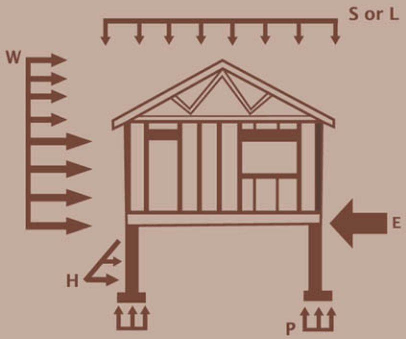

residential building loads

review and roadmap for future progress

Prepared by Special Project Committee on Residential Building Loads of the Structural Engineering Institute (SEI) of ASCE

Edited by Jay H. Crandell, P.E., Thomas M. Kenney, P.E., and David V. Rosowsky, Ph.D., P.E.

ASCE

SEI
Structural Engineering Institute
of the American Society of Civil Engineers

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PREPARED BY
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WITH SUPPORT PROVIDED BY
National Association of Home Builders
U.S. Department of Housing and Urban Development

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Published by the American Society of Civil Engineers

Library of Congress Cataloging-in-Publication Data

Residential building loads : review and roadmap for future progress / prepared by Special Project Committee on Residential Building Loads of the Structural Engineering Institute (SEI) of ASCE with support provided by National Association of Home Builders, U.S. Department of Housing and Urban Development.

p. cm.

Includes bibliographical references and index.

ISBN 0-7844-0852-1

1. House construction. 2. Loads (Mechanics) 3. Architecture, Domestic. I. Structural Engineering Institute. Special Project Committee on Residential Building Loads.

TH4812.R47 2006
728--dc22 2006012208

American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, Virginia 20191

www.pubs.asce.org

ISBN 0-7844-0852-1
Manufactured in the United States of America.

15 14 13 12 11 10 09 08 07 2 3 4 5

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Acknowledgments

Special appreciation is extended to the Structural Engineering Institute of the American Society of Civil Engineers for the support provided in sponsoring this special project on residential building loads. In addition, this project would not have been possible without the support provided by the National Association of Home Builders and the U.S. Department of Housing and Urban Development. The special project committee members were as follows:

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- Dr. Timothy Reinhold, Clemson University
- Brad Douglas, American Forest and Paper Association
- Jeff Sciandone, Institute for Business and Home Safety
- William Freeborne, U.S. Department of Housing and Urban Development (retired)
- Jim Rossberg, Structural Engineering Institute (ASCE)

The committee also wishes to express its gratitude to the following individuals and several others who contributed to this effort with their comments and assistance.

- Richard Zimmerman and Kirk Grundahl, Wood Truss Council of America
- Charlie Goehring and Kelly Gutting, Truss Plate Institute
- Mark Nunn, Manufactured Housing Institute
- Dr. Michael J. O'Rourke, Rensselaer Polytechnic Institute
- Thomas Kenney, Jay Jones, and Vladimir Kochkin, NAHB Research Center, Inc.

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INTRODUCTION

The topic of structural design loads for buildings has experienced continued interest and numerous technical advancements over the past 80 years. The process of establishing and maintaining uniformity in building loads began in 1924 as a building code committee report by the U.S. Department of Commerce. However, the most significant technical achievements have occurred over the past 20 to 30 years. The collective knowledge on this topic consists of design and construction experience, judgment, data on actual loading or hazard characteristics, and laboratory testing or analytical methods of simulating building loads. Probabilistic methods now provide a general framework with which to communicate, evaluate, and establish design loads for an acceptable level of building performance or reliability in coordination with various material design specifications. However, many design load criteria are still deterministically-based and provide opportunities for further expansion of probabilistic or performance-based methods of design.

In the United States, the body of applied research on structural loads is now largely represented in *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002), known simply as ASCE 7. This consensus-based engineering standard has become the preeminent source for building structural design loads in the United States. It also serves as a focal point for the application of new information on structural loads, as well as a repository for significant technical contributions that have occurred over time. As such, the ASCE 7 standard relies on a large number of technical resources and judgments to provide rules for determining loads for a wide variety of structural design applications.

In recent years, the application of structural loads for residential building design has seen increased interest for a number of reasons. Technical reasons relate to real or perceived needs for specialized research on residential building loads and applications of this research. In addition, changing house styles, materials, and regulations point toward an increased demand for specialized design of homes, particularly in high natural hazard areas of the United States. Functional reasons relate to simplicity, clarity, and specificity of loading requirements for a narrow scope of building design applications and practices, such as dwellings, which represent a large proportion of building activity.

PURPOSE

In response to the above concern, this document is intended to serve as both a technical resource and a roadmap for coordination of future research and implementation efforts. As a technical resource, this report identifies and reviews existing knowledge on a total of 14 topics related to design loads for residential building applications. As a roadmap, this document provides strategic recommendations to address technical and functional needs. The recommendations provided are based primarily on existing knowledge available to and the experience of the special project committee. Potential economic impacts to design, construction, and housing affordability are not specifically addressed. Technical references are provided at the end of this document.

While all topics were researched, reviewed, and discussed by the special project committee, not all findings reported herein received unanimous support. The intent of this work was to identify needs and concerns related to residential building loads as a road mapping exercise. Due to the subjective nature of this type of activity, differing opinions were anticipated. However, one of the main goals of this work was to review and compile existing knowledge pertaining to residential building loads such that the consideration of research and implementation needs may guided by a careful and consistent understanding of existing knowledge associated with each need or topic identified by the committee. While significant progress has been made toward this goal, more work is needed and this realization is reflected in many of the recommendations presented in this document. In some cases, there is a need for additional literature review and research. These types of recommendations are directed to those in the business of directing, sponsoring, and conducting research. In other cases, the need may be more appropriately categorized as implementation (e.g., interpretation and application of existing knowledge). Recommendations in the latter case are strategically directed toward appropriate consensus standards committees, such as ASCE 7, as the final decision-making entity.

SCOPE

The types of structures considered within this effort are one- and two-family dwellings and manufactured housing units (i.e., HUD-code dwellings); however, many of the design load topics herein are not necessarily limited to this scope of buildings. As a group, these buildings comprise a special class of low-rise construction because they are typically less than three stories in height and are usually constructed using conventional light-frame materials and methods (including dimension lumber, masonry, concrete, engineered wood components, and steel components). Building configurations are generally rectangular in plan and use steep-sloped roof systems (e.g., gable roof). By definition, HUD-code homes are further limited to one story in height although two-story HUD-code homes may be permitted through an alternative construction process. Collectively, these types of buildings dominate the residential construction market and the overall population of buildings in the United States.

While this paper focuses on the building load side of the design equation, it is not intended to diminish the equally important need to seek specialized advancement in understanding the structural system behavior (response to loads) of typical residential buildings (Crandell and Kochkin, 2003; HUD, 2000). This need is perhaps of greater significance and even more challenging to address, but is beyond the scope of this document. However, it is insightful to recognize all potential sources of uncertainty and bias in the overall design process which strives to integrate knowledge regarding loads and resistance in a practical manner. For example, in Queensland, Australia, wall bracing requirements in building regulations were relaxed by approximately 50 percent after full-scale tests indicated that "non-structural" elements provided sufficient bracing (St. Pierre, et al., 2003). Because of the unpredictable nature of system effects, appropriate solutions to account for these effects are difficult to generalize and usually require the exercise of judgment based on relevant experience or data. Thus, system effects pose technical challenges as well as political challenges in terms of fostering acceptance of proposed solutions among those who may have differing experiences, knowledge, and motives.

The topic of earthquake loads, except as affected by Topic #2, Minimum Partition Wall Weight for Seismic Load Analysis, is not addressed in this document. It was expressed by some members of the special project committee that the topic of earthquake loads on residential buildings should be addressed within other existing programs such as the National Earthquake Hazard Reduction Program (BSSC, 2000). Furthermore, earthquake loads and structural response (resistance) are inextricably linked in such a way that loads cannot be addressed separately from structural behavior and physical properties (e.g., mass). Thus, this topic inherently includes issues that are considered to be beyond the scope of this special project activity on residential building loads.

APPROACH

The approach used in this special project activity was as follows:

1. Convene a select group of experts to participate on the special project committee.
2. Identify and discuss existing technical knowledge on select residential building load topics.
3. Consider design loads for residential buildings as contained in existing standards and building codes.
4. Formulate research and implementation recommendations aimed at improving the characterization and application of residential building loads.

Two special project committee meetings were held over the course of about two years from 2002 to 2003. The first meeting served to identify topics to investigate. The second meeting and subsequent communications focused on reviewing and drafting this document.

BACKGROUND

Structural load provisions specifically intended for design of one- and two-family dwellings have been developed and implemented in the past. One early attempt is found in a government housing construction guide intended to bring uniformity to construction practices for homes (HEW, 1931). Structural loads as well as wood material design guidance were included. Guidance on building loads, however, focused primarily on recommended gravity load design values based at least in part on earlier determinations by the Building Code Committee of the U.S. Department of Commerce in 1923. Some of the gravity load criteria found in this 1931 publication are excerpted as follows:

Table 1 – Recommended live loads and dead weight per square foot of construction such as flooring, roofing, plaster, and the like

Live load, all floors used for living purposes	40 psf
Live load for attic (used for light storage only)	20 psf
Dead weight (called dead load) for average double floor and joists, but without plaster	10 psf
Dead weight of plaster ceiling, including joists on light unfloored attics	10 psf
Roof of light construction, including both live and dead loads	20 psf
Roof of medium construction with light slate or asbestos roofing, including both live and dead loads	30 psf
Roof of heavy construction with heavy slate or tile roofing, including both live and dead loads	40 psf

Although a live load of 40 pounds per square foot should be used in selecting all floor joists, such a load will not occur over a large floor area at the same time. The larger the area, the less chance there is of its being heavily loaded all over. In fact, the Building Code Committee of the U.S. Department of Commerce, in 1923, after careful investigation, recommended that, in computing the load on girders carrying floors more than 200 square feet in area, a live load of 30 pounds per square foot is used.

In the same publication, lateral bracing for wind loading was addressed subjectively as follows:

> *"Bracing is any means taken to stiffen a building against a tendency to lean or collapse as the result of high winds or the effects of time. It is of prime importance in sections of the country subject to wind storms."*

No clear definition of lateral wind loads was given nor was a magnitude specified. And, there was no recognition of wind uplift or earthquake lateral loads. Snow loads were grouped with live loads and dead loads were provided as nominal design values.

In 1948, the National Bureau of Standards (NBS, now known as the National Institute of Standards and Technology (NIST)) published *Building Materials and Structures Report 109* entitled *Strength of Houses: Application of Engineering Principles to Structural Design* (NBS, 1948). The report embodied the first comprehensive attempt to evaluate the actual structural performance of various housing constructions, develop suitable design load criteria, and apply engineering principles to the design of one- and two-family dwellings. Over 100 different construction assemblies reflective of practices at that time were weighed for dead load and tested for transverse, compression, and racking load effects. The proposed methodology to determine design loads included a wind velocity pressure map, a snow load map, and a review of existing building load practices and data. The NBS Report 109 also provided the rationale behind various recommended design loads. For example, the following excerpt explains the origination of the floor live load value still used in the design of homes today:

> *"After careful study, several competent architects and builders stated that for residential occupancy the furniture loads seldom exceed 15 psf uniformly distributed, but that the load caused by a crowd of people averaged 40 psf and might occur in any room at any time. It is evident, therefore, that the floor of a house should be designed for a uniformly distributed load of 40 psf."*

Furthermore, the report also relies on prior sources as shown below:

> *"A load of 20 psf on an attic floor used for light storage only is recommended in Light Frame House Construction."*

In turn, the values referenced in *Light Frame House Construction* (HEW, 1931) appear to be based on the determinations made in 1923 by the Building Code Committee of the U.S. Department of Commerce in its effort to establish uniformity of design loads.

By 1958, the Federal Housing Administration (FHA, now known as the U.S. Department of Housing and Urban Development or HUD) had formulated the first comprehensive building code for one- and two-family dwelling construction known as the *Minimum Property Standards* or MPS (FHA, 1958). This document was comprehensive in all respects and was developed and formatted to include performance objective statements and associated construction criteria. It was based on *Building Code Requirements for New Dwelling Construction* (NBS, 1946) produced by the National Housing Agency in consultation with the National Bureau of Standards. The 1958 MPS document included an appendix entitled "Structural Design Data" which provided the basis for engineering of individual homes or portions of homes not meeting the limitations of prescriptive requirements. It also served as a performance basis for evaluating alternative constructions. In addition, the structural load requirements were simplified to meet the intended application and scope of the MPS (see Appendix A to this document). For conditions outside of the scope of the "Structural Design Data" appendix, users were referred to American Standards Association (ASA, now known as the American National Standards Institute (ANSI)) standard A58.1 "Minimum Design Loads in Buildings and Other Structures" (a precursor of the ASCE 7 standard). The load criteria in the MPS appear to have been based on simplifications of or extractions from the A58.1 standard which probably extends to judgments made previously, dating as far back to the 1923 Building Codes Committee of the U.S. Department of Commerce.

By the 1970's, several changes had taken place in the regulation of residential building construction and design. First, the effort to maintain and periodically update the HUD Minimum Property Standards was terminated. It was replaced by a residential building code developed as a joint effort of existing building official organizations operating as the Council of American Building Officials (CABO). However, the CABO code was never fully adopted by all states and local jurisdictions. For example, the Uniform Building Code (UBC) in the western states maintained its own provisions for conventional wood frame construction which addressed one- and two-family dwellings as well as other low-rise building types. While updated several times, these UBC provisions are still in use today (ICBO, 1997). More importantly, the simplified engineering design and performance criteria for one- and two-family dwellings were not carried forward from the *Minimum Property Standards* era. Eventually, the CABO code and the major model building codes in the United States were subject to yet another major change in organization which is addressed in the next section.

In the 1970's, the manufactured housing industry also experienced a major change in regulatory process and requirements. By 1974, 24 CFR Part 3280 of the Code of Federal Regulations (CFR) was created to provide for the design and construction of manufactured housing units under the authority of the National Manufactured Housing Construction and Safety Standards Act of 1974. As such, it addresses load criteria, test criteria, and other matters related to the production and installation of manufactured housing units. For wind loads, the current regulation specifically references the 1988 edition of the ASCE 7 standard. Other loads are specified directly in 24 CFR Part 3280. More recently, the NFPA 501 standard has been developed to serve as a resource for updating the provisions of 24 CFR Part 3280 (NFPA, 2000).

Meanwhile, the development of building design load regulations in the United States was progressing. Some key benchmarks are summarized as follows:

1924 – Building Code Committee of U.S. Department of Commerce (DOC) publishes a report to establish uniform design loads for the United States.

1945 – American Standards Association (ASA) A58.1 standard is published with the title *Building Code Requirements for Minimum Design Loads*. The A58.1 standard broadened the scope of the 1924 DOC report to include weights of materials and equipment, occupants, and moveable contents; wind pressures; weight of snow; and earthquake forces.

1955 – A58.1 standard updated.

1972 – A58.1 standard was updated a second time and sections on wind and snow loads were expanded based on rapid research advancements providing a statistical basis for fundamental wind and snow load variables.

1982 – A58.1 standard was further expanded to include performance requirements for general structural integrity and probability-based load combinations for strength design.

In addition, several other changes were made:

- Some live loads were revised and a new live-load reduction procedure based on load survey data and probabilistic live load modeling was included.

- The wind map was revised based on analysis of extreme fastest-mile winds and pressure coefficients were updated based on wind tunnel studies and actual wind pressure measurements on buildings.
- Snow loads were improved based on a greatly expanded database of ground snow loads and recently completed measurements of snow accumulation on roofs (in addition, thermal effects were numerically considered for the first time).
- Earthquake loads were updated to account for an Applied Technology Council effort to develop comprehensive seismic regulations for buildings, including a new seismic risk map, while retaining features of the 1979 edition of the Uniform Building Code.

1988 to present – The first edition of the ASCE 7 standard, *Minimum Design Loads for Buildings and Other Structures,* replaced the A58.1 standard. Subsequent editions of ASCE 7 were published in 1993, 1995, 1998, and 2002. Revisions for a 2005 edition were in progress at the time of this writing. The 1993 provisions primarily addressed an update to earthquake provisions based on the National Earthquake Hazard Reduction Program's recommended provisions for seismic regulations. Subsequent editions have introduced numerous incremental advancements and expansions of load requirements. For example, flood and atmospheric ice loads have been recently added.

The above sequence of technical improvements to the ASCE 7 standard has not been without consequence. For example, many users of ASCE 7 and members of the committee have become concerned with its complexity and length. The 1982 edition of ANSI A58.1 was 45 pages in length (not including commentary). The 2002 edition of ASCE 7 is now 215 pages in length (not including commentary). While there are many good reasons for the increased length of ASCE 7 (e.g., expansion of scope, expansion of knowledge, etc.), the volume of information and its complexity may interfere with efficient and accurate (or adequate) use of its provisions, many of which may not apply to specific or common applications. Therefore, simplicity is a topic that is addressed later in this document in regard to residential design applications (see Topic #13). Educational efforts and simplicity are the two "tools" by which users can be influenced to properly and consistently apply the content of ASCE 7. Furthermore, specific needs may vary depending upon user groups and the nature and extent of their design applications.

EXISTING SITUATION

By the year 2000, the three major model building code organizations in the United States had joined together and developed a single national model building code, the International Building Code (ICC, 2000). As a part of this effort, a separate residential building code was also continued. This new code, the International Residential Code (IRC), was derived primarily from the previous CABO and UBC codes (ICC, 2000). In addition, many changes to engineering and construction criteria were introduced during the drafting and subsequent code development process as a result of concerns with earthquake and wind resistant construction of housing.

Structural performance concerns with residential construction, as in the past, were also attended by a strong desire to rationalize all conventional construction provisions using engineering principles as represented in standardized engineering criteria for loads and resistance. Indeed, some conventional construction practices have long been rationalized by using engineering principles (e.g., header span tables, joist span tables, etc.). In other cases, standard engineering methods used in the development of newer IRC provisions to improve conventional construction practices may not accurately model system performance effects in conventional residential construction (HUD, 2000; Crandell and Kochkin, 2003). For example, the lateral resistance of conventional residential construction has been found to be substantially stronger than predictions made by code-compliant engineering calculation in a number of whole building tests (HUD, 2000; HUD, 2001).

Many of the above issues in the engineering of conventional residential construction have been recently reviewed in detail and many of the issues are actually continuations of technical challenges that have been known for more than a half century (Crandell and Kochkin, 2003; NBS, 1948). Because of these unanswered technical challenges and differing opinions on the application of engineering principles to housing design, varying engineering design decisions with regard to loads and resistance for residential buildings continue to be made in professional practice and are implicit in the development of prescriptive construction requirements for conventional residential construction in the IRC. This situation points to the need for continuing research as well as uniformity, clarity, and simplicity in unique design procedures for housing.

As mentioned in the Background section, a straight-forward approach to housing design was incorporated as a part of the *Minimum Property Standards* promulgated by HUD in the 1950's and 1960's (see Appendix A), and the design load provisions were presented in about two pages of text and tables. While outdated, it serves as a useful model for recognizing unique design load and resistance criteria in a simplified design approach for housing. In recent years, an attempt has been made to mimic and modernize this simplified approach for the design of homes while including some unique technical considerations based on current knowledge. The resulting document, *Structural Design Loads for One- and Two-Family Dwellings*, is based on a simplification of the ASCE 7 standard and provides comprehensive design load criteria in about 29 pages of text, figures, tables, and commentary (HUDa, 2001). Based in part on that document, an example of a simplified wind load provision based on the ASCE 7 standard is shown in Appendix B. The intent, as with the MPS load provisions, is to provide adequate loads for the design of one- and two-family dwellings with minimal engineering effort and/or complexity. This objective is particularly important since the high cost of design relative to the value of an individual home can detract from the benefits of using individually engineered house plans (St. Pierre et al., 2003; Crandell and Kochkin, 2003).

REVIEW AND ROADMAPPING TOPICS

In this section of the report, various load topics identified by the special project committee are presented. For each topic, a standard format is used to describe the topic, summarize existing knowledge, discuss implementation barriers and progress, and make recommendations toward future research and implementation. The order of presentation is not intended to reflect priority or level of significance.

Topic #1 Actual vs. Nominal Dead Loads

Description: Dead loads used for residential building design are commonly based on prescribed nominal dead loads for generic floor, roof and wall assemblies. Engineering design criteria for dead loads generally have required that an estimated actual dead load (including structural and non-structural component weights) be used in design; the dead load factor for strength design is based on this presumption. Discrepancies between actual estimated and prescribed nominal dead loads can result in unfavorable impacts on design, particularly for counteracting load combinations (e.g., uplift connection requirements may be under-designed if dead load is over-estimated). Furthermore, generic nominal dead loads for various residential building assemblies (e.g., ASCE 7 commentary data) appear outdated relative to modern residential construction practice and are in need of updating and possible expansion.

Existing Knowledge: Several references explain the historic treatment of and existing knowledge on the topic of dead loads for residential construction (HEW, 1931; NBS, 1948; FHA, 1958; HUDb, 2001; ASCE, 2002). Nominal dead loads are necessary for development of prescriptive construction provisions and, therefore, become a scope limit on the use of such provisions (ICC, 2000; AF&PA, 2001). A current listing of nominal or generic assembly dead loads is found in the commentary to the ASCE 7 standard and many date back to the 1945 edition of ASA A58.1. The provisions of ASCE 7 require that estimated actual dead loads be used for design as has been common to most design load standards (FHA, 1958; ASA, 1945; ANSI, 1982; 24 CFR 3280, 1999; NFPA 501, 2001; ASCE 7, 2002). Therefore, the use of nominal or generic building assembly dead loads should reflect this intent. The BMS 109 report provides the most comprehensive testing of weights of construction assemblies for residential construction (NBS, 1948). However, many of these assemblies are outdated while others still appear to be relevant. Nominal dead loads for a few modern conventional residential construction assemblies have been promulgated (AF&PA, 2001; HUDa, 2001). For determination of actual dead loads, material densities are available from a variety of sources, one of which is the commentary of ASCE 7.

Implementation Progress and Barriers: The treatment of dead loads as actual or nominal values in residential design practice varies to an unknown degree. Existing published information on nominal or assembly dead weights may be outdated and based on information dating back as far as 1945. However, there are few barriers toward improving knowledge on dead loads whether they are considered as nominal or estimated actual values in design.

Recommendations:

- Develop and submit a proposal that updates nominal assembly dead load information found in the commentary of ASCE 7.

- Assembly weights should address modern construction materials and typical assemblies including estimates of the mean and variation (or provide a min-max range) for each assembly type. Sources of variability to consider may include dimensional tolerances in manufacturing and installation and variance in material density (e.g., wood species and moisture content).
- Typical dead weights of modern fenestration and door assemblies also should be addressed (about 15 percent of wall surface area on average is used for windows and doors in typical residential construction).
- Dead load of typical roof assemblies should include compensation for the effect of roof slope (e.g., the TPI 1-02 standard for wood truss design includes such an approach for estimation of roof system dead loads).
- The possible use of different load factors on dead load when determined using actual weights of materials and when nominal values are used needs additional investigation to maintain consistent levels of reliability for the two approaches.
- Variation in dead load during the life of the building, due to unregulated but otherwise normal remodeling or maintenance activities, should also be considered.

Topic #2 Minimum Partition Wall Weight for Seismic Load Analysis

Description: Seismic design provisions have for some time required a minimum partition wall weight of 10 psf (based on floor area) to be included in the determination of the weight of the building for seismic lateral load analysis (ASCE, 2002). Based on a limited study of residential floor plans and use of a slightly conservative 6 psf partition wall dead load (based on wall area) (HUDb, 2001), it has been determined that typical partition wall dead loads on the basis of floor areas range from about 2 to 4 psf (upper stories are generally divided into smaller areas and have the higher range of partition loads). Using the nominal 10 psf floor area load to cover partition loads in seismic design of typical homes can result in conservative lateral loads for the design of shear walls and diaphragms (e.g., a 25 percent increase in base shear relative to the use of actual dead load values for partitions). If inadvertently applied to analysis of counteracting dead and wind uplift loads, use of a 10 psf minimum partition load is unconservative.

Existing Knowledge: Typical partition wall dead loads in one- and two-family dwellings have been briefly addressed in only one document considered by the special project committee (HUDb, 2001). In residential construction, it is also widely known that interior partition walls, when present, contribute significantly to lateral resistance of homes; however, engineering design criteria generally require that the mass of these walls, and not any resistance, be considered in seismic design (HUD, 2000; HUD, 2001). Therefore, it is important that interior partition wall dead loads and lateral resistance effects be treated together in terms of seismic design of homes.

Implementation Progress and Barriers: No known attempt has been made to adjust the minimum partition wall weight value of 10 psf (based on floor area) for analysis of seismic loads on dwellings and there are no significant technical barriers to consideration of such an adjustment. However, given that seismic load considerations are generally handled as a part of the NEHRP provisions (BSSC, 2000) and the ASCE 7 standard, a coordinated effort may be required.

Recommendations:

- A variety of typical residential floor plans should be studied to characterize interior partition wall loads as a function of floor area by story level. Information on partition wall loads is already found in one resource based on a limited survey of modern residential building plans (HUDb, 2001).
- Based on the above substantiation, formulate and submit a proposal to address an appropriate minimum partition wall load to use in the analysis of seismic lateral load on residential buildings. The proposal should be submitted to the appropriate committees as described in the previous section.
- Variation in partition wall dead load (as a function of floor area) during the life of a structure, due to normal remodeling or maintenance activities, should also be considered. However, for light-frame residential buildings, the inclusion of additional partition walls in a given story level tends to increase resistance in greater proportion than increases in inertial loads. Conversely, removing partition walls may decrease building mass, but can cause a disproportionately larger reduction in lateral resistance. Therefore, treatment of variation in partition wall dead load needs to be coordinated with practices used for calculating lateral resistance.

Topic #3 Use of Miscellaneous Roof Live Load

Description: The miscellaneous roof live load, L_r, is intended to account for maintenance and repair activities. Based on the experience of some members of the special project committee, this load is sometimes miss-applied in load combinations such as D+W+ (L_r or S). In addition, the combination of L_r with other live loads (floor and attic) may lead to unintended combinations of design loads.

Existing Knowledge: A nominal roof live load of 20 psf appears to date back to the 1955 edition of the ASA A58.1 standard and it was used in the 1958 edition of the Minimum Property Standards (MPS) where a minimum roof live load of 20 psf was required for a roof pitch of 3:12 or less and 15 psf for a roof pitch of greater than 3:12; roofs used as decks were required to use a 40 psf live load (FHA, 1958). See Appendix A.

In ASCE 7-02 the following description of roof live loads is given:

> **SECTION 4.1 DEFINITIONS**
>
> *...Live loads on a roof are those produced (1) during maintenance by workers, equipment, and materials, and (2) during the life of the structure by movable objects such as planters and by people.*

In Section 4.9 of ASCE 7-02, roof live loads are determined by equations that adjust from a nominal value of 20 psf (associated with a 200 ft^2 tributary area on a flat roof) to a value no less than 12 psf for sloped roofs and for tributary areas in excess of

200 ft^2. These adjustments are consistent with earlier editions of the standard and the area adjustments are similar to those used for floor live loads.

Implementation Progress and Barriers: Currently, there is no known research addressing the issue of an appropriate nominal live load for typical residential sloped roofs. The nominal roof live load value of 20 psf and adjustments for roof slope do not appear to have an empirical basis. Furthermore, the provisions of Chapter 2 of ASCE 7 provide load combinations where roof live load, L_r, is combined with other design loads such as wind. Guidance in the commentary implies that judgment is required in the use of load combinations, but does not explicitly identify combined loading situations where roof live load may be appropriately dismissed from consideration. This lack of specificity in ASCE 7 and building codes is considered a source of confusion among local authorities and design professionals. This issue is more clearly addressed in 24 CFR Section 3280.305 (and NFPA 501 Section 4.5.2.2) for manufactured housing as follows:

> *"... The roof live load or snow load shall not be considered as acting simultaneously with the wind load, and the roof live or snow load and floor live loads shall not be considered as resisting the overturning moment due to wind."*

Recommendations: Based on the above information, the special project committee recommends the following:

- A proposal should be prepared and submitted to the strength design task group of ASCE 7 responsible for the load combinations of Chapter 2. The proposal should clarify the intent in using roof live loads in combination with other design loads and provide some examples where it may be excluded (e.g., combination with wind on a sloped roof and combination with other live loads such as floor and attic live loads).
- A study should be conducted to validate the use of the nominal roof live load of 20 psf in relation to typical residential or light-frame sloped roof construction, including roof maintenance and repair activities and practices for staging roofing materials on the roof surface. The study should also consider any limitations to benefits in changing the nominal roof live load due to the effects of other loads on a roof such as wind, snow, and a concentrated live load. Furthermore, the study should examine ways to simplify the prescription of roof live loads for residential buildings, possibly by specifying an appropriate roof live load for two or more categories (i.e., flat roof deck vs. sloped roof construction) as has been done in the past (FHA, 1958; HUDa, 2001; HUDb, 2001). See Appendix A.

Topic #4 Attic Live Loads

Description: Similar to roof live load situation, attic live loads do not appear to be based on a load survey and statistics on sustained or transient attic storage loads. Building codes and standards have varied in judgments on and requirements for attic live (storage) loads. For example, in recent IBC building code deliberations, a long-standing provision for attic live loads was revised on the basis of new judgments without introducing new technical information to support the proposal or refute previous practice. In addition, the application of attic live loads in combination with other building live loads is not clearly addressed in current building codes and standards, resulting in varying interpretations and design impacts.

Existing Knowledge: No study of attic storage load characteristics for typical residential attic areas has been identified. In past design practice and building code provisions (e.g., BOCA, 1996), attic live loads have been treated in two ways for light-frame construction. In one method, a 10 psf live load is applied to the full extent of the attic "floor" area to account for personnel loading under conditions of limited access. As a second approach for attics with limited storage, a 20 psf live load is used only in areas that meet criteria in relation to horizontal and vertical clearances in the attic space. In the recent International Building Code (ICC, 2000), this practice has been changed to require a 20 psf attic live load to be applied in all cases irregardless of attic accessibility and space limitations. This change has caused significant design impacts for many residential and commercial buildings and particularly roof trusses.

In response to this issue, the TPI 1-02 wood truss design standard has included the following special provision for treatment of attic live loads in combination with other building (or roof) live loads:

> *6.2.1.1 Attic live loads, other than floor live loads, that are applied to the entire length of the bottom chord shall not be required to be applied concurrently with other live loads.*

The commentary of TPI 1-02 also explains that live loads in attic spaces applied to areas on the basis of accessibility and degree of obstruction to the use of the space (i.e., by attic floor to roof clearance and truss web members) is consistent with past successful practice and building code provisions.

The ASCE 7-02 standard is silent on this issue but includes attic live loads of 10 psf (without storage) and 20 psf (with storage). In the 1958 MPS, attic live loads were specified as 15 psf (dead + live load) for roof slopes of 3:12 or less without storage; 20 psf (live load) for attics with limited storage and roof slope of over 3:12; and 30 psf for attics served by a permanent or disappearing stair.

Implementation Progress and Barriers: As discussed above, the treatment of attic loads has relied primarily on judgment and accepted practice. This has created some instability in design practice, building code provisions, and design impacts. ASCE 7 does not address any specific practice in the application of attic live loads and simply provides two values. Given that most light-frame roofs are designed using metal plate connected wood trusses, the provisions of the TPI 1-02 standard and ASCE 7-02 standard should be coordinated in regard to the application of attic live loads.

Recommendation:

- A proposal to address the application of attic live loads in ASCE 7 should be prepared and coordinated with attic load provisions in the TPI 1-02 wood truss design standard. In the absence of new technical information, the proposal should be based on past successful industry design practice. The proposal should be submitted to the live load task committee of ASCE 7 for consideration. The proposal should address space and area limitations in attic spaces as well as intentions for use of attic live loads in combination with other extreme loads. This latter issue may also require coordination with or a separate proposal to the strength task committee of ASCE 7 in regard to use of attic loads in load combinations.

- A study of attic use characteristics should be conducted (i.e., load survey) to provide a qualitative, probabilistic treatment of attic storage loads. The study or survey should address use of attic spaces in homes constructed in accordance with modern building codes and practices while considering effects such as type of access, regulatory restrictions, amount of attic insulation, type of roof framing (e.g., truss or rafter), slope of roof (i.e., roof to attic floor clearances), and other factors that may govern the ability to store items in attic areas.

Topic #5 Multi-story Floor Live Load Coincidence

Description: The live load area reductions in ASCE 7 are limited to influence areas of greater than 400 ft^2. In design of residential construction, influence areas for individual repetitive framing elements (i.e., studs and joists) are narrow, long, and commonly less than the 400 ft^2 influence area limitation in ASCE 7, even for multi-story loading conditions. In such cases, live load area reductions of ASCE 7 do not apply even though a multi-story live load coincidence effect may be present for influence areas of less than 400 ft^2.

Existing Knowledge: The probabilistic treatment of floor live loads is addressed in several references (Peir and Cornell, 1973; Ellingwood and Culver, 1977; Corotis, et al., 1981; Harris, et al., 1981; Chalk and Corotis, 1980). More recently, multi-story live load coincidence effects have been subject to limited investigation (Rosowsky, 2001). The findings indicate that, depending on the level of correlation between live load processes on different stories, the accumulative multi-story nominal design live load may be reduced by a factor of 0.4 to 1.0. It is further recognized that differences in level of multi-story live load temporal correlation may be dependent on the nature of the residential occupancy. For typical single-family detached dwellings and townhouses of multiple stories, upper stories are often used as sleeping areas whereas the lower story level is used for day-time activity and gathering purposes. Based on this information, a live load reduction factor of 0.7 has been recommended and used in *Structural Design Loads for One- and Two-Family Dwellings* (HUDa, 2001). However, this multi-story live load coincidence factor is not applied simultaneously with live load area reductions as currently formulated in ASCE 7. Instead, use of the greater of the two reductions (as applicable) is recommended.

Implementation Progress and Barriers: Existing knowledge on multi-story live load coincidence effects is limited and lacks confirmation in terms of loading statistics and load process modeling that reflects actual use characteristics of different types of residential occupancies. Current recommendations for consideration of multi-story live load coincidence are based primarily on judgment in applying a limited study of the effects for one- and two-family residential construction (Rosowsky, 2001; HUDa, 2001). In addition, the consideration of multi-story live load coincidence should be carefully coordinated with application of live load area reductions in ASCE 7 to avoid potential miss-application resulting in unconservative live load determinations.

Recommendations:

- The effects of multi-story live load coincidence should be subject to further study to develop and confirm appropriate load process model for residential occupancies (mainly one- and two-family dwellings) and to better substantiate an appropriate live load adjustment.
- Based on existing knowledge, and additional study as deemed necessary, a proposal should be prepared to account for multi-story live load coincidence for consideration by the live load task committee of ASCE 7.

Topic #6 Floor Live Load Area Reduction for Residential Buildings

Description: Live load area reductions differ according to building occupancy. The equation currently used in ASCE 7 is based on data and analysis of various types of building uses including residential occupancies (Harris et al., 1981). Based on use of a residential live load of 40 psf, the analysis by Harris, et al. shows that area live load reductions should apply to tributary areas of 200 ft^2 or greater (see also HUD, 2001; FHA, 1958 – see Appendix A). In addition, the influence area factors (K_{LL} in Table 4-2 of ASCE 7) do not specifically address residential bearing wall conditions which should appear to use a K_{LL} factor of 2.

Existing Knowledge: See Topic #5.

Implementation Progress and Barriers: There appear to be no significant barriers or technical challenges in addressing this item with existing knowledge.

Recommendations: Based on the above information, the following recommendations are given:

- A proposal should be prepared for consideration by the live load task committee of ASCE 7 for the purpose of clarifying appropriate K_{LL} factors to use for design of light-frame building systems. It appears that light-frame bearing walls should be designed using a K_{LL} of 2 and that interior support columns (e.g., basement columns) should be designed for a K_{LL} of 4.

Topic #7 Wind Shielding

Description: The consideration of wind exposure and shielding is very important to efficient design. This observation is particularly true for residential buildings because they are typically located in a near-ground wind environment with numerous obstructions to wind. (HUD, 2001, HUD, 1993; NAHB-RC, 1996; NAHB-RC, 2002; HUD, 1999; Crandell & Kochkin, 2003; Crandell et al., 2000; Ho, 1992, HUD, 1998; St. Pierre et al., 2003). Currently, the effects of wind shielding are not permitted to be considered in the design of residential buildings following ASCE 7. The closely related topic of wind exposure is addressed as Topic #8.

Existing Knowledge: The power-law or log-law wind velocity profile is used in ASCE 7 and in most wind engineering standards to account for various building site exposures (surface roughness categories) and their effect on wind velocity with height (HUD, 1998). These boundary layer wind velocity profile relationships apply only to "straight winds" in the earth's boundary layer (lower atmosphere) and to wind flow above the interfacial layer. The interfacial layer is the layer nearest to ground that is occupied by objects that obstruct wind flow (e.g., trees, buildings, etc.). In effect, the boundary layer wind velocity profile is formed in response to the apparent roughness sensed below in the interfacial layer.

In truly open, grassy, flat terrain (exposure C) the interfacial layer is of a negligible height and the boundary layer wind velocity profile is displaced above the earth surface no more than the height of a stand of grass. However, in exposure B settings (suburban and /or wooded terrain) the boundary layer wind velocity profile is displaced considerably above the ground surface depending on the density, porosity, and height of obstructions to wind flow (HUD, 1998). Several analytical methods are available to estimate the displacement height of the boundary layer wind velocity profile based on configuration or type of obstructions to wind flow (HUD, 1998). This displacement of the boundary layer wind profile is a by-product of wind shielding effects.

Below the displacement height (within the interfacial layer), the wind is still flowing but has different characteristics than in the "free flowing" boundary layer above. First, the wind velocity profile below the displacement height (within the interfacial layer) follows an exponential form (concave downward). Wind shear or rate of change of velocity with height is fairly high in the transition between the boundary layer wind velocity profile above and the interfacial layer wind velocity profile below. Thus, wind speeds tend to drop off sharply as height above ground decreases into the interfacial layer. Second, while wind velocity is significantly decreased in the interfacial layer relative to wind speeds in the boundary layer above, the turbulence intensity of wind is dramatically increased. This is an important effect because it tends to increase wind surface pressure coefficients used to determine wind loads on buildings based on a reference velocity pressure (which for wind engineering purposes is based on a turbulence intensity associated with a standard open terrain boundary layer wind flow as modeled in a wind tunnel). Thus, increases in turbulence intensity can result in a smaller reduction in wind load than would be expected by considering reductions in wind velocity alone due to shielding effects in the interfacial layer.

The effects of shielding have been investigated in a wind monitoring project (Crandell et al., 2000). The study included a 187-foot tower and five near ground wind stations (9.8 feet) distributed more or less randomly in a typical built-up suburban setting with large open areas, streets, buildings, treed areas, and minor topographic features. Wind characteristics reported included gust and average wind speeds, velocity profiles, turbulence intensity, annual extremes, and spatial variability of extreme wind speeds. The study also made comparisons to wind velocity profiles used in the ASCE 7 standard using the observed annual extreme wind velocity profile shape (as opposed to a single "straight wind" event's profile shape). Among several findings considered to be important to the characterization of wind for engineering purposes, the findings relevant to shielding indicate that the average annual extreme wind speed in the interfacial layer can be over-stated by as much as 50 percent on average compared to use of a boundary layer wind profile as used in the ASCE 7 standard to predict wind speeds below a standard 32.8 feet height above ground. This over-estimation error is increased to 70 percent if the boundary layer wind profile is truncated at an elevation of 32.8 feet as is currently done in the ASCE 7 standard. As mentioned, the degree of conservative error in wind speed characterization does not necessarily correspond to an equivalent degree of error in wind load estimation for reasons of increased turbulence intensity in the interfacial layer and its effect on surface pressure coefficients.

The effect of shielding on building wind loads due to the combined effects of reduced wind speed and increased turbulence in built-up near ground wind environments has been quantified in an extensive wind tunnel study (Ho, 1992). The study found that shielding in a typical built-up (exposure B) wind environment results in a 25 percent reduction in average peak local area wind loads based on generation of several thousand aerodynamic data sets from several different building models in several different arrangements of built-up surroundings. Changes were recommended for the Canadian wind load provisions. An additional study on shielding effects was reported to be in progress at the University of Western Ontario wind tunnel facility at the time of this writing.

The Australian Wind Code for Housing (AS 4055-1992) includes simplified and conservative reduction factors of 0.85 and 0.95 to account for two categories of shielding in otherwise suburban wind exposures (exposure B). Greater reductions for shielding are possible in accordance with the more detailed methods of the Australian Wind Code (AS1170.2, 2002).

One recently reported study of shielding effects on manufactured homes was conducted at Texas Tech University wind tunnel (Gurley, Kiesling, and Letchford, 2003). The authors pose the following question for their research:

> *"The disregard of shielding in ASCE 7-02, yet its incorporation in the Australian Wind Load Code begs the question: How important is shielding on a structure and what parameters influence the magnitude of shielding?"*

In their study, reductions in wind uplift and drag (lateral force) due to variations in shielding effect (spacing of buildings and number of upwind rows of buildings) were investigated and compared to similar Australian wind tunnel experiments, the Australian wind code provisions for shielding, and ASCE 7 wind provisions (which disregard shielding). An open country upwind exposure was used throughout the study. Relative to loads experienced with an isolated model, the following observations were made based on a preliminary analysis of results:

1. Shielding effects were present with one or two rows of upwind buildings and additional rows provided little additional shielding effect [This finding demonstrates the short transition length needed to displaced the boundary layer wind velocity profile and realize shielding effects – a matter of a 100 feet or less

(full-scale) whereas it may require several thousand feet for the entire boundary layer wind profile to respond up to gradient height which is the basis of the exposure transition lengths required by ASCE 7].

2. Pronounced reductions in drag and uplift due to shielding was realized for building spacings of up to eight eave heights in the upwind direction. Shielding effects were negligible at spacings of about 16 eave heights or more. [This study did not include treed conditions that are common to residential developments].

3. Compared to ASCE 7-02, shielding accounted for a mean drag and uplift coefficient reduction of 80 percent for the closest building spacing of four times eave height. At a spacing of eight times the eave height, the shielding uplift load reduction was about 60 percent and the drag load reduction was about 25 percent.

4. Compared to a similar Australian wind tunnel study used as the basis for the Australian Wind Code shielding adjustments, the Australian results are conservative (result in higher loads) for building spacings of 8h or less (drag) and 16h or less (uplift). However, the trend in shielding effects is similar. For example, the Australian study shows a roughly 40 percent reduction in wind load for a building spacing of 4h (about one-half the amount of reduction found in the TTU wind tunnel study). This appears to indicate that the simplified shielding adjustment factors used in the Australian Wind Code for Housing (AS 4055-1992) are quite conservative.

In a similar study, surface pressures and load actions were studied on a two-story house model subjected to an up-wind suburban wind profile with surroundings varying between isolated model and developed (built-up) conditions (St. Pierre et al., 2003). Future studies are also planned. The following typical reductions in wind loads due to shielding afforded by surrounding development were reported:

1. Sidewall component and cladding pressure (middle of wall) was reduced 25 percent relative to the isolated building condition.

2. Roof corner component and cladding pressure was reduced by 60 percent.

3. Roof ridge component and cladding pressure was reduced by 40 percent.

4. Wood truss vertical uplift reaction (roof to wall connection force) was reduced by 40 percent.

In observation of these findings, the authors of the study make the following statement:

> *"It can be concluded from these general observations that surrounding houses significantly reduce wind loads and this should be taken into account in any design of these structures."*

The same observation would also apply to houses surrounded by trees or mixed trees and development. As an additional indication that wind shielding effects are an important consideration in determination of building loads, the ground-to-roof snow load conversion in ASCE 7 provides adjustment for wind "shading" (shielding) to account for increases in roof snow loads.

Implementation Progress and Barriers: The ASCE 7 standard currently disallows any consideration of shielding unless a site-specific wind tunnel study is conducted. While this recognizes that shielding effects can affect building wind loads, it creates an economic barrier to the consideration of shielding for residential and similar scales of building projects. A generalized treatment of shielding effects for low-rise building populations in built-up, suburban, or wooded terrain conditions has yet to be implemented in the United States, though some consideration has been recently given to this topic by the ASCE 7 wind task committee with action deferred. The Australian wind code for housing has addressed this important issue by creating two categories of wind shielding and establishing conservative wind load reduction factors for use in suburban or wooded exposure conditions. The Australian wind code also provides a more detailed method to account for shielding effects in combination with a more detailed site-specific exposure analysis. Existing research and international wind loading standards should provide an adequate basis for generalization of shielding effects in ASCE 7.

Recommendation:

- A task committee within ASCE SEI TAD Wind Effects should be formed to evaluate available technical resources on wind shielding effects and to develop appropriate guidelines for the consideration of generalized shielding effects for low-rise buildings in built-up or wooded exposure B environments for consideration by ASCE 7. Exposure transition lengths in shielded conditions should also be considered. For example, the transition length to fully displace the boundary layer wind profile may be more important than the transition length required to develop the full boundary layer wind profile up to a gradient height of more than 1000 feet above ground. The Australian wind code provisions for shielding should be considered in developing a plausible approach.

Topic #8 Characterization of Suburban and Wooded Wind Exposure

Description: Appropriately defining the wind exposure or roughness category of a building site may be as important to risk-consistent building design as the wind climate itself. For residential buildings and low-rise construction in general, the exposure B terrain condition suburban/wooded surroundings) is commonly applicable. However, the current derivation of velocity pressure coefficients in ASCE 7 for this terrain condition tends toward a conservative wind load determination. In part, this tendency is due to the broad range of terrain conditions addressed by the exposure B classification.

Existing Knowledge: The effect of surface roughness on the boundary layer wind profile (for "straight" or large scale cyclonic winds) is well established. The difficulty however, lies in selection of an appropriate categorization of surface roughness for design purposes. One study has indicated that designers will tend to select a conservative exposure category for a given site (Ellingwood and Tekie, 1998). In addition, the K_z values are conservatively defined for exposure B in ASCE 7 because they are based on a lower-bound surface roughness (z_o = 5.9 inches) for this broad category of building exposure. Compared to a more typical exposure B condition

(z_0 = 0.98 feet), the ASCE 7 design wind load is conservative by about 67 percent (based on ratio K_z values for the two z_0 values, 0.7/0.42 = 1.67) which also agrees with wind tunnel and wind monitoring data (Ho, 1992; Crandell, et al., 2000). The degree of conservatism is even greater for common exposure B conditions where surface roughness may include dense development or development mixed with trees (z_0 = 2.29 feet).

According to several residential building performance surveys comprising a random selection of more than 1000 homes in several southern and eastern regions of the United States, as many as 95 percent of sampled homes were located in a suburban (exposure B) setting as described by ASCE 7 and, in about two-thirds of these cases, the homes were embedded within or surrounded by stands of trees (HUD, 1993; NAHB Research Center, 1996; NAHB Research Center, 2002; HUD, 2001). Therefore, the boundary layer wind profile for the majority of home sites is more reflective of z_0 values of 0.98 feet or greater. In these conditions, shielding is an additional factor that should be considered for buildings of comparable or lesser height than the surrounding roughness (see Topic #7).

Implementation Barriers and Progress: The ASCE 7 wind task committee has begun to consider modifications to the K_z values for exposure B in the current update cycle for ASCE 7-02, but has chosen to defer action. In addition, a truncated exposure B profile has been maintained. The reasons for these actions include concerns that designers may incorrectly specify exposure and that ongoing research may provide additional insights into an appropriate level of adjustment for main wind force vs. components and cladding wind loads. However, the recent studies discussed above provide mounting evidence that changes should be considered. The Australian wind code addresses this concern by including two exposure conditions for suburban settings by defining two development densities, one of which also corresponds to wooded terrain. Similarly, the snow load provisions of ASCE 7 include several wind and exposure and sheltering conditions for the explicit purpose of minimizing the magnitude of user "error" that may occur as a result of differing opinions in classifying a site (see ASCE 7 commentary C7.3.1).

Recommendation:

- The Wind Effects Committee in SEI TAD should continue to consider available and new technical resources on wind exposure for suburban settings, including consideration of typical residential buildings and exposures, to develop improved K_z factors for exposure B that are representative of wind loads experienced by the population of buildings in these conditions. Also, the need to truncate K_z factors at an elevation of 30 feet should be re-evaluated. Alternatively or additionally, an added exposure category should be considered to establish a more representative suburban exposure and minimize the degree of under- or over-estimation error that may occur in current practice with use of the ASCE 7 exposure B wind loads. The Australian wind code provisions may serve as a model for this purpose. Any differing effect of exposure condition (wind speed and turbulence) on main wind force and local area (components and cladding) wind loads should also be considered based on recent research. As an additional consideration, methods to calculate K_z based on an assessment of surrounding surface roughness should be moved from the commentary of ASCE 7 and introduced to the text of the standard.

Topic #9 Wind-borne Debris

Description: A wind-borne debris region definition was added to recent building codes and the 1998 edition of the ASCE 7 standard to regulate consideration of wind-debris protection in the design of buildings. This definition is associated with an ASTM standard referenced by ASCE 7 that provides debris impact resistance criteria within the declared wind-borne debris region (ASTM, 2002). The basis for the wind-borne debris region definition in ASCE 7 and ASTM E 1996 is founded on judgment and overlooks the effects of some fundamentally important variables such as wind exposure. In addition, the principles of performance-based engineering, whereby target reliability is established in association with a particular design limit state and is uniformly applied across all applicable hazard levels, has not been implemented. Instead, a rather unique and unprecedented approach has been used by the ASTM committee to justify selected impact criteria on the basis of "improving reliability by 50 percent" rather than establishing and uniformly implementing a target level of reliability based on a rational calibration to accepted risk as has been done for other building loads in ASCE 7 through a landmark study conducted by NIST (NBS, 1980).

In summary, the basis of the wind-borne debris region definition in ASCE 7 as well as the basis of impact criteria in the ASTM standard is technically inconsistent with the reliability basis of loading criteria used in the development of loads and load combinations in ASCE 7. Furthermore, key variables affecting debris hazard are neglected (e.g., wind exposure). Therefore, it is questionable that the current wind debris provisions in ASCE 7 and ASTM E1996 are technically robust.

Existing Knowledge: Much of the existing knowledge on the effects of wind-borne debris is based on anecdotal wind damage surveys (e.g., Beason et al., 1983; Kareem, 1986; etc.) and are often related to specific conditions affecting debris hazard (e.g., rooftop gravel) and building vulnerability (e.g., glass building facades). Therefore, they are of limited application and must be extrapolated to make more generalized inferences on debris risks in a population of buildings under varying hazard and vulnerability conditions.

More recently, wind-borne debris damage has been statistically quantified in experimentally controlled building performance surveys following major wind events (HUD, 1993; NAHB Research Center, 1996; NAHB Research Center, 2002). The results of these studies are useful for quantification of debris hazard and building vulnerability and have been used for the purpose of calibrating and verifying probabilistic building damage risk models (HUD, 1999; ARA, 2003). For example, in Hurricane Andrew about 90 percent of residential buildings experienced one or more broken windows in a wind field of about 160 mph over the region from which over 400 building samples were randomly selected. Many of the buildings had no window protection or make-shift protections in place at the time the hurricane struck. The design wind speed for this region of South Florida is 145 mph. In contrast, about

2 percent of the buildings in a sample of 200 homes in Hurricane Opal experienced one or more broken windows and were located on the barrier islands. Hurricane Opal's wind speeds were a maximum of 125 mph gust at one location on landfall and the wind speed over the study region ranged from 100 to 110 mph (standard exposure C, 33 feet elevation). The lack of window and building damage inland was associated with the treed and suburban exposure condition (see Topics #7 and #8).

In addition, wind-borne debris impact data was collected and evaluated based on a representative sample of 200 buildings experiencing a tornado ranging in F-scale from F2 to F4 (F3 on average along the distance of the tornado path surveyed) (NAHB Research Center, 2002). In the vortex region, 81 percent of the sampled homes experienced one or more glazed opening penetrations; within 150 feet of the vortex (i.e., in-flow winds), this number dropped to 52 percent; farther out from the vortex the number dropped again to about 13 percent. For building wall penetrations, the comparative frequencies of one or more penetrations were 43 percent, 22 percent, and 0 percent, respectively. From this data as well as data on impact fragility of building products (HUD, 2002; Clemson, 2000) and normalization of wall and opening areas, it was found that debris impacts to glazed openings were rarely more than 10 lb-s (momentum), but typically greater than 2 lb-s in an average F3 tornado. Thus, a 10 lb-s protective device on glazed openings would prevent about 92 percent of the homes from having even a single glazed opening failure due to penetration from flying debris in an average F3 tornado. Furthermore, the level of structural damage experienced on average was correlated to wind speed of approximately 165 mph based on similarity of damage statistics documented in the Hurricane Andrew study. For this type of wind condition, these impact levels are significantly less than those currently required by the provisions of ASCE 7 and ASTM E 1996.

Hurricane damage risk models have also been developed and verified using actual event data for the purpose of evaluating cost-benefits of various wind-resistant building features, including various wind-borne debris protection strategies (Twisdale and Vickery, 2003; Young and Twisdale, 2003). Based on selected benefits and costs, the model shows a general economic benefit for wind-borne debris protection strategies (e.g., 33 percent reduction in average annual loss or about a 15 percent reduction in relative loss in combination with other typical wind-resistant design features in new construction). The individual relative importance of various wind-resistant building features in reducing modeled building loss relativities is approximately as follows:

1. Use of hurricane clips instead of toe-nails (gives about a 60 percent reduction in relative loss)
2. Use of wind resistant shingles instead of standard shingles (gives about 30 percent reduction in relative loss)
3. Use of 8d nail instead of 6d nail at standard spacings in roof sheathing (gives about a 20 to 40 percent reduction in relative loss)
4. Use of foundation anchors in lieu of nothing (gives about a 30 percent reduction in relative loss)

5. Use of hip roof instead of gable roof (gives about a 25 percent reduction in relative loss)
6. Use of one story instead of two story construction (gives about a 20 to 25 percent reduction in relative loss)
7. Use of basic debris protection instead of none (gives about a 15 to 20 percent reduction in relative loss)
8. Use of secondary roof water barrier only provides about 3 percent reduction in relative loss if wind resistant shingles are used (15 percent if they are not used).

However, the economic benefits of these loss reductions are generally accrued to insurers rather than building owners since many states lack a risk-consistent structure for regulation of insurance rates and rate incentives for recognition of building features that lower economic loss vulnerability, such as wind-borne debris protection. Instead, policies in lower hazard areas are often used to subsidize higher loss ratios in higher hazard areas. The actual dollar magnitude of annual average loss is also very dependent on the value of the construction, site exposure, wind speed climate, building configuration (e.g., roof shape, building height, amount of fenestration, etc.), and value of the building and its contents. Therefore, the benefits of debris protection are not consistent for all types of buildings (or occupants) that may be required to have debris protection. As shown above, the benefits of wind-borne debris protection ranks about seventh among eight wind-resistant construction features and do not appear to be more significant than the choice of roof style or number of stories, which currently are not regulated.

Localities have been politically motivated to reject or modify current wind-borne debris criteria found in model building codes, ASCE 7, and ASTM standards for a variety of reasons such as:

- the benefits are variable and primarily economic in nature,
- the economic concern is considered to be an "insurance rate issue,"
- local experience and judgment differs significantly from that represented on national standards and building code committees,
- debris regions (particularly in areas with basic wind speed of less than 120 mph) have not been sufficiently justified relative to local experience and on a credible basis of maintaining an acceptable and consistent target reliability, and
- the variable cost and limited availability of wind-borne debris protection devices meeting ASTM E1996 specifications and other criteria (e.g., energy code and aesthetics).

Recent examples of rejection or significant modification of debris protection criteria include the North Carolina coast and the Florida gulf coast (panhandle). Conversely, other areas have adopted the provisions without modification. In general, the adoption process is in a state of uncertainty and change. In addition, building code provisions maintain alternative protections that have been considered as acceptable practice by experience (e.g., plywood window coverings) and that provide a significant level of debris protection (ARA, 2002), but are not necessarily consistent with debris impact requirements in ASTM E 1996 as referenced by ASCE 7 and current model building codes.

Recommendations:

- Research toward development of target reliability for establishing limits to a defined wind borne debris region and impact criteria to be applied within that region (including effects of key variables such as exposure) is needed. The research should consider calibration to a target reliability representing a range of accepted practice. The calibration exercise should be conducted using a clearly defined damage limit state criteria in relation to economic and building design impacts (e.g., internal pressurization of a breached building). If necessary, an average result of the reliability calibration conditions should be selected to minimize departures from the selected calibration points as a whole. Calibration points representing currently accepted levels of debris risk include South Florida's 140+ mph wind climate with locally required debris protection, 130 mph wind climate using plywood covering as debris protection, and a 110 mph wind climate with standard glazing and located at or near the coast without wind debris protection. Following such a calibration methodology, wind debris criteria and the wind borne debris region can then be defined following a consistent and credible basis for performance (e.g., target reliability) and in consideration of fundamental hazard variables such as wind exposure.
- Based on the above research, a proposal should be prepared for consideration by the ASCE 7 wind task committee and coordinated with the ASCE 7 strength (load combination) task committee and the ASTM wind borne debris task committee.

Topic #10 Air-permeable Cladding Wind Loads

Description: Wind loads on air-permeable claddings are typically less than loads calculated across the entire building wall or roof system in accordance with ASCE 7 provisions. While ASCE 7 recognizes that air-permeable cladding load reductions are valid, guidance is lacking on methods of testing to determine wind load adjustments for air-permeability cladding products, and a generalized calculation method for air-permeable cladding load effects based on the degree of porosity or venting of the cladding system does not exist.

Existing Knowledge: Certain material standards (e.g., ASTM standard for vinyl siding) give recognition of a 50% cladding load reduction due to air-permeability, although the technical justification of this level of reduction is not known. Air-permeable cladding load reductions for various other cladding types (e.g. wood lap siding, brick veneer, etc.) is lacking.

The Forest Products Laboratory has conducted full-scale wind pressure measurements on one type of air-permeable cladding (hardboard siding) installed on a test building (TenWolde et al., 1998). The cladding pressures experienced was approximately two-thirds of the total pressure differential across the wall system.

Research sponsored by the National Roofing Contractors Association has developed a method for determining roof shingle loads, also relying on full-scale pressure monitoring on a test building (Peterka, et al, 1997). In this full-scale study, air-permeable cladding wind load reductions were as high as 75 percent. Studies of various cladding systems with varying degrees venting and resulting pressure-equalization (known as pressure-equalized or pressure-moderated rain-screen cladding) have also shown cladding load reductions similar to that reported above (CMHC, 2000, 2001, 1998, 1997, and 1996). A simplified computer model has also been developed to predict pressure equalization effects for air-permeable or "rain-screen" cladding systems (CMHCa, 1996). Methods to accurately assess air-permeability wind load reductions are not standardized and often rely on expensive whole building pressure measurements under actual wind loads.

Implementation Progress & Barriers: Current building codes and standards do not give adequate guidance on air-permeable cladding wind loads. In some cases, air-permeable claddings are specifically not permitted to be considered as air-permeable and must be designed for significantly higher loads than actual. This problem affects accuracy and economy of design of claddings as well as attachment methods. For example, 24 CFR Part 3280 is interpreted by HUD to require that vinyl siding cladding loads cannot be reduced for air-permeability. However, the ASTM D3679-96a standard for vinyl siding specifically includes a reduction for air-permeability. Attempts to include available air-permeability data (in terms of cladding load reductions factors) for some exterior finish materials in the ASCE 7 wind provisions have been rejected without clarifying a method by which air-permeability load reductions may be considered in the future.

Recommendations:

- A literature search on the topic of air-permeable cladding wind loads is needed. Methods to calculate wind loads on air-permeable cladding systems or porous wall systems should also be sought in the literature.

- Basic research is needed to develop a suitable general test methodology for determining air-permeable cladding load effects, develop a data set of load characteristics for a variety of air-permeable cladding products with differing degrees of porosity, and evaluate the data to formulate a generalized method for calculating air-permeability load adjustments based on fundamental cladding properties (e.g., degree of porosity or ventilation).

- A proposal should be submitted to the ASCE 7 wind task committee to clarify intentions for the proper characterization of air-permeable cladding loads and in what form air-permeable cladding load adjustments may be recognized for use by the design community and cladding product manufacturers. Any position taken by ASCE 7 wind task committee should be communicated and coordinated with appropriate standards development activities within ASTM.

Topic #11 Unbalanced and Drift Snow Loads

Description: For residential buildings, designers have often used a ground snow load value as a conservative uniform roof snow load (without conversion to roof snow load). This practice has been successful for the design of typical sloped roof systems on homes and similar buildings. Unbalanced and drift snow load is generally not explicitly considered, except on occasion at features such as down-steps in a roof system. While there have been few problems with this simple approach to residential building design, there is room for improved characterization of unbalanced and drift snow loads on residential-scale roofs.

Existing Knowledge: In the research forming the basis for ground-to-roof snow load conversion factor used in ASCE 7 (e.g. $p_f = 0.7p_g$), drift or unbalanced roof snow loads were not studied and statistics on the location, shape, and magnitude of drift loads are generally not available for probabilistic evaluation (O'Rourke, Koch, and Redfield, CRREL Report 83-1, 1983). In substitution, case studies of primarily commercial building roofs (i.e., long spans and low slopes) and analysis of the same have been used to define drift and unbalanced snow loads in ASCE 7. Thus, these loads are not necessarily quantified on a probabilistic bases (e.g., what is a 50-year return period drift load for a given roof configuration?) Conversely, the statistical knowledge of ground snow loads and variables affecting ground-to-roof snow load conversion for the determination of uniform snow load on roofs is relatively well established and provides a probabilistic basis for the uniform snow load parameters found in ASCE 7 and the load factor applied to uniform roof snow loads in load combinations (Ellingwood and Redfield, 1983; O'Rourke, Koch, and Redfield, 1983)

Implementation Progress and Barriers: Unbalanced and drift snow loads in ASCE 7 are essentially deterministic in nature and are assumed to occur at levels prescribed in perfect correlation with the occurrence of extreme uniform ground and roof snow loads. The primary obstacle in treating unbalanced and drift snow loads in a probabilistic manner is in the difficulty of obtaining statistically robust field data characterizing unbalanced and drift snow occurrences on roofs of a representative sample of buildings (similar to that done for the study of uniform roof snow loads by CRREL). Modeling of snow transport processes and the stochastic nature of variables affecting drift and unbalanced snow loading (e.g., wind speed, wind direction, building aerodynamics, and snow data) may be used to generate extreme value statistics for typical drift and unbalanced snow load formations. In addition, special features of small residential roofs (such as flow attachment and saltation distance for snow particles) could be addressed through aerodynamic studies (e.g., wind tunnel testing). The modeling work could be verified by a limited collection of drift and unbalanced snow load case histories for typical residential (and commercial) buildings. From the data, the magnitude and spatial extent of drift and unbalanced roof snow loads could be characterized and also represented as simplified equivalent uniform roof snow loads.

Recommendation:

- Conduct a research program (as described above) to characterize drift and unbalanced snow loading provisions in ASCE 7 on a probabilistic basis. The impact of such an effort on current design practice is unknown since the probabilities associated with current unbalanced and drift snow load criteria in ASCE 7 are unknown. However, it can be expected to improve risk-consistency of designing for unbalanced and drift snow loads.

Topic #12 Lateral Soil Load on Residential Foundations

Description: Lateral soil loads in ASCE 7-02 and similar building regulations are based on soil conditions (e.g., degree of compaction) that are not representative of backfilling practice for residential foundation walls. At the same time, the traditional use of a 30 pcf soil design value for residential foundation walls has been discontinued from code recognition for unknown reasons. As a result, soil lateral design loads have been significantly increased without clear justification relative to past accepted practice for the design of residential foundation walls.

Existing Knowledge: In past design practice for residential and other similar shallow foundation walls, a 30 pcf (equivalent fluid density) soil lateral design load has been used successfully for backfill and soil placement conditions relevant to typical residential construction practice. Provisions in ASCE require use of a 60 to 100 pcf equivalent fluid density value for soils typically found on residential sites. The past successful design experience in using a 30 pcf value is largely due to differences in how soil is placed when backfilling foundation walls on residential buildings as opposed to other applications where compaction of soils to near optimum density is necessary and specified (e.g., retaining wall supporting a highway surface). Footnote A of Table 5-1 of ASCE 7-02 clearly states that the soil lateral loads are based on "moist soil at their optimum densities." The footnote further states that "actual field conditions shall govern." One problem has been that the "actual field conditions" of residential foundation wall backfill have not been known in terms of actual lateral soil loads.

A recent study has monitored soil pressures on a residential foundation wall for two soil types (clay and sand) and for several variations in typical soil placement practice of loose, tamped, and frozen conditions (University of Alberta, 1992). The findings from a roughly 1.5-year monitoring project justify the following earth pressure coefficients based on observed lateral earth pressures on a typical 8-foot tall residential foundation wall:

Clay, CL (frozen/thawed)	$K_a = 0.21$
Clay, CL (tamped)	$K_a = 0.19$
Clay, CL (loose)	$K_a = 0.34$
Sand (loose)	$K_a = 0.26$

Thus, for a typical clay soil (CL by Unified Soil Classification), an equivalent fluid density of about 36 pcf (Ka x w = 0.34 x 105 pcf = 35.7 pcf) was the largest observed value in the study. The other three values were in the range of 20 to 25 pcf equivalent fluid density. Thus, the past use of a 30 pcf equivalent fluid density value for determination of soil lateral loads on residential foundation walls does not appear to be significantly in error, particularly considering that these loads and material resistances are factored in the design of foundation walls. For comparison, the ASCE 7 standard (Table 5-1) requires that a value of 100 pcf be used for clay (CL) soil backfill. The International Building Code (IBC) requires a 60 pcf value for this condition. Both of these values are very conservative relative to the available data on actual conditions of residential foundation soil pressures and relative to past accepted design practice of using a 30 pcf value.

Implementation Progress and Barriers: Past accepted practice of using a 30 pcf equivalent fluid density for determination of lateral soil pressures on shallow (less than 10 feet deep) residential foundation walls has essentially disappeared from current building codes. In replacement, the ASCE 7 standard's soil values are being increasingly used, although some significant differences in current building code requirements persist. While the 30 pcf value may have been rightly considered to be unsuited for general application (all buildings and all soils), it has been shown in recent research on residential foundations to be in reasonable agreement with observed soil pressure for sand and clay soil types. Currently, there are no suitable guidelines available for determining lateral soil pressures on residential or similar foundation walls.

Recommendation:

- A proposal should be submitted to the ASCE 7 task committee responsible for soil loads to consider re-instating an improved, simplified residential foundation design criteria similar to that traditionally used for the design of residential foundation walls under typical residential backfill placement conditions.

Topic #13 Simplification of Design Loads

Description: The theme of simplification of design load provisions for a narrow scope of buildings, such as one- and two-family dwellings and manufactured homes, was present in the committee discussions on several different load topics. It is a concern that is voiced by many design practitioners. In the past, building codes have contained very simple procedures that, given the uncertainties in any given design application, have resulted in efficient and functional designs with minimal design effort. One simplified approach in an early (1958) residential building code is included in Appendix A. The approach in Appendix A was continued until such a time that the HUD *Minimum Property Standards* were terminated by act of Congress in the late 1960s and a new prescriptive residential building code, based in part on the MPS, was developed by CABO. Thus, a simplified set of building load provisions for residential construction has been absent from model residential building codes in the United States for about 30 years.

Today, more homes than ever before require engineering design due to recent changes in scope limitations to conventional construction requirements, changes in housing styles, and changes in regulations for housing construction in earthquake and hurricane-prone regions of the United States. Therefore, a modern simplified method for determination of building loads for homes is probably in greater need today than in the past. Such a method can effectively eliminate about 80 percent of the content of the ASCE 7 by focusing only on coordinated and simplified provisions that provide an adequate means of a complete design for typical residential buildings (HUDa, 2001).

Existing Knowledge: Simplified design load methods are dispersed throughout the ASCE 7 standard and also are found in some newer and older building codes. However, these methods are not easily identified and they are not necessarily coordinated in terms of scope of application. For example, scope limits toward application of the simplified wind load method in ASCE are not coordinated with scope limits proposed for a new simplified method for seismic loads. Furthermore, simplified methods that are intended for broad applications tend to mimic detailed loading conditions (simply tabulating results of the detailed analysis method) without considering the benefits of rounding-up and eliminating various loading conditions that may be of little consequence to a particular application. An attempt has been made, under sponsorship of HUD, to produce a coordinated and simplified building load guide for application to homes and possibly other similar buildings (HUDa, 2001). Based on provisions found in *Structural Design Loads for One- and Two-Family Dwellings* (HUDa, 2001), a simplified wind load procedure is included in Appendix B of this document as an example. Simplified provisions for other loads, with the possible exception of seismic loads, would require even less specification when narrowly scoped for light-frame construction applications.

Recommendation:

- A simplified residential building load method should be considered as a separate guide, included as a companion document to, or integrated with ASCE 7. Alternatively, such a method could be formatted and submitted as a separate chapter or appendix to the current model residential building code. The information in Appendix A, Appendix B, and in *Structural Design Loads for One- and Two-Family Dwellings* (HUDa, 2001) should serve as model for such an effort.

The HUD simplified load guide is available as a PDF download at www.huduser.org by searching for the title "Structural Design Loads for One- and Two-Family Dwellings."

Topic #14 Performance Objectives and Target Reliability for Housing Design

Description: Residential buildings are a unique class of structures with unique and varied structural performance requirements. Several efforts in the past have attempted to quantify the performance attributes of housing and to develop performance objectives and criteria based on those attributes. One example in the not too distant past is HUD's Operation Breakthrough. More recently, guidelines for specifying housing performance have been developed by ASTM committee E06. Perhaps of greater importance, the housing market in the United States has expressed a variety of levels of performance expectations in meeting housing needs over time. For example, differences in performance expectations and market need for housing was recognized in the early 1900s as a part of Sears and Roebuck Co.'s successful offering of "standard-built" and "honor-built" kit homes (HUD, 2001). Today, manufactured housing follows a unique performance-based regulation known as the HUD-code which has existed for approximately 30 years. The HUD-code does not include prescriptive construction requirements similar to those used for one- and two-family dwelling construction in current model building codes.

Differences in end-use and associated performance expectations of the end-users are not consistently reflected in current design or construction criteria for housing. In addition, target reliabilities for minimum structural design requirements have been conservatively calibrated to past design practice for primarily steel and concrete (commercial) buildings (Ellingwood, et al, 1982; Galambos, et al, 1982; NBS, 1980). Only recently has the structural reliability of housing been the subject of specialized study for the purpose of eventually benchmarking a range of acceptable structural reliability represented by housing characteristics over the time period of the past century (Rosowsky, 2001). A framework for establishing structural safety and serviceability performance guidelines for one- and two-family dwellings also has been investigated (NIST, 1999). However, a minimum target reliability especially intended for housing design that is consistent with past acceptable practice (as was done when calibrating modern probabilistic design loads to past design reliability) has not been thoroughly investigated or proposed.

Existing Knowledge: See above.

Implementation Progress and Barriers: There are complications to be considered in how a separate performance basis for design of homes might be implemented. It would not be appropriate to introduce a separate set of load combinations with different factored loads. Should different target reliabilities for housing be justified as might be expected, they would probably be similar to existing target reliabilities and any difference may be handled within the existing format of importance factors. Thus, different types of housing construction (with different levels of implied importance) could have separately defined importance factors. In addition, enhanced as well as minimum levels of performance could be codified for use. Furthermore, it may be practical to indicate a "recommended" level of performance with a plus or minus range indicative of uncertainties in calibration of target reliabilities to past successful practice such that designers have some latitude in exercising reasonable judgment in the design process.

Recommendations: There are many uncertainties and difficulties in how to approach the topic of performance objectives for housing structural design. Investigation of different target reliabilities for housing structural design should be continued with the purpose of quantifying a range of "acceptable" performance based on past practice as it may have varied for different housing needs addressed in the marketplace over time. Based on an identified range of acceptable performance, a format for implementing this information in the context of "performance-based design" also needs consideration. One approach, building on the existing building code format for structural design, should explore the use of "performance factors" in lieu of current importance factors which are based solely on building use category without consideration of different performance levels that may be acceptable (or desired) within any given building type or use category.

CONCLUSIONS AND RECOMMENDATIONS

The special project committee has identified and reviewed a total of 14 topics related to residential building loads over the course of two years, two committee meetings, and three drafts of this document. The primary goals have been to compile existing knowledge and to identify needs in regard to improving the accuracy and application of structural loads for the purpose of efficient residential building design. The scope of the work was focused mainly on issues and knowledge relevant to one- and two-family dwellings as well as manufactured housing. However, there are many instances were the findings are more broadly applicable. In addition, this work should not necessarily be considered as exhaustive or complete, although every effort has been made to include available and relevant information.

The key recommendations in regard to the 14 topics addressed in this document are listed below. For a more detailed description of the topics and additional recommendations, the reader is referred to the appropriate sections of this document.

Topic #1: Actual vs. Nominal Dead Loads – Need to update and possibly expand generic (nominal) assembly dead loads in the ASCE 7 commentary to better reflect actual estimated assembly dead loads of modern residential construction materials, components, and assemblies.

Topic #2: Minimum Partition Wall Weight for Seismic Load Analysis – Use of a 10 psf minimum partition wall dead weight for seismic base shear analysis should be re-evaluated in view of partition wall dead weights and structural implications relevant to light-frame residential construction.

Topic #3: Use of Miscellaneous Roof Live Load – Use of the miscellaneous roof live load, L_r, in ASCE 7 Chapter 4 should be clarified in regard to its use with load combinations in Chapter 2; specifically, conditions where L_r may be appropriately dismissed from consideration should be addressed (e.g., combinations involving L_r and design wind load), particularly for typical residential sloped roofs.

Topic #4: Attic Live Loads – An attic live load study is needed to provide lacking data on attic usage and loading characteristics; in the interim, ASCE 7 should incorporate guidance in regard to application of specified attic loadings in regard to area loads on attic members as well as combined loads on other portions of the building.

Topic #5: Multi-story Floor Live Load Coincidence – The inclusion of a live load adjustment for multi-story conditions relevant to single-occupancy residential buildings should be considered in ASCE 7. Supplemental research should be conducted as necessary to address technical concerns and develop an acceptable solution.

Topic #6: Floor Live Load Area Reduction for Residential Buildings – The use of influence area factors (K_{LL}) in ASCE 7 should be clarified in terms of their application to light-frame, residential building assemblies and structural components.

Topic #7: Wind Shielding – The ASCE 7 wind task committee should continue to consider new and existing technical information regarding wind shielding effects on residential and similar low-rise construction. As considered feasible, an acceptable and defensible methodology should be developed to account for shielding effects on wind loads (MWFRS vs. components and cladding) and exposure transition lengths.

Topic #8: Characterization of Suburban and Wooded Wind Exposure – The ASCE 7 wind task committee should continue efforts to improve wind exposure classification and to consider relevant sources of data. In particular, several recommendations in regard to characterizing wind loads in the broadly defined exposure B category should be considered (see previous section of report) as they are very relevant to efficient and accurate residential and low-rise building design.

Topic #9: Wind-borne Debris – Research should be conducted in coordination with ASCE 7 and ASTM committees to define an acceptable performance objective (target reliability) for wind-borne debris protection and to use that target as an objective and risk-consistent basis for determining extents of wind-borne debris hazard region as well as required impact resistance levels in those regions.

Topic #10: Air-permeable Cladding Wind Loads – Methods by which air-permeable cladding wind loading effects may be evaluated and recognized for use in design need to be clarified and or developed in coordination with ASCE 7 and appropriate ASTM committee(s).

Topic #11: Unbalanced and Drift Snow Loads – Research should be conducted to establish a probabilistic basis for drift and unbalanced snow load provisions of ASCE 7; field data on drift and unbalanced snow load magnitudes and patterns from representative samples of residential roofs and other buildings should be collected to underpin probabilistic modeling and resulting design loads.

Topic #12: Lateral Soil Load on Residential Foundations – Residential foundation construction practices are not represented in the basis of the soil loads in Table 5.1 of ASCE 7. For residential basement foundation walls, a design soil lateral load(s) should be developed in consideration of recent research and past accepted practice.

Topic #13: Simplification of Design Loads – Simplification of building load provisions is considered to be important to efficient and consistent residential building design. Such a method has not existed in U.S. model codes for residential construction since the late 1960s, yet the present need may be greater now than before. Implementation of simplified load provisions for residential construction should be considered and coordinated with ASCE 7 and current model building codes in the United States.

Topic #14: Performance Objectives and Target Reliability for Housing Design – This topic is one that was only casually discussed by the committee. However, it may be considered important to overall design and future building load framework for residential construction. Additional research in regard to actual structural performance and benchmarking of acceptable target reliabilities for residential construction is needed.

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APPENDIX A - Structural Design Data As Excerpted From Appendix A of *Minimum Property Standards* (FHA, 1958)

Appendix A

Structural Design Data

CONTENTS

A GENERAL

A–1 The purpose of this appendix is to provide basic structural design data for use in design of structural assemblies not covered by or differing from conventional construction as set forth in the Minimum Property Standards.

A–2 The structure, including the component parts, shall have sufficient strength and rigidity to support the design load and to resist deformation without exceeding the allowable design stress or deflection provided herein.

A–3 The strength and rigidity of individual members or assemblies shall be determined by a qualified engineer or architect in accordance with recognized engineering analysis procedures. Where assemblies or details of construction are of such nature that the strength, rigidity and other properties cannot be determined by analysis, these properties shall be determined by suitable tests.

B DESIGN DEAD LOADS

B–1 Dead loads used in calculations shall consist of actual weights of all materials making up the construction including walls, floors, roofs, ceilings, partitions, stairways and fixed service equipment.

B–2 Where a choice of finishing or covering material is possible, design dead load shall be based upon the heavier material.

C DESIGN LIVE LOADS

C–1 Design live loads shall consist of the weight of all moving and variable loads that may be placed in the building, including loads on floors, operational loads on roofs and ceilings and wind, snow and earthquake loads which may act upon the structure, either singly or in combination with other dead and live loads.

Note: The design live loads specified herein are minimum and may not meet local building code requirements.

C–2 FLOOR LOADS

Design live floor loads shall be not less than the uniformly distributed loads shown in Table I.

TABLE I
Floor Loads

Location	Live load (psf)[1]
Dwelling rooms (other than sleeping quarters)	[2] 40
Dwelling rooms (sleeping quarters only)	[2] 30
Attics (served by permanent or disappearing stair)	[2] 30
Attics (limited storage roof slope over 3 in 12)	[2] 20
Attics (without storage roof slope 3 in 12 or less)	[2 3] 15
Stairs	60
Public stairs and corridors (two family dwellings)	60
Garages and carports (passenger cars)	[4] 100

[1] Design live load on any member supporting 150 square foot or more may be reduced at the rate of 0.08% per square foot of area supported by the member.
[2] Wood joists span tables for these live loads in combination with average dead loads are contained in Appendix B.
[3] Minimum combined live plus average dead load.
[4] Consideration must be given to effect of concentrated wheel loads on floor.

C–3 ROOF LOADS

Design live roof loads shall not be less than the uniformly distributed loads shown in Table II.

TABLE II
Roof Loads

Roof slope	Live load (psf)[1]
Slope 3 in 12 or less:	
Minimum load [2]	[3] 20
Roof used as deck	40
Slope over 3 in 12: Minimum load [2]	[3] 15

[1] Actual area measured along slope of roof.
[2] Where unusual snow or wind conditions occur, higher design loads may be required to prevent overstressing members. See ASA A58.1 "Minimum Design Loads in Buildings and Other Structures".
[3] Wood joist and rafter span tables for these live loads in combination with average dead loads are contained in Appendix B.

C–4 DESIGN WIND LOADS

a. Design live loads for wind shall not be less than those shown below. Loads are assumed to act horizontally on the gross area of the vertical projections of the structure except as noted for roof design.

(1) Buildings (for overturning or racking), 20 psf.

(2) Chimneys, 30 psf.

(3) Exterior walls, 20 psf. acting inward or outward.

(4) Partitions, 15 psf.

(5) Roofs:

(a) Design to withstand pressure acting outward normal to the surface, equal to 1¼ times the design wind load.

(b) Roofs with slopes greater than 6 in 12 shall be designed to withstand pressures acting inward normal to the surface, equal to the design wind load.

b. Roof framing shall be adequately secured to walls or columns to resist design wind loads. Walls and columns shall be anchored to foundations to resist uplift, overturning or displacement unless calculations are provided which indicate that the overturning moment due to wind forces is less than ⅔ the moment of stability of the structure due to dead load only.

c. In areas where extreme winds occur (30 psf or higher) higher design wind loads than those shown above will be required to prevent overstressing members. See ASA A58.1 "Minimum Design Loads in Buildings and Other Structures".

C–5 EARTHQUAKE LOADS

a. Where earthquake design is required, the structure and all component parts shall be designed to resist horizontal forces resulting therefrom.

b. The minimum lateral force at each floor or roof level shall be assumed to be a static force equal to 10 percent of the dead load at or above such floor or roof for structures less than 35 feet in height and 15 percent of the dead load if height is over 35 feet.

c. Parapet walls shall be designed to resist a lateral force equal to 100 percent of their weight.

C–6 COMBINED LOADS

a. All structural members and connections shall be designed for the combined effect of horizontal and vertical loads. It may be assumed that wind and earthquake loads will not occur simultaneously.

b. Members and connections subject to either wind or earthquake forces may be designed on the basis of increased unit stress 33⅓ percent greater than the basic allowable design stresses, provided that the member or connection so designed is not less than that required for other combined dead and live loads computed without increased stresses.

D DESIGN DEFLECTIONS

D–1 Design deflections of structural members when subjected to total loads of live loads specified herein plus actual dead loads shall not exceed the following:

a. Floor joists, beams and girders, and ridge beams supporting roof joists, $\frac{1}{360}$ of the clear span of the member up to 15 feet. Over 15 feet, deflection shall not exceed $\frac{1}{2}$ inch.

b. Ceiling joists and low-slope roof joists, $\frac{1}{240}$ of the clear span of the member up to 15 feet. Over 15 feet, deflection shall not exceed $\frac{3}{4}$ inch.

c. Rafters, $\frac{1}{180}$ of the clear span of the member up to 15 feet. Over 15 feet, deflection shall not exceed 1 inch.

D–2 The design deflections set forth herein have been established on the basis of reducing vibration on floors or that which will not be visually objectionable on ceilings and roofs. These deflections will not prevent cracking of plaster. Where greater rigidity is desired or where it is desired to reduce the possibilty of plaster cracking, smaller design deflections should be used.

E ALLOWABLE DESIGN STRESSES

E–1 MASONRY CONSTRUCTION

Design stresses for masonry shall be in accordance with ASA A41.1—1953, "American Standard Building Code Requirements for Masonry".

E–2 REINFORCED CONCRETE CONSTRUCTION

Design stresses for reinforced concrete shall be in accordance with ACI-318-56, "Building Code Requirements for Reinforced Concrete", published by the American Concrete Institute.

E–3 STEEL CONSTRUCTION

a. Design stresses for structural steel shall be in accordance with "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings", June 1949 Revision, published by the American Institute of Steel Construction.

b. Design stresses for light gage cold-formed steel structural members shall be in accordance with "Specification for the Design of Light Gage Cold-Formed Steel Structural Members", 1956 edition, published by the American Iron and Steel Institute.

E–4 WOOD CONSTRUCTION

a. Design stresses for lumber, both stress grade and non-stress grade, shall be in accordance with Table III.

b. The allowable design stresses for stress graded lumber shown in Table III were determined in accordance with ASTM D245-57T, "Tentative Methods for Establishing Structural Grades of Lumber", using the basis stresses provided for therein and taking into account knot sizes, slope of grading rules for each species. The allowable design stresses for non-stress graded lumber were similarly determined on the basis of knot sizes in accordance with ASTM D 245-57T, assuming other defects to be no more critical than the effect of knots.

c. The design stresses shown in Table III are for normal short time loading conditions, usually associated with housing construction, and are approximately 10 percent greater than design stresses for full load permanently applied. When the minimum design live loads shown in Section C, Tables I and II, are used, no further increase in stress for short time loading shall be used.

d. Where design stresses for stress grade lumber are used, the lumber shall be identified by the grade mark of an association or independent inspection agency recognized by the American Lumber Standards Committee, Washington, D. C. to grade the species.

APPENDIX B - Simplified ASCE 7-02
Wind Loads For Typical Low-Rise Buildings
January 31, 2004

A.1 General. This appendix provides simplified wind loads that result in designs reasonably consistent with the requirements ASCE 7. It is intended for use by qualified design professionals and is subject to the limitations of Section A.2. In this method, a single wind pressure for each roof and wall vertical projected area and the roof horizontal projected area is used to determine main wind force resisting system loads. For components and cladding loads, surface pressures are determined for specific building elements such that multiple pressure zones are not required to be separately evaluated.

A.2 Limitations. These provisions are applicable to buildings meeting the following conditions:

- Light-frame, concrete, or masonry construction using shear walls and horizontal diaphragms to resist lateral loads.
- Mean roof height of 40 feet or less.
- One- and two-family dwellings, apartments, commercial buildings, and other building uses or occupancies with a wind load importance factor of 1.0.

A.3 Wind Design Criteria

A.3.1 Basic Wind Speed
The basic (design) wind speed shall be determined in accordance with Figure A1 or as required by the local governing building code.

A3.2 Wind Exposure and Topography. The provisions of this Appendix are based on wind exposure category B (suburban, urban, or wooded terrain) as defined in ASCE 7. For buildings located in wind exposure category C (open or coastal terrain), tabulated exposure B wind loads shall be increased by a factor 1.4 (see table footnotes as applicable in Section A4). Buildings sited within 10 building heights from the top edge of a prominent topographic feature shall be designed in accordance with ASCE 7. A prominent topographic feature has a ground slope of greater than 15 percent and a vertical rise of greater than 50 feet, and is separated from features of similar or greater height by a distance of more than approximately 100 times the height of the topographic feature.

A3.3 Wind-borne Debris Region. The *wind-borne debris region* shall be defined in accordance with the Figure A1 for Atlantic Ocean and Gulf of Mexico coastal areas as follows:

Basic Wind Speed ≥ 120 mph – all areas.
110 mph ≤ Basic Wind Speed < 120 mph – all areas within 1 mile of coastline.

A3.4 Building Enclosure Condition
Building enclosure condition shall be classified in accordance with Table A1 for the purpose of determining wind loads in accordance with Section A4.2 and A4.3.

A3.5 Counteracting Dead Load
When dead load is used to counteract

effects of wind pressure, it shall be factored as follows for Allowable Strength Design (ASD) and Load and Resistance Factor or Strength Design (LRFD) methods:

ASD: W - 0.6D

LRFD: 1.6W - 0.9D

where W is wind load effect due to wind loads determined in accordance with Section A4 and D is dead load effect due to estimated actual dead load. Load effects include stresses in or forces applied to structural members, connections, or systems.

Other load combinations and design load effects shall be considered in accordance with ASCE 7, Chapter 2.

A.4 Wind Loads

A4.1 Lateral Force Resisting System Loads. Wind pressures from Table A2 shall be applied to building roof and wall vertical projected areas (VPA) corresponding to each of four elevations of the building to determine maximum lateral wind forces (shear) tributary to horizontal diaphragms, shear walls, and related connections.

A4.2 Roof System Uplift Loads. Wind pressures from Table A3 shall be applied to the horizontal projected area (HPA) of a roof system to determine uplift loads tributary to structural elements, assemblies, and connections that experience loads from multiple roof surfaces.

A4.3 Components and Cladding Loads. Table A4 shall be used to determine inward (positive) and outward (negative) acting wind loads tributary to wall and roof components, cladding, and related connections. Design wind pressures shall be applied perpendicular to the tributary area of the component, cladding, or connection under consideration.

Figure B1
Basic Gust Wind Speed (Gust), MPH

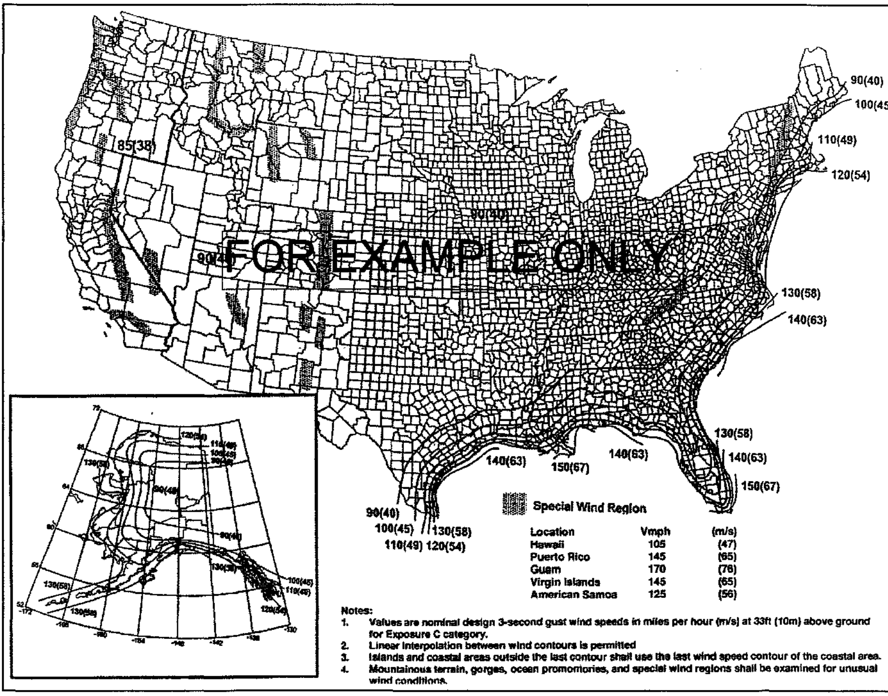

Table B1
Classification of Building Enclosure Condition

Partially-Enclosed Building	Enclosed Building
Buildings meeting one of the following: • All buildings with intentional openings in the exterior envelop exceeding the lesser of 4 ft^2 or 1 percent of the total projected wall or roof area on any building side, or • Buildings within the *wind-borne debris region* with conventional exterior glazing (unprotected from debris impact) exceeding the above opening amounts	All buildings not classified as 'partially enclosed' including: • Buildings not within the *wind-borne debris region*, and • Buildings within the *wind-borne debris region* with glazing protection or impact resistant glazing in accordance with ASCE 7 or the local governing building code.

Table B1 Notes:

1. Building enclosure condition affects internal pressures experienced within the building. Because internal pressure acts inward or outward on all exterior building surfaces simultaneously, the net effect on lateral building loads is zero. Therefore, building enclosure condition does not affect determination of lateral building loads in Section A4.1.
2. Open buildings are not addressed; refer to ASCE 7 for appropriate wind loads. Open buildings have openings in each wall which exceed 80 percent of the wall area.

TABLE B2
Lateral Wind Loads for Application to Vertical Projected Wall and Roof Area [Exposure B, Mean Roof Height 30 feet]

Basic Wind Speed (mph)	Design Wind Pressure (psf)					
	For Roof VPA by Roof Slope			For Wall VPA by Roof Slope		
	≤ 20° (4:12)	25° (5.6:12)	≥ 30° (7:12)	≤10° (2:12)	20° (4:12)	≥30° (7:12)
85	0	2.4	7.7	10.2	12.5	11.2
90	0	2.7	8.6	11.4	14.0	12.6
100	0	3.3	10.7	14.0	17.3	15.5
110	0	4.0	12.9	17.0	20.9	18.8
120	0	4.8	15.4	20.2	25.3	22.4
130	0	5.6	18.0	23.7	29.2	26.3
140	0	6.5	20.9	27.5	33.6	30.5
150	0	7.4	24.0	31.6	38.9	35.0

Table B2 Notes:

1. Table applies to wind exposure category B (urban, suburban, or wooded terrain). For exposure category C (open or coastal exposure), multiply table values by 1.4.
2. Table applies to a mean roof height of 30 feet. For other mean roof heights from 15 feet to 40 feet, multiply table values by the following factor: $f_h = 0.0087\,(h) + 0.74$ where h is the mean roof height in feet.
3. Interpolation between reported wind speeds and roof slopes shall be permitted. For roof slopes greater than 45° (12:12), use wall VPA value.
4. Extrapolation to wind speeds other than shown shall be permitted by multiplying tabulated values by the ratio of squared wind speeds. For example, a wall VPA pressure of 20.9 psf at 110 mph from the table can be used to determine a pressure for a 170 mph wind speed by multiplying as follows: $(20.9 \text{ psf}) \times (170/110)^2 = 49.9$ psf.

TABLE B3
Wind Uplift Loads for Application to Roof System Horizontal Projected Area [Exposure B, Mean Roof Height 30 feet]

Basic Wind Speed (mph)	Roof Uplift Pressure (psf) by Building Enclosure Condition and Roof Slope				Overhang Uplift Pressure (psf) by Roof Slope	
	Partially-Enclosed		Enclosed			
	≤20° (4:12)	≥25° (5.6:12)	≤20° (4:12)	≥25° (5.6:12)	≤20° (4:12)	≥25° (5.6:12)
85	17	11	13	8	19	12
90	19	13	14	9	21	13
100	23	16	17	11	26	17
110	28	19	21	13	32	20
120	33	23	25	16	38	24
130	39	27	29	18	45	28
140	45	31	34	21	52	32
150	52	35	39	24	60	37

Table B3 Notes:

1. Table applies to wind exposure category B (urban, suburban, or wooded terrain). For exposure category C (open or coastal exposure), multiply table values by 1.4.
2. Table applies to a mean roof height of 30 feet. For other mean roof heights from 15 feet to 40 feet, multiply table values by the following factor: $f_h = 0.0087\,(h) + 0.74$ where h is the mean roof height in feet.
3. For hip roofs, multiply roof uplift pressure by 0.9 for roof slope less than 25° (5.6:12) and 0.8 for roof slope greater than 25° (5.6:12). This adjustment does not apply to overhangs on hip roofs.
4. Apply roof uplift pressure to horizontal projected area bounded by exterior walls. Apply overhang uplift pressure to horizontal projected area of overhangs projecting outward from exterior walls.
5. Interpolation for roof slopes between 20° (4:12) and 25° (5.6:12) and reported wind speeds shall be permitted.
6. Extrapolation of tabulated pressures to wind speeds other than shown shall be done in accordance with note 4 of Table A2.

TABLE B4
Design Wind Pressures (psf)
For Components and Cladding
[Enclosed Building, Exposure B, Mean Roof Height 30 feet]

Component	85 mph		90 mph		100 mph		110 mph	
	Inward	Outward	Inward	Outward	Inward	Outward	Inward	Outward
Roof Components								
Roof Framing	12	-12	13	-13	17	-17	20	-20
Roof Sheathing, Purlins, etc.	12	-18	13	-20	17	-25	20	-30
Roof Skylights, Glazing, etc.	12	-18	13	-20	17	-25	20	-30
Roofing (non-air permeable)[6]	12	-21	13	-23	17	-29	20	-35
Roof Overhang (framing only)[7]	12	-24	13	-27	17	-33	n/a	-40
Wall Components								
Wall Framing	12	-12	13	-13	17	-17	20	-20
Wall Sheathing (panels, boards, girts)	12	-18	13	-20	17	-25	20	-30
Windows, Doors, Glazing	12	-18	13	-20	17	-25	20	-30
Garage Doors	12	-12	13	-13	17	-17	20	-20
Siding (non-air permeable)[6]	12	-18	13	-20	17	-25	20	-30

Component	120 mph		130 mph		140 mph		150 mph	
	Inward	Outward	Inward	Outward	Inward	Outward	Inward	Outward
Roof Components								
Roof Framing	24	-24	27	-27	32	-32	37	-37
Roof Sheathing, Purlins, etc.	24	-36	27	-42	32	-49	37	-56
Roof Skylights, Glazing, etc.	24	-36	27	-42	32	-49	37	-56
Roofing (non-air permeable)[5]	24	-42	27	-49	32	-57	37	-65
Roof Overhang (framing only)[6]	n/a	-48	n/a	-56	n/a	-65	n/a	-74
Wall Components								
Wall Framing	24	-24	27	-27	32	-32	37	-37
Wall Sheathing (panels, boards, girts)	24	-36	27	-42	32	-49	37	-56
Windows, Doors, Glazing	24	-36	27	-42	32	-49	37	-56
Garage Doors	24	-24	27	-27	32	-32	37	-37
Siding (non-air permeable)[5]	24	-36	27	-42	32	-49	37	-56

Table B4 Notes:

1. Table applies to wind exposure category B (urban, suburban, or wooded terrain). For exposure category C (open or coastal exposure), multiply table values by 1.4.
2. Table applies to enclosed buildings. For partially-enclosed buildings, multiply table values by 1.25.
3. Table applies to a mean roof height of 30 feet or less. For mean roof heights greater than 30 feet and not exceeding 40 feet, multiply table values by the following factor: $f_h = 0.0087\ (h) + 0.74$ where h is the mean roof height in feet.
4. Interpolation between reported wind speeds shall be permitted. Extrapolation of tabulated pressures to wind speeds other than shown shall be done in accordance with note 4 of Table A2.
5. Non-air permeable claddings (siding and roofing) do not allow venting of air either through the siding or through cavities behind the cladding that lead to vent openings on the same face of the building. Most claddings are air-permeable to some degree and provide some reduction in wind load, provided the supporting wall behind the cladding is relatively non-air permeable. For vinyl cladding, ASTM Standard D3679 permits a 50 percent reduction in wind load for this reason. Similarly, claddings such as brick veneer (with weeps and vent space) and hardboard lap siding have been reported to experience cladding wind load reductions of 30 percent or more. Wind loads on roofing, such as asphalt shingles, have been reported to experience wind load reductions of as much as 25 percent. Refer to the cladding manufacturer for an appropriate air-permeable cladding reduction factor to use. Consideration should also be given to the dynamic nature of wind pressures (e.g., fluttering) and its potential effect (e.g., fatigue) on some cladding systems and related connections.
6. Roof overhang pressure includes pressure from underside of the overhang as well as on the upper surface. If an "open soffit" is used, the roof overhang pressure should also apply to the roof sheathing (if sheathed) or the roofing (if not sheathed underneath).

References: *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-02, ASCE, Reston, VA. 2002.

Structural Loads for One- and Two-Family Dwellings, U.S. Dept. of Housing and Urban Development, Washington, DC. 2001.

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