50%	18%
	P59_29	1326	1870	970	838	687	614	10%	52%	19%	20%	57%	15%
	P53_32	1418	2109	1041	1024	762	572	8%	58%	18%	25%	61%	11%
	P73_8	1006	1060	594	161	427	916	3%	8%	2%	2%	3%	11%
	P90_15	1158	1126	686	354	512	768	6%	2%	5%	0%	10%	2%
	P94_20	1336	1389	800	526	648	784	10%	14%	10%	0%	26%	12%
	P81_24	1486	1673	904	700	788	779	12%	25%	13%	3%	40%	18%
	P71_27	1618	1964	997	884	921	756	12%	34%	15%	7%	50%	19%
	P63_29	1740	2252	1081	1077	1043	725	11%	42%	15%	11%	58%	18%
	P62_9	1246	1268	778	187	419	1186	13%	13%	5%	4%	8%	26%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

	Case	Total Shear a	b	c	d	a'	b'	Percentage torsion of direct shear: Total Shear a	b	c	d	a'	b'
		(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
Case 1	*P77_18*	1225	1355	824	402	429	973	0%	9%	0%	2%	0%	16%
	P91_24	1444	1391	952	583	542	802	8%	2%	5%	0%	15%	3%
	P95_28	1627	1566	1079	751	673	780	11%	6%	10%	0%	32%	7%
	P83_31	1784	1829	1199	920	810	794	13%	14%	13%	2%	47%	15%
	P74_34	1923	2102	1308	1096	943	793	13%	22%	14%	4%	59%	19%
	P54_10	1504	1438	932	211	402	1442	21%	15%	4%	5%	9%	37%
	P67_20	1450	1571	1007	453	432	1228	7%	14%	2%	3%	7%	32%
	P80_26	1520	1660	1085	648	460	1036	3%	11%	1%	1%	5%	21%
	P92_31	1736	1709	1222	819	573	875	9%	6%	6%	0%	21%	10%
	P95_35	1922	1738	1357	982	702	754	12%	0%	9%	0%	38%	0%
	P85_37	2084	1980	1487	1145	835	781	14%	7%	12%	1%	54%	9%
	P48_10	1771	1586	1057	235	380	1676	29%	14%	1%	5%	7%	44%
	P59_21	1692	1750	1167	503	426	1474	13%	16%	3%	4%	10%	44%
	P71_28	1648	1879	1237	715	429	1281	2%	15%	1%	3%	3%	38%
	P82_33	1820	1973	1351	896	494	1104	5%	13%	2%	1%	11%	28%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

	Case	Total Shear a	b	c	d	a'	b'	Percentage torsion of direct shear: Total Shear a	b	c	d	a'	b'
		(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
	P93_37	2033	2033	1493	1059	607	951	10%	9%	6%	0%	27%	18%
	P96_40	2220	2068	1635	1222	732	824	13%	4%	9%	0%	44%	7%
	P43_11	2042	1723	1225	258	357	1889	35%	13%	3%	5%	4%	48%
	P53_22	1942	1906	1305	551	414	1703	19%	16%	1%	5%	12%	52%
	P63_30	1875	2064	1403	782	432	1519	7%	17%	2%	4%	9%	51%
	P74_35	1885	2192	1480	975	434	1340	1%	16%	0%	2%	2%	44%
	P84_39	2123	2290	1619	1144	529	1175	7%	14%	3%	1%	17%	36%
	P94_42	2333	2361	1765	1302	641	1029	11%	11%	6%	0%	34%	26%
	P39_12	2316	1855	1429	280	337	2085	41%	12%	7%	5%	0%	50%
	P48_23	2199	2049	1442	598	400	1915	25%	15%	1%	5%	11%	58%
	P58_31	2112	2227	1550	848	429	1744	12%	17%	1%	4%	12%	61%
	P67_36	2059	2380	1638	1054	427	1571	3%	18%	1%	3%	5%	58%
	P76_41	2198	2508	1745	1232	467	1405	4%	17%	1%	2%	8%	52%
	P85_44	2429	2611	1888	1393	565	1250	9%	16%	4%	1%	24%	43%
	P82_10	1024	1129	624	189	460	862	6%	13%	7%	0%	11%	11%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

							Percentage torsion of direct shear					
Case	Total Shear a	b	c	d	a'	b'	Total Shear a	b	c	d	a'	b'
	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
P100_0	1043	1685	0	0	1140	1023	5%	70%	15%	3%	15%	3%
P88_8	866	1031	642	125	307	806	7%	22%	10%	5%	13%	14%
P93_16	984	1328	756	299	382	763	8%	39%	15%	12%	26%	18%
P78_22	1084	1624	852	475	452	706	8%	53%	17%	20%	36%	16%
P67_26	1178	1908	936	654	515	647	7%	63%	17%	27%	42%	12%
P59_29	1269	2179	1013	836	570	592	6%	70%	16%	33%	45%	7%
P53_32	1359	2439	1086	1019	621	543	5%	75%	15%	39%	47%	1%
P73_8	994	1015	625	139	405	853	3%	2%	3%	0%	4%	2%
P90_15	1168	1329	751	314	511	874	8%	18%	9%	1%	17%	14%
P94_20	1309	1680	865	486	620	862	9%	33%	13%	6%	30%	19%
P81_24	1433	2039	966	665	721	825	9%	45%	14%	12%	38%	19%
P71_27	1547	2392	1057	852	810	778	8%	55%	14%	18%	44%	17%
P63_29	1658	2735	1142	1045	889	727	7%	63%	14%	24%	46%	12%
P62_9	1140	1242	786	159	378	1089	3%	10%	2%	1%	3%	15%
P77_18	1276	1280	891	347	435	872	5%	1%	4%	0%	8%	1%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

	Case	Total Shear a	b	c	d	a'	b'	Percentage torsion of direct shear: Total Shear a	b	c	d	a'	b'
		(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
Case 2	*P91_24*	1449	1549	1029	515	537	887	8%	10%	9%	1%	23%	10%
	P95_28	1595	1876	1158	681	642	898	10%	22%	12%	4%	36%	18%
	P83_31	1724	2218	1275	853	742	889	10%	32%	14%	8%	46%	22%
	P74_34	1843	2564	1383	1032	834	866	9%	41%	15%	13%	54%	23%
	P54_10	1358	1453	958	179	372	1371	10%	15%	4%	2%	7%	29%
	P67_20	1358	1559	1016	385	381	1132	0%	11%	0%	0%	1%	18%
	P80_26	1562	1608	1162	560	465	936	6%	4%	5%	0%	14%	6%
	P92_31	1735	1753	1308	718	564	874	9%	4%	9%	1%	28%	5%
	P95_35	1884	2057	1447	877	666	902	10%	13%	12%	3%	42%	14%
	P85_37	2017	2380	1577	1042	766	913	11%	21%	14%	5%	54%	21%
	P48_10	1590	1628	1108	199	361	1640	16%	16%	4%	3%	7%	39%
	P59_21	1552	1784	1187	426	380	1402	4%	16%	2%	1%	5%	33%
	P71_28	1654	1890	1285	610	410	1188	3%	12%	2%	0%	6%	23%
	P82_33	1852	1951	1435	778	496	1006	7%	7%	6%	1%	20%	12%
	P93_37	2023	1974	1587	924	592	858	10%	1%	9%	0%	35%	1%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

							Percentage torsion of direct shear					
Case	Total Shear a	b	c	d	a'	b'	Total Shear a	b	c	d	a'	b'
	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
P96_40	2174	2232	1733	1075	692	881	11%	6%	12%	1%	49%	9%
P43_11	1829	1784	1236	219	347	1891	22%	16%	2%	3%	6%	46%
P53_22	1768	1973	1344	466	377	1662	9%	17%	2%	2%	8%	45%
P63_30	1748	2123	1415	662	376	1447	1%	16%	0%	0%	1%	39%
P74_35	1951	2232	1556	838	440	1251	5%	14%	3%	1%	11%	29%
P84_39	2143	2304	1711	1000	526	1081	8%	10%	6%	1%	26%	19%
P94_42	2312	2344	1866	1143	621	937	10%	5%	9%	1%	41%	8%
P39_12	2073	1928	1366	238	333	2124	27%	15%	1%	3%	4%	51%
P48_23	1991	2140	1486	507	371	1909	13%	18%	2%	2%	9%	54%
P58_31	1951	2321	1578	717	379	1700	4%	18%	1%	1%	5%	52%
P67_36	2037	2469	1677	894	398	1502	2%	17%	1%	0%	4%	45%
P76_41	2248	2582	1829	1068	471	1321	7%	15%	4%	1%	18%	36%
P85_44	2435	2664	1987	1225	557	1160	9%	12%	7%	1%	32%	26%
P82_10	1345	102	609	366	485	1627	39%	10%	4%	0%	17%	8%
P100_0	1005	331	0	0	1006	1978	2%	136%	2%	7%	2%	7%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

	Case	Total a	Shear b	c	d	a'	b'	Percentage torsion of direct shear Total a	Shear b	c	d	a'	b'
		(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
	P88_8	1133	32	637	247	322	1578	39%	19%	9%	4%	18%	12%
	P93_16	241	1729	1164	469	552	822	3%	101%	4%	60%	7%	15%
	P78_22	252	1974	1301	709	621	820	2%	102%	4%	65%	9%	24%
	P67_26	262	2211	1435	933	686	819	2%	102%	4%	69%	10%	32%
	P59_29	272	2442	1567	1146	748	817	1%	103%	3%	72%	10%	38%
	P53_32	281	2669	1699	1352	809	815	1%	103%	3%	74%	10%	43%
	P73_8	1508	32	617	276	427	1676	56%	1%	1%	0%	9%	1%
	P90_15	1298	264	714	575	501	1581	20%	15%	4%	1%	15%	12%
	P94_20	460	2211	1285	717	847	797	3%	90%	4%	44%	10%	2%
	P81_24	477	2543	1433	967	947	828	3%	94%	4%	52%	12%	12%
	P71_27	490	2857	1579	1209	1040	855	2%	97%	4%	58%	12%	20%
	P63_29	500	3160	1725	1444	1127	877	2%	99%	3%	62%	12%	28%
	P62_9	1820	38	864	314	373	2009	65%	4%	12%	0%	2%	6%
	P77_18	1638	260	858	636	470	1600	34%	1%	0%	0%	17%	1%
	P91_24	1444	496	958	884	477	1502	8%	8%	2%	1%	9%	9%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

	Case	Total Shear						Percentage torsion of direct shear					
		a	b	c	d	a'	b'	a	b	c	d	a'	b'
		(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
Case 3	*P95_28*	714	2459	1639	934	826	939	5%	71%	5%	33%	15%	15%
	P83_31	748	2819	1796	1176	916	840	4%	78%	5%	40%	18%	8%
	P74_34	779	3160	1948	1413	997	755	4%	83%	5%	46%	20%	1%
	P54_10	2115	43	1122	352	391	2328	71%	5%	22%	1%	12%	10%
	P67_20	1954	304	1072	706	428	1930	44%	6%	6%	0%	13%	10%
	P80_26	1753	520	1115	964	475	1590	19%	3%	1%	0%	16%	5%
	P92_31	1602	726	1200	1172	445	1411	1%	3%	0%	0%	1%	4%
	P95_35	84	3132	2553	1350	995	900	3%	83%	3%	48%	10%	7%
	P85_37	49	3471	2768	1604	1082	803	2%	87%	3%	53%	12%	0%
	P48_10	2401	48	1385	389	405	2619	75%	5%	29%	1%	21%	12%
	P59_21	2255	338	1305	777	388	2222	52%	7%	12%	1%	7%	15%
	P71_28	2056	593	1289	1048	457	1896	28%	8%	2%	0%	18%	14%
	P82_33	1880	804	1367	1269	460	1602	9%	5%	1%	0%	11%	9%
	P93_37	1910	967	1466	1447	471	1346	4%	1%	1%	0%	7%	1%
	P96_40	339	3342	2904	1548	971	1033	4%	68%	4%	38%	15%	20%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

							Percentage torsion of direct shear					
Case	Total a	Shear b	c	d	a'	b'	Total a	Shear b	c	d	a'	b'
	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(%)	(%)	(%)	(%)	(%)	(%)
P43_11	2683	52	1651	426	417	2892	78%	4%	36%	1%	28%	12%
P53_22	2547	370	1548	847	353	2485	57%	7%	18%	1%	1%	18%
P63_30	2351	651	1497	1137	433	2172	35%	9%	6%	0%	16%	21%
P74_35	2161	898	1519	1366	459	1890	16%	9%	1%	0%	16%	20%
P84_39	2022	1105	1614	1560	434	1631	2%	7%	0%	1%	4%	14%
P94_42	2239	1271	1737	1724	509	1400	7%	4%	2%	1%	16%	7%
P39_12	2961	57	1919	463	426	3155	81%	4%	41%	1%	34%	13%
P48_23	2834	399	1798	916	357	2728	61%	6%	23%	1%	5%	20%
P58_31	2639	703	1717	1227	408	2423	41%	9%	10%	1%	13%	26%
P67_36	2441	976	1705	1458	450	2156	23%	11%	3%	0%	18%	27%
P76_41	2281	1216	1763	1664	445	1905	8%	11%	0%	1%	11%	25%
P85_44	2279	1420	1869	1841	440	1671	2%	9%	0%	1%	4%	20%
Smith	2578	2654	355	1673	2174	1108	8%	11%	3%	15%	6%	18%

Table B.7 cont'd. Calculation of Torsional Shear Percentage.

Min	0%		
Average	16%		
Max	136%	Median	10%

Table B.8. Calculated Diaphragm Flexibility for 4:12 Gable Roofs.

		Diaphragm A			Diaphragm B			Diaphragm C			Diaphragm D		
	Case	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**
Case 1	P82_10	0.33	0.17	1.99	0.41	0.16	2.58	0.28	0.13	2.18	0.68	0.10	6.83
	P100_0	####	####	####	0.69	####	0.00	####	####	####	0.63	####	0.00
	P88_8	0.30	0.21	1.45	0.35	0.20	1.75	0.23	0.15	1.56	0.66	0.10	6.50
	P93_16	0.33	0.12	2.79	0.48	0.11	4.36	0.29	0.16	1.79	0.74	0.11	6.79
	P78_22	0.35	0.09	3.71	0.61	0.08	7.24	0.23	0.17	1.36	0.81	0.12	7.00
	P67_26	0.37	0.08	4.41	0.75	0.07	10.34	0.75	0.18	4.18	0.87	0.12	7.16
	P59_29	0.39	0.08	5.03	0.89	0.07	13.63	0.78	0.19	4.19	0.92	0.13	7.29
	P53_32	0.41	0.07	5.61	1.03	0.06	17.08	0.81	0.19	4.20	0.97	0.13	7.41
	P73_8	0.37	0.22	1.70	0.43	0.22	1.99	0.28	0.12	2.30	0.68	0.09	7.37
	P90_15	0.35	0.13	2.82	0.49	0.12	4.22	0.32	0.13	2.47	0.76	0.10	7.67
	P94_20	0.39	0.10	3.84	0.62	0.09	6.94	0.39	0.14	2.81	0.85	0.11	7.99
	P81_24	0.42	0.09	4.63	0.77	0.08	9.92	0.68	0.15	4.53	0.93	0.11	8.21
	P71_27	0.45	0.09	5.28	0.92	0.07	13.08	0.73	0.16	4.56	1.00	0.12	8.38
	P63_29	0.48	0.08	5.86	1.08	0.07	16.37	0.78	0.17	4.59	1.06	0.12	8.52
	P62_9	0.64	0.23	2.73	0.51	0.23	2.26	0.32	0.11	2.79	0.78	0.08	9.74
	P77_18	0.61	0.13	4.55	0.59	0.12	4.73	0.32	0.12	2.71	0.78	0.08	9.57
	P91_24	0.59	0.11	5.59	0.64	0.09	6.84	0.38	0.13	3.05	0.89	0.09	10.10
	P95_28	0.61	0.09	6.47	0.74	0.08	9.39	0.68	0.13	5.03	0.99	0.09	10.47
	P83_31	0.63	0.09	7.20	0.87	0.07	12.39	0.73	0.14	5.07	1.07	0.10	10.74
	P74_34	0.66	0.08	7.81	1.02	0.07	15.58	0.78	0.15	5.11	1.15	0.10	10.95
	P54_10	0.71	0.25	2.87	0.58	0.24	2.46	0.35	0.11	3.17	0.89	0.07	12.28
	P67_20	0.69	0.15	4.73	0.68	0.13	5.17	0.35	0.11	3.14	0.88	0.07	12.00
	P80_26	0.66	0.12	5.77	0.75	0.10	7.56	0.38	0.12	3.26	0.93	0.08	12.13
	P92_31	0.64	0.10	6.50	0.81	0.08	9.77	0.67	0.12	5.42	1.04	0.08	12.68
	P95_35	0.63	0.09	7.11	0.85	0.07	11.90	0.72	0.13	5.46	1.14	0.09	13.08
	P85_37	0.66	0.09	7.76	0.98	0.07	14.92	0.78	0.14	5.52	1.23	0.09	13.40
	P48_10	0.78	0.26	3.00	0.65	0.25	2.62	0.36	0.10	3.47	1.00	0.07	14.98

Table B.8 cont'd. Calculated Diaphragm Flexibility for 4:12 Gable Roofs.

	Diaphragm A			Diaphragm B			Diaphragm C			Diaphragm D		
Case	Dia. Defl (in)	Ave. SW Defl (in)	Ratio D/S	Dia. Defl (in)	Ave. SW Defl (in)	Ratio D/S	Dia. Defl (in)	Ave. SW Defl (in)	Ratio D/S	Dia. Defl (in)	Ave. SW Defl (in)	Ratio D/S
P59_21	0.76	0.16	4.88	0.76	0.14	5.48	0.38	0.11	3.50	0.99	0.07	14.60
P71_28	0.74	0.12	5.94	0.85	0.11	8.06	0.39	0.11	3.56	0.99	0.07	14.36
P82_33	0.72	0.11	6.67	0.93	0.09	10.51	0.67	0.12	5.75	1.08	0.07	14.84
P93_37	0.70	0.10	7.26	0.98	0.08	12.87	0.71	0.12	5.80	1.19	0.08	15.40
P96_40	0.69	0.09	7.78	1.03	0.07	15.13	0.77	0.13	5.86	1.30	0.08	15.84
P43_11	0.84	0.27	3.10	0.70	0.26	2.76	0.39	0.10	3.83	1.13	0.06	17.81
P53_22	0.83	0.17	5.01	0.83	0.15	5.72	0.40	0.11	3.80	1.10	0.06	17.35
P63_30	0.81	0.13	6.09	0.94	0.11	8.43	0.43	0.11	3.93	1.10	0.06	17.02
P74_35	0.79	0.12	6.84	1.03	0.09	11.05	0.67	0.11	6.04	1.12	0.07	16.98
P84_39	0.77	0.10	7.43	1.11	0.08	13.61	0.71	0.12	6.10	1.24	0.07	17.70
P94_42	0.75	0.09	7.94	1.17	0.07	16.11	0.76	0.12	6.17	1.36	0.07	18.27
P39_12	0.90	0.28	3.19	0.76	0.26	2.88	0.43	0.10	4.27	1.27	0.06	20.76
P48_23	0.90	0.18	5.13	0.90	0.15	5.92	0.43	0.10	4.09	1.22	0.06	20.24
P58_31	0.89	0.14	6.22	1.02	0.12	8.72	0.45	0.11	4.24	1.21	0.06	19.82
P67_36	0.87	0.12	6.98	1.13	0.10	11.46	0.69	0.11	6.33	1.21	0.06	19.54
P76_41	0.85	0.11	7.59	1.22	0.09	14.17	0.71	0.11	6.38	1.29	0.06	19.98
P85_44	0.83	0.10	8.10	1.30	0.08	16.85	0.76	0.12	6.45	1.41	0.07	20.69
P82_10	0.36	0.17	2.08	0.46	0.17	2.75	0.29	0.15	1.97	0.71	0.11	6.24
P100_0	####	####	####	0.87	####	0.00	####	####	####	0.63	####	0.00
P88_8	0.33	0.22	1.50	0.41	0.21	1.93	0.24	0.18	1.36	0.68	0.11	5.90
P93_16	0.34	0.12	2.82	0.56	0.12	4.84	0.29	0.19	1.51	0.75	0.12	6.07
P78_22	0.35	0.10	3.67	0.71	0.09	7.96	0.22	0.20	1.10	0.80	0.13	6.18
P67_26	0.37	0.08	4.33	0.87	0.08	11.26	0.73	0.21	3.46	0.85	0.14	6.28
P59_29	0.38	0.08	4.91	1.02	0.07	14.72	0.76	0.22	3.44	0.90	0.14	6.36
P53_32	0.40	0.07	5.47	1.17	0.06	18.35	0.78	0.23	3.42	0.95	0.15	6.44
P73_8	0.35	0.23	1.53	0.41	0.22	1.84	0.28	0.14	2.00	0.68	0.10	6.70
P90_15	0.39	0.13	3.00	0.56	0.12	4.63	0.33	0.15	2.19	0.77	0.11	6.92

Table B.8 cont'd. Calculated Diaphragm Flexibility for 4:12 Gable Roofs.

		Diaphragm A			Diaphragm B			Diaphragm C			Diaphragm D		
	Case	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**
Case 2	*P94_20*	0.42	0.10	3.98	0.73	0.09	7.72	0.40	0.17	2.39	0.85	0.12	7.04
	P81_24	0.44	0.09	4.70	0.91	0.08	11.01	0.67	0.18	3.78	0.92	0.13	7.11
	P71_27	0.45	0.09	5.29	1.09	0.08	14.45	0.71	0.19	3.73	0.98	0.14	7.16
	P63_29	0.47	0.08	5.82	1.27	0.07	18.00	0.75	0.20	3.70	1.04	0.14	7.20
	P62_9	0.61	0.24	2.59	0.50	0.23	2.15	0.30	0.13	2.37	0.74	0.09	8.65
	P77_18	0.58	0.13	4.39	0.55	0.12	4.44	0.33	0.14	2.44	0.81	0.09	8.84
	P91_24	0.61	0.11	5.67	0.68	0.09	7.22	0.40	0.15	2.68	0.91	0.10	9.09
	P95_28	0.63	0.10	6.61	0.85	0.08	10.36	0.68	0.16	4.25	0.99	0.11	9.24
	P83_31	0.65	0.09	7.33	1.02	0.07	13.72	0.72	0.17	4.22	1.06	0.11	9.34
	P74_34	0.67	0.08	7.93	1.20	0.07	17.26	0.76	0.18	4.20	1.13	0.12	9.42
	P54_10	0.69	0.25	2.77	0.58	0.24	2.42	0.34	0.12	2.77	0.84	0.08	10.91
	P67_20	0.65	0.14	4.63	0.66	0.13	5.08	0.34	0.13	2.66	0.85	0.08	10.72
	P80_26	0.63	0.11	5.68	0.72	0.10	7.33	0.39	0.14	2.90	0.96	0.09	11.13
	P92_31	0.63	0.10	6.55	0.80	0.08	9.84	0.68	0.15	4.64	1.05	0.09	11.40
	P95_35	0.66	0.09	7.33	0.96	0.07	13.02	0.72	0.16	4.62	1.14	0.10	11.58
	P85_37	0.68	0.09	7.97	1.12	0.07	16.44	0.77	0.17	4.61	1.22	0.10	11.70
	P48_10	0.76	0.26	2.94	0.65	0.25	2.63	0.37	0.12	3.12	0.94	0.07	13.34
	P59_21	0.73	0.15	4.87	0.76	0.14	5.54	0.37	0.12	3.02	0.94	0.07	13.02
	P71_28	0.70	0.12	5.94	0.84	0.10	8.09	0.40	0.13	3.12	1.00	0.08	13.16
	P82_33	0.68	0.10	6.68	0.89	0.09	10.43	0.68	0.14	4.97	1.11	0.08	13.56
	P93_37	0.66	0.09	7.27	0.93	0.07	12.61	0.72	0.15	4.96	1.21	0.09	13.84
	P96_40	0.68	0.09	7.94	1.07	0.07	15.74	0.77	0.16	4.96	1.30	0.09	14.04
	P43_11	0.84	0.27	3.09	0.72	0.26	2.80	0.39	0.11	3.43	1.05	0.07	15.92
	P53_22	0.81	0.16	5.07	0.84	0.14	5.88	0.40	0.12	3.35	1.04	0.07	15.50
	P63_30	0.78	0.13	6.17	0.94	0.11	8.65	0.41	0.12	3.36	1.05	0.07	15.25
	P74_35	0.75	0.11	6.92	1.02	0.09	11.26	0.68	0.13	5.27	1.16	0.07	15.72
	P84_39	0.73	0.10	7.51	1.08	0.08	13.74	0.72	0.14	5.27	1.27	0.08	16.11

Table B.8 cont'd. Calculated Diaphragm Flexibility for 4:12 Gable Roofs.

	Diaphragm A			Diaphragm B			Diaphragm C			Diaphragm D		
Case	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**
P94_42	0.71	0.09	8.02	1.13	0.07	16.10	0.77	0.15	5.27	1.37	0.08	16.40
P39_12	0.91	0.28	3.23	0.78	0.26	2.95	0.41	0.11	3.73	1.16	0.06	18.63
P48_23	0.89	0.17	5.25	0.92	0.15	6.16	0.42	0.12	3.65	1.15	0.06	18.11
P58_31	0.86	0.13	6.38	1.03	0.11	9.08	0.44	0.12	3.69	1.15	0.06	17.78
P67_36	0.83	0.12	7.15	1.13	0.10	11.89	0.68	0.12	5.55	1.21	0.07	17.90
P76_41	0.81	0.10	7.75	1.21	0.08	14.60	0.72	0.13	5.55	1.32	0.07	18.41
P85_44	0.78	0.09	8.25	1.28	0.07	17.23	0.77	0.14	5.56	1.43	0.08	18.79
P82_10	0.69	0.20	3.37	0.11	0.62	0.17	0.30	0.15	2.01	0.83	0.12	6.94
P100_0	####	####	####	0.36	####	0.00	####	####	####	0.61	####	0.00
P88_8	0.65	0.25	2.61	0.06	1.63	0.04	0.25	0.18	1.38	0.78	0.12	6.44
P93_16	0.40	0.13	3.03	0.75	0.14	5.42	0.43	0.22	1.99	0.54	0.33	1.65
P78_22	0.44	0.11	4.17	0.90	0.11	8.39	0.32	0.23	1.41	0.57	0.35	1.62
P67_26	0.49	0.09	5.14	1.05	0.09	11.49	0.84	0.24	3.46	0.60	0.38	1.59
P59_29	0.53	0.09	6.01	1.20	0.08	14.76	0.87	0.26	3.43	0.63	0.40	1.56
P53_32	0.57	0.08	6.82	1.35	0.07	18.21	0.91	0.27	3.41	0.66	0.43	1.54
P73_8	0.69	0.26	2.64	0.06	1.66	0.04	0.29	0.14	2.05	0.88	0.11	7.88
P90_15	0.71	0.16	4.50	0.21	0.30	0.69	0.32	0.15	2.14	0.82	0.11	7.21
P94_20	0.44	0.11	3.93	0.99	0.12	8.51	0.56	0.19	2.95	0.61	0.24	2.56
P81_24	0.50	0.10	4.94	1.18	0.10	11.64	0.79	0.21	3.84	0.64	0.26	2.48
P71_27	0.55	0.09	5.85	1.36	0.09	14.86	0.84	0.22	3.78	0.67	0.28	2.39
P63_29	0.61	0.09	6.69	1.55	0.09	18.21	0.88	0.24	3.73	0.71	0.31	2.31
P62_9	0.87	0.28	3.13	0.07	1.68	0.04	0.32	0.13	2.45	1.02	0.10	10.39
P77_18	0.81	0.16	4.92	0.22	0.30	0.71	0.34	0.14	2.47	0.95	0.10	9.76
P91_24	0.81	0.13	6.13	0.35	0.19	1.87	0.36	0.14	2.51	0.89	0.10	9.09
P95_28	0.67	0.10	6.58	1.13	0.10	11.36	0.79	0.18	4.35	0.74	0.18	4.13
P83_31	0.68	0.09	7.23	1.34	0.09	14.70	0.83	0.19	4.31	0.79	0.19	4.08
P74_34	0.69	0.09	7.82	1.54	0.08	18.16	0.88	0.21	4.27	0.83	0.21	4.03

Table B.8 cont'd. Calculated Diaphragm Flexibility for 4:12 Gable Roofs.

	Case	Diaphragm A			Diaphragm B			Diaphragm C			Diaphragm D		
		Dia. Defl (in)	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**	**Dia. Defl (in)**	**Ave. SW Defl (in)**	**Ratio D/S**
Case 3	*P54_10*	0.96	0.30	3.25	0.08	1.70	0.05	0.38	0.13	3.05	1.17	0.09	12.97
	P67_20	0.90	0.18	5.10	0.25	0.32	0.78	0.36	0.13	2.82	1.09	0.09	12.37
	P80_26	0.85	0.14	6.12	0.38	0.19	1.95	0.39	0.14	2.87	1.02	0.09	11.65
	P92_31	0.83	0.12	6.94	0.49	0.14	3.42	0.63	0.14	4.59	0.98	0.09	11.07
	P95_35	0.72	0.10	7.25	1.51	0.10	15.40	0.96	0.20	4.84	0.66	0.89	0.74
	P85_37	0.73	0.09	7.86	1.73	0.09	18.92	1.01	0.21	4.81	0.68	1.56	0.44
	P48_10	1.05	0.31	3.35	0.09	1.72	0.05	0.45	0.12	3.67	1.32	0.08	15.66
	P59_21	0.99	0.19	5.23	0.27	0.33	0.83	0.39	0.12	3.18	1.24	0.08	15.08
	P71_28	0.94	0.15	6.30	0.42	0.20	2.08	0.42	0.13	3.20	1.16	0.08	14.34
	P82_33	0.89	0.13	7.05	0.54	0.15	3.60	0.66	0.13	4.95	1.11	0.08	13.65
	P93_37	0.85	0.11	7.64	0.64	0.12	5.27	0.68	0.14	4.92	1.14	0.08	13.56
	P96_40	0.78	0.10	8.08	1.65	0.09	18.49	1.00	0.19	5.23	0.80	0.30	2.72
	P43_11	1.13	0.33	3.44	0.10	1.73	0.06	0.53	0.12	4.32	1.48	0.08	18.46
	P53_22	1.08	0.20	5.32	0.30	0.34	0.88	0.43	0.12	3.56	1.40	0.08	17.88
	P63_30	1.03	0.16	6.43	0.46	0.21	2.19	0.45	0.13	3.55	1.31	0.08	17.13
	P74_35	0.99	0.14	7.22	0.60	0.16	3.79	0.68	0.13	5.25	1.24	0.08	16.38
	P84_39	0.95	0.12	7.84	0.71	0.13	5.58	0.69	0.13	5.24	1.20	0.08	15.77
	P94_42	0.91	0.11	8.37	0.81	0.11	7.48	0.72	0.14	5.23	1.31	0.08	16.23
	P39_12	1.22	0.35	3.52	0.11	1.75	0.06	0.61	0.12	4.99	1.64	0.08	21.36
	P48_23	1.16	0.22	5.40	0.32	0.35	0.93	0.48	0.12	4.03	1.56	0.08	20.80
	P58_31	1.12	0.17	6.54	0.50	0.22	2.29	0.48	0.12	3.91	1.46	0.07	20.02
	P67_36	1.08	0.15	7.36	0.64	0.16	3.95	0.70	0.13	5.55	1.38	0.07	19.22
	P76_41	1.04	0.13	8.01	0.78	0.13	5.82	0.71	0.13	5.52	1.32	0.07	18.53
	P85_44	1.00	0.12	8.56	0.89	0.11	7.84	0.72	0.13	5.53	1.34	0.07	18.34
	Smith	0.92	0.21	4.30	1.17	0.17	6.85	0.91	0.24	3.85	1.14	0.23	4.90

Table B.8 cont'd. Calculated Diaphragm Flexibility for 4:12 Gable Roofs.

Individual	**Flex**	127	109	122	122
	Semi-rigid	5	23	10	10
All diaphragms	**Flex**				96
	Semi-rigid				0