

Panel Design Specification



WOOD: THE NATURAL CHOICE



Engineered wood products are among the most beautiful and environmentally friendly building materials. They are produced efficiently from a renewable, sustainable, biological resource. Their various sizes and dimensions mean less construction jobsite waste and lower disposal costs. In completed buildings, they store carbon and deliver decades of strong, dependable structural performance. Plus, wood's natural properties, combined with highly efficient wood-frame construction systems, make it a top choice in energy conservation.

Forest land comprises about **33% of the total land** area of the United States^a.



Only about **one-third** of that forest land is open to logging^b.



The volume of annual net timber **growth in the U.S. is 36% higher** than the volume of annual timber removals^c.



American landowners **plant more than 2.5 billion trees** every year^d.



A young, growing forest **produces 1 ton of oxygen** and **absorbs 1.4 tons of CO₂** for every 1 ton of wood^e.



Forests in the United States continue to sequester more carbon than they emit each year, and combined with urban forests and harvested wood products, **offset nearly 15%** of total greenhouse gas emissions in 2012^a.



A FEW FACTS ABOUT WOOD:

Life Cycle Assessment (LCA) measures the long-term green value of wood.

Studies by Consortium for Research on Renewable Industrial Materials (CORRIM) scientifically validate the environmental credentials of wood as a green building product. LCAs examine building products' life cycle—from extraction of the raw material to demolition of the building at the end of its long lifespan. CORRIM found that wood had a more benign environmental footprint than steel or concrete in energy use, climate change, air emissions, water emissions and solid waste production. Report details are available at www.CORRIM.org.

Environmental Product Declarations (EPDs) verify specific products.

The American Wood Council and Canadian Wood Council have published declarations and transparency briefs for engineered wood products, including I-joists, plywood, oriented strand board, glulam, laminated veneer lumber and laminated strand lumber. Use of products with verifiable EPDs may be used toward LEED v4 credit. Similar incentives may be found in Green Globes, the International Green Construction Code and the Architecture 2030 Challenge. EPDs and transparency briefs are available from www.awc.org/greenbuilding/epd.

Wood is the natural choice for the environment, design and strong, resilient construction.

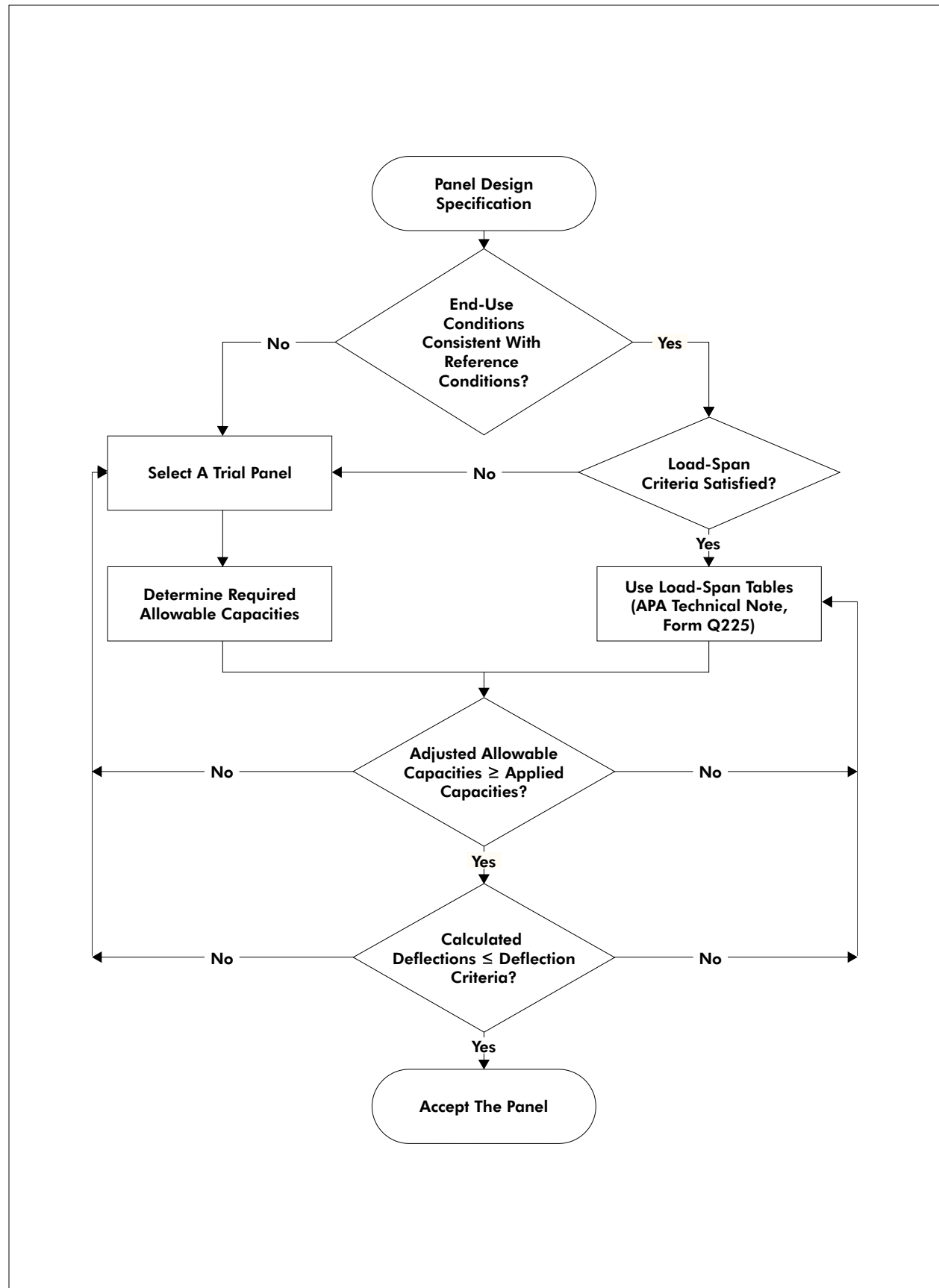
a. U.S. Forest Resource Facts and Historical Trends, U.S. Department of Agriculture, U.S. Forest Service, 2014
b. Consortium for Research on Renewable Industrial Materials (CORRIM)
c. Alvarez, M. The State of America's Forests: Society of American Foresters, 2007
d. Jones, S. Creating a Sustainable Supply for Woody Biomass, Forest Landowners Association, 2011
e. The American Hardwood Information Center, Hardwood Manufacturers Association

The Panel Design Specification presents recommended design capacities and design methods for wood structural panels when used in building construction and related structures. Design information on other wood structural panel applications such as concrete forming, trench shoring, pallets, bins, tanks, shelving and agricultural structures can be found in other publications by APA – The Engineered Wood Association. The information stems from extensive and continuing test programs conducted by APA, other wood structural panel certification bodies and the United States Forest Products Laboratory and is supported by years of satisfactory experience. Information in this specification applies to untreated (except as noted) wood structural panels made in accordance with Voluntary Product Standard PS 1-19 or PS 2-18, promulgated by the United States Department of Commerce (DOC), and with APA manufacturing standards and specifications.

The technical data in this specification are presented as the basis for competent engineering design. For such design to result in satisfactory service, adequate materials and fabrication are also required. All wood structural panels should bear the trademark of a product certification body, such as APA.

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Panel Design Specification

1. INTRODUCTION

Wood structural panels available today respond to changes in wood resources, manufacturing and construction trends; meet designer needs for excellent strength; and are lightweight while using the only renewable building material. A wood structural panel is a panel product composed primarily of wood, which, in its end use, is essentially dependent upon certain structural and physical properties for successful performance in service. Such a product is manufactured to standards that clearly identify its intended end use. Today, wood structural panels include plywood and mat-formed panels such as oriented strand board (OSB). Composite panels containing a combination of veneer and wood-based material have also been produced.

In the early days of plywood manufacturing, every mill worked with a small number of species only and used nearly identical technology. Manufacturing techniques did not vary much from mill to mill. To produce panels under prescriptive standards, a mill used wood of a certain species, peeled it to veneer of a prescribed thickness, and then glued the veneers together in a prescribed manner using approved adhesives.

As technology changed, mills started using a broader range of species and different manufacturing techniques. With the development of *DOC Product Standard PS 1-66 for Softwood Plywood – Construction & Industrial*, three existing plywood standards were combined into one. And, for the first time, span ratings for construction uses were incorporated into the standard. The span rating concept would later be used as a basis for the development of performance standards.

At the same time, there was a growing need to increase efficient use of forest resources. Working in cooperation with the U.S. Forest Service, the American Plywood Association (now APA – The Engineered Wood Association) tested panels manufactured with a core of compressed wood strands and traditional wood veneer on the face and back for use in structural applications. By using cores composed of wood strands, manufacturers were able to make more efficient use of the wood resource and use a broader range of species. These panels are called composite panels.

In the course of the research on composite panels, performance standards were developed that led to a system of performance-rated panels. Soon, manufacturers were making wood structural panels composed entirely of wood strands. Most current production of these panels, intended for use in structural applications, is referred to as oriented strand board or OSB.

1.1 Plywood

Plywood is the original wood structural panel. It is composed of thin sheets of veneer or plies, arranged in layers to form a panel. Plywood always has an odd number of layers, each one consisting of one or more veneers or plies.

In the manufacturing of plywood, a log is turned on a lathe and a long knife blade peels the veneer. The veneers are clipped to a suitable width, dried, graded, and repaired if necessary. Next, the veneers are laid up in cross-laminated layers. Sometimes a layer will consist of two or more plies with the grain running in the same direction, but there will always be an odd number of layers, with the face layers typically having the grain oriented parallel to the long dimension of the panel.

Moisture-resistant adhesive is applied to the veneers that are to be laid up. Laid-up veneers are then put in a hot press where they are bonded to form panels.

Wood is strongest along its grain and shrinks and swells most across the grain. By alternating grain direction between adjacent layers, strength and stiffness in both directions are maximized and shrinking and swelling are minimized.

1.2 Oriented Strand Board

Panels manufactured of compressed wood wafers or strands have been marketed with such names as waferboard and oriented strand board (OSB). Today, virtually all mat-formed wood structural panels are manufactured with oriented strands, and are commonly called oriented strand board or OSB.

OSB is composed of compressed strands arranged in layers (usually three to five) oriented at right angles to one another and bonded under heat and pressure with a moisture-resistant adhesive. The orientation of strands into directional layers achieves the same advantages of cross-laminated veneers in plywood. Since wood is stronger along the grain, the cross-lamination distributes wood's natural strength in both directions of the panel.

Most OSB sheathing panels have a non-skid surface on one side for safety on the construction site, particularly when used as sheathing on pitched roofs.

2. SELECTING PANELS

Wood structural panels are selected according to a number of key attributes. These attributes are identified in the APA trademark found on the panel. Only information bracketed between double horizontal or vertical bars is part of the APA trademark. Examples are seen in Figure 1, and further explained in the paragraphs that follow.

2.1 Standards

Manufacturing standards for wood structural panels are primarily of two types: prescriptive- or performance-based. In the past, plywood standards have been primarily of the prescriptive type. The prescriptive standard approach provides a recipe for panel layup, specifying the species of veneer and the number, thickness and orientation of plies that are required to achieve panels of the desired nominal thickness and strength. An alternative approach is to utilize performance-based standards. Such standards specify performance levels required for common end uses rather than manufacturing aspects of construction. Performance standards permit OSB and plywood to be rated similarly for uses in the construction market.

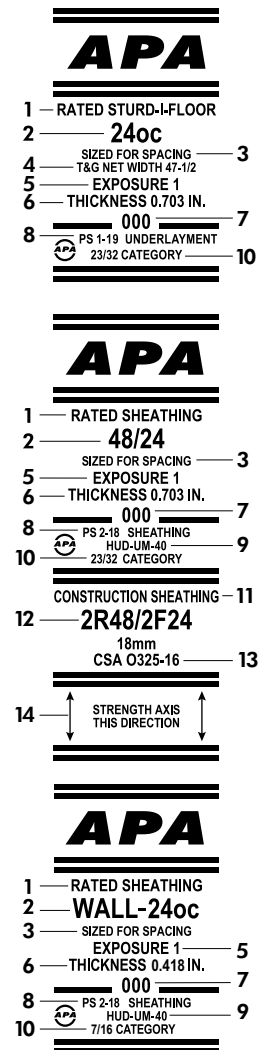
Another distinction between standards is whether they are consensus-based or proprietary. Consensus-based standards are developed following a prescribed set of rules that provide for input and review by a product standard committee of varying interests following one of several recognized procedures. Other standards are of a proprietary nature and may be developed by a single company or industry group. Sometimes proprietary standards become the forerunners of consensus standards. This was the case with APA's proprietary standard PRP-108, *Performance Standards and Qualification Policy for Wood Structural Panels*, which became the foundation for the consensus-based Voluntary Product Standard PS 2, which was developed to achieve broader recognition of performance standards for wood structural panels.

2.1.1 Voluntary Product Standard PS 1

Voluntary Product Standard PS 1, *Structural Plywood*, is a consensus standard that originated in 1966 when it combined several preceding U.S. Commercial Standards, each covering a

FIGURE 1

TYPICAL APA PANEL TRADEMARKS



1. Panel grade
2. Span rating
3. Sized to accommodate panel spacing
4. Tongue-and-groove and net width
5. Bond classification
6. Decimal thickness designation (this value is generally at or near the lower tolerance specified in PS 1 or PS 2)
7. Mill number
8. Product Standard
9. HUD recognition
10. Performance Category
11. Panel grade, Canadian standard
12. Panel mark—Rating and end-use designation per the Canadian standard
13. Canadian performance rated panel standard
14. Panel face grain orientation indicator

different species of plywood. While originating as a prescriptive standard, the 1983 version added performance-based provisions as an alternative method of qualifying sheathing and single-floor grades of plywood for span ratings. PS 1 continues to offer only prescriptive provisions for other panel grades, such as a variety of sanded plywood grades. Plywood manufactured in conformance with PS 1 is recognized in all model and local building codes in the United States. Consult www.apawood.org for the latest version of PS 1.

2.1.2 Voluntary Product Standard PS 2

Voluntary Product Standard PS 2, *Performance Standard for Wood Structural Panels*, was promulgated in 1992 as the first consensus-based performance standard for wood structural panels. The standard was based on APA's PRP-108.

PS 2 applies to all wood structural panels, including plywood, OSB and composite panels. It covers sheathing and single-floor grades and includes performance criteria, qualification requirements and test methods. Wood structural panels manufactured in conformance with PS 2 are recognized in all model and local building codes in the United States. Also developed in concert with PS 2, with virtually identical provisions, was CSA O325, *Construction Sheathing*, which is recognized in the National Building Code of Canada. Consult www.apawood.org for the latest version of PS 2.

2.1.3 Proprietary Standard

The prototype proprietary performance standard for wood structural panels is APA PRP-108. The APA standard includes performance provisions for sheathing and single-floor grades, but also includes provisions for siding, as recognized in the International Code Council Evaluation Service (ICC-ES) Evaluation Report ESR-2586. Although PRP-108, promulgated in 1980, is quite mature, it remains in effect to take advantage of technical developments more expeditiously than would be possible with the rather time-consuming consensus process required by PS 2.

2.1.4 Thickness Designation and Performance Category

Up until 2008, the thickness of wood structural panels like plywood and OSB was designated and labeled by the panel's nominal thickness, which was subject to a plus and minus tolerance specified in the Voluntary Product Standards PS 1 and PS 2. The tolerance on unsanded panel types used in construction is plus or minus 1/32 inch of the designated thickness. The tolerance on sanded grades of plywood is plus or minus 1/64 inch of the designated thickness. These thickness tolerances were applied at the time of manufacturing or at a standard dry condition since it is recognized that actual panel thickness may naturally change due to changes in panel moisture contents.

Model codes, technical recommendations, designs and specifications have been based upon the use of these panel nominal thicknesses. However, packaging and labeling regulations adopted as state and local law specify that labeling of dimensions comply with standards developed by the National Conference on Weights and Measures. These regulations require dimensional labeling that is incompatible with the thickness tolerances specified in PS 1 and PS 2. To jointly comply with these regulations while maintaining the specifications within model codes and other existing

specifications used in the construction industry, wood structural panels are now labeled with both a “Performance Category” and a decimal thickness designation. The decimal thickness designation is generally at or near the lower thickness tolerances permitted in PS 1 and PS 2.

The term “Performance Category” is defined within PS 1 and PS 2 as a panel designation related to the panel thickness range that is linked to the nominal panel thickness designations used in the International Building Code (IBC) and International Residential Code (IRC). For purposes of labeling, abbreviations *PERF CAT*, *CAT*, or *Category* are permitted within the panel grade mark. Section 2303.1.5 of the 2018 and 2015 IBC, Section 2303.1.4 of the 2012 IBC and Section R503.2.1 of the 2018, 2015 and 2012 IRC state the following:

The Performance Category value shall be used as the “nominal panel thickness” or “panel thickness” whenever referenced in this code.

This publication widely uses the Performance Category as the panel designation. There are some places where traditional nominal thickness designations are used, and in those instances, they should be considered as equivalent to the Performance Category.

2.2 Veneer

Wood veneer is at the heart of a plywood panel. The veneer used is classified according to species group and grade requirements of PS 1.

2.2.1 Species Groups

While plywood can be manufactured from nearly any wood species, under PS 1, over 70 species of wood are rated for use based on strength and stiffness. This grouping into five Groups is presented in Table 1. Strongest species are in Group 1, the next strongest in Group 2, and so on. The Group number that appears in the trademark on most non-span-rated panels—primarily sanded grades—is based on the species used for face and back veneers. Where face and back veneers are not from the same species Group, the higher Group number (the lower strength species) is used, except for sanded panels with Performance Category of 3/8 or less and decorative panels of any thickness. These latter panels are identified by face species because they are chosen primarily for appearance and used in applications where structural integrity is not critical. Sanded panels with Performance Category greater than 3/8 are identified by face species if C- or D-grade backs are at least 1/8 inch and are no more than one species group number higher. Some species are used widely in plywood manufacturing, others rarely. The specifier should check local availability if a particular species is desired.

TABLE 1

CLASSIFICATION OF SPECIES^a

Group 1	Group 2	Group 3	Group 4	Group 5	
North American Species – Applicable to trees grown in North America					
Beech, American Birch Sweet Yellow Douglas-fir ^b Larch, Western Maple, Sugar Pine, Southern Loblolly Longleaf Shortleaf Slash Tanoak	Cedar, Port Orford Cypress Douglas-fir ^b Fir Balsam California Red Grand Noble Pacific Silver White Hemlock, Western Maple, Black	Pine Pond Red Virginia Western White Spruce Black Red Sitka Sweetgum Tamarack Yellow Poplar	Alder, Red Birch, Paper Cedar, Alaska Fir, Subalpine Hemlock, Eastern Maple, Bigleaf Pine Jack Lodgepole Ponderosa Spruce Redwood Spruce Engelmann White	Aspen Bigtooth Quaking Cedar Incense Western Red Cottonwood Eastern Black (W. Poplar) Pine Eastern White Sugar	Basswood Poplar, Balsam
Non North American Species					
Apitong ^{c,d} Kapur ^c Keruing ^{c,d} Pine Caribbean Ocote	Lauan Almon Bagtikan Mayapis Red Lauan Tangile White Lauan	Mengkulang ^c Meranti, Red ^{c,e} Mersawa ^c		Cativo	

a. Table 1 species classified in accordance with ASTM D2555 as discussed in Appendix A of *Voluntary Product Standard PS 1-19, Structural Plywood*, APA Form L870. The species groupings are only valid for species grown in the regions referenced in Appendix A of PS 1-19. (See Section 5.2.1. of PS 1-19 for additional information.)

b. Douglas-fir from trees grown in the states of Washington, Oregon, California, Idaho, Montana, Wyoming, and the Canadian Provinces of Alberta and British Columbia shall be classed as Group 1 Douglas-fir. Douglas-fir from trees grown in the states of Nevada, Utah, Colorado, Arizona and New Mexico shall be classed as Group 2 Douglas-fir.

c. Each of these names represents a trade group of woods consisting of a number of closely related species.

d. Species from the genus *Dipterocarpus* marketed collectively: Apitong if originating in the Philippines, Keruing if originating in Malaysia or Indonesia.

e. Red Meranti shall be limited to species having a specific gravity of 0.41 or more based on green volume and oven-dry weight.

2.2.2 Veneer Grades

Veneer grades define veneer appearance in terms of natural unrepaired growth characteristics and allowable number and size of repairs that may be made during manufacture. See Table 2. The highest quality commonly available veneer grade is A. The minimum grade of veneer permitted in Exterior plywood is C-grade. D-grade veneer is used in panels intended for applications protected from long-term exposure to weather.

2.3 Panel Grades

Wood structural panel grades are generally identified in terms of the veneer grade used on the face and back of the panel (e.g., A-B, B-C), or by a name suggesting the panel's intended end use (e.g., APA Rated Sheathing or simply Sheathing, APA Rated Sturd-I-Floor or simply Single Floor). See Table 3. Unsanded and touch-sanded panels and panels with B-grade or better veneer on one side only usually carry the trademark of a qualified inspection and testing agency, such as APA, on the panel back. Panels with both sides of B-grade or better veneer, or with special overlaid surfaces (such as High Density Overlay) usually carry the trademark on the panel edge.

2.3.1 Unsanded

Sheathing panels are unsanded since a smooth surface is not a requirement of their intended end use for subfloor, roof and wall applications. Sheathing panels are classified by span ratings, which identify the maximum recommended support spacings for specific end uses. Design capacities provided in 4.4 are on the basis of span ratings.

Structural I sheathing panels meet the requirements of sheathing grades as well as enhanced requirements associated with use in panelized roof systems, diaphragms and shear walls for increased cross-panel strength and stiffness and increased racking shear resistance.

2.3.2 Touch-Sanded

Underlayment, Single Floor, C-D Plugged, and C-C Plugged grades require only touch sanding for "sizing" to make the panel thickness more uniform. Panels rated for single floor (combined subfloor and underlayment) applications are usually manufactured with tongue-and-groove (T&G) edge profiles and are classified by span ratings. Single Floor panel span ratings identify the maximum recommended support spacings for floors. Design capacities provided in 4.4 are on the basis of span ratings. Other thinner panels intended for separate underlayment applications (Underlayment or C-C Plugged) are identified with a species group number but no span rating.

TABLE 2

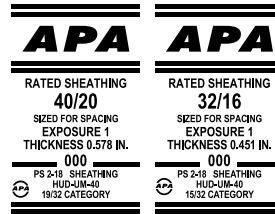
VENEER GRADES

A	Smooth, paintable. Not more than 18 neatly made repairs, boat, sled or router type, and parallel to grain, permitted. Wood or synthetic repairs permitted. May be used for natural finish in less demanding applications.
B	Solid surface. Shims, sled or router repairs, and tight knots to 1 inch across grain permitted. Wood or synthetic repairs permitted. Some minor splits permitted.
C Plugged	Improved C veneer with splits limited to 1/8 inch width and knotholes or other open characteristics limited to 1/4 x 1/2 inch. Wood or synthetic repairs permitted. Admits some broken grain.
C	Tight knots to 1-1/2 inches. Knotholes to 1 inch across grain and some to 1-1/2 inches if total width of knots and knotholes is within specified limits. Synthetic or wood repairs. Discoloration and sanding defects that do not impair strength permitted. Limited splits allowed.
D	Knots and knotholes to 2-1/2 inch width across grain and 1/2 inch larger within specified limits. Limited splits are permitted. Limited to Exposure 1.

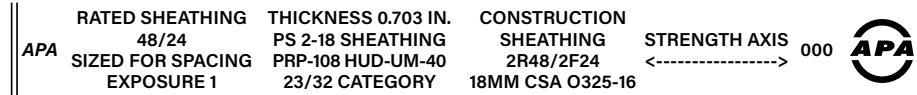
TABLE 3

GUIDE TO APA PERFORMANCE RATED PANELS^{a,b}

APA RATED SHEATHING
Typical Trademark



Typical Inkjet Trademark



Specially designed for subflooring and wall and roof sheathing. Also good for a broad range of other construction and industrial applications. Can be manufactured as OSB, plywood or other wood-based panel.

BOND CLASSIFICATIONS: Exterior, Exposure 1.

COMMON PERFORMANCE CATEGORIES: 3/8, 7/16, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4.

APA RATED SHEATHING
(WALL-24 OR W24)
Typical Trademark

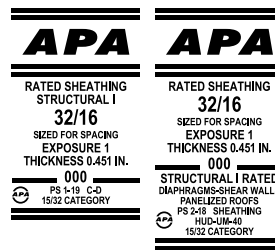


Specially designed for wall applications only. Not intended for roof or floor sheathing. Can be manufactured as OSB panel.

BOND CLASSIFICATIONS: Exposure 1.

COMMON PERFORMANCE CATEGORIES: 3/8, 7/16, 15/32

APA STRUCTURAL I
RATED SHEATHING^c
Typical Trademark

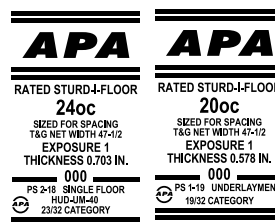


Unsanded grade for use where shear and cross-panel strength properties are of maximum importance, such as panelized roofs and diaphragms. Can be manufactured as OSB, plywood or other wood-based panel.

BOND CLASSIFICATIONS: Exterior, Exposure 1.

COMMON PERFORMANCE CATEGORIES: 3/8, 7/16, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4.

APA RATED
STURD-I-FLOOR
(SINGLE FLOOR)
Typical Trademark



Specially designed as combination subfloor-underlayment. Provides smooth surface for application of carpet and pad and possesses high concentrated and impact load resistance. Can be manufactured as OSB, plywood or other wood-based panel. Available square edge or tongue-and-groove.

BOND CLASSIFICATIONS: Exterior, Exposure 1.

COMMON PERFORMANCE CATEGORIES: 19/32, 5/8, 23/32, 3/4, 7/8, 1-1/8.

Typical Inkjet Trademark



*The 7/16 is available in OSB only.

Continued on next page

TABLE 3 (Continued)

GUIDE TO APA PERFORMANCE RATED PANELS^{a,b}

APA STRUCTURAL I RATED
STURD-I-FLOOR (SINGLE FLOOR)
Typical Trademark

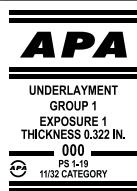


Specially designed as combination subfloor-underlayment. Provides smooth surface for application of carpet and pad and possesses high concentrated and impact load resistance and shear wall and diaphragm performance. Can be manufactured as OSB, plywood or other wood-based panel. Available square edge or tongue-and-groove. Not as common as APA RATED STURD-I-FLOOR, check with supplier before specifying.

BOND CLASSIFICATIONS: Exterior, Exposure 1

COMMON PERFORMANCE CATEGORIES: 19/32, 5/8, 23/32, 3/4, 7/8, 1-1/8.

APA PLYWOOD UNDERLAYMENT
Typical Trademark

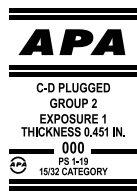


For application over structural subfloor. Provides smooth surface for application of carpet and pad and possesses high concentrated and impact load resistance. For areas to be covered with resilient flooring, specify panels with "sanded face."^d

BOND CLASSIFICATION: Exposure 1 or Exterior.

COMMON PERFORMANCE CATEGORIES^e: 1/4, 11/32, 3/8, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4.

APA C-C PLUGGED or UNDERLAYMENT
C-C PLUGGED PLYWOOD
Typical Trademark

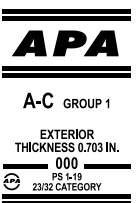


Generally applied where a high quality surface is required. Includes APA A-A, A-B, A-C, A-D, B-B, B-C and B-D grades.

BOND CLASSIFICATION: Exposure 1.

COMMON PERFORMANCE CATEGORIES: 3/8, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4.

APA SANDED PLYWOOD GRADES
Typical Trademark



For use where appearance of only one side is important in exterior or interior applications, such as soffits, fences, farm buildings, etc.^c

BOND CLASSIFICATION: Exterior.

COMMON PERFORMANCE CATEGORIES: 1/4, 11/32, 3/8, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4.

APA MARINE PLYWOOD
Typical Trademark (mark on panel edge)

|| APA MARINE A-A EXT 0.609 IN. 000 PS 1-19 5/8 CAT ||

Ideal where solid cores are needed, such as boat hulls, or where application demands smooth face and minimal voids. Made only with Douglas-fir or western larch. Subject to special limitations on core gaps and face repairs. Also available with HDO or MDO faces.

BOND CLASSIFICATION: Exterior.

COMMON PERFORMANCE CATEGORIES: 1/4, 3/8, 1/2, 5/8, 3/4.

- a. See *APA Engineered Wood Construction Guide*, Form E30, for additional information. Specific grades, Performance Categories and bond classifications may be in limited supply in some areas. Check with your supplier before specifying.
- b. Specify Performance Rated Panels by Performance Category and span rating. Span ratings are based on panel strength and stiffness. Since these properties are a function of panel composition and configuration as well as thickness, the same span rating may appear on panels of different Performance Categories. Conversely, panels of the same Performance Category may be marked with different span ratings.
- c. For Structural I plywood panel constructions, the plies are special improved grades. Panels marked PS 1 are limited to Group 1 species. Other panels marked Structural I qualify through special performance testing.

2.3.3 Sanded

Plywood panels with B-grade or better veneer faces are always sanded smooth in manufacture to fulfill the requirements of their intended end use—applications such as cabinets, shelving, furniture, built-ins, etc. Sanded grades are classed according to Performance Category and the species group of the faces. Design capacities provided in Section 4.4 are based on Performance Category and assume Group 1 faces.

2.3.4 Overlaid

High Density Overlay (HDO) and Medium Density Overlay (MDO) plywood may or may not have sanded faces, depending on whether the overlay is applied at the same time the panel is pressed (one-step) or after the panel is pressed (two-step). For purposes of assigning design capacities provided in Section 4.4, HDO and MDO panels are assumed to be sanded (two-step) with Group 1 faces and B-grade veneer under the overlay.

2.4 Bond Classifications

Wood structural panels may be produced in two bond classifications—Exterior and Exposure 1. The bond classification relates to adhesive bond and thus to structural integrity of the panel. However, the bond classification does not affect the strength and stiffness or span rating of wood structural panels when used as intended.

Bond classification relates to moisture resistance of the glue bond and does not relate to fungal decay resistance of the panel. Fungal decay of wood products may occur when the moisture content exceeds approximately 20% for an extended period. Prevention of fungal decay is a function of proper design to prevent prolonged exposure to moisture, of material specification, of construction and of maintenance of the structure, or may be accomplished by pressure preservative treatment. See *APA Data File: Preservative-Treated Plywood*, Form Q220, and *APA Technical Note: Controlling Decay in Wood Construction*, Form R495, for additional information.

Aesthetic (nonstructural) attributes of panels may be compromised to some degree by exposure to weather. Panel surfaces may become uneven and irregular under prolonged moisture exposure. Panels should be allowed to dry, and panel joints and surfaces may need to be sanded before applying some finish materials.

2.4.1 Exterior

This bond classification signifies plywood capable of withstanding repeated wetting and redrying or long-term exposure to weather or other conditions of similar severity.

2.4.2 Exposure 1

This bond classification signifies panels suitable for uses not involving long-term exposure to the weather. Panels classified as Exposure 1 are intended to resist the effects of moisture on structural performance during construction or other conditions of similar severity. Exposure 1 panels may also be used when exposure to the outdoors is protected from direct exposure, such as the under side of panels at roof overhangs, although appearance characteristics of the panel grade should

also be considered. Exposure 1 panels are made with the same exterior adhesives used in Exterior panels. However, because other compositional factors may affect bond performance, only Exterior panels should be used for long-term exposure to the weather.

C-D Exposure 1 plywood, sometimes called “CDX” in the trade, is occasionally mistaken as an Exterior panel and erroneously used in applications for which it does not possess the required resistance to weather. “CDX” should only be used for applications as outlined under Exposure 1 above.

2.5 Span Ratings

Sheathing and Single Floor grades carry numbers in their trademarks called span ratings. These denote the maximum recommended center-to-center spacing in inches of supports, over which the panels should be placed in normal code-conforming construction. The span rating applies when the long panel dimension or strength axis is across supports, unless the strength axis is otherwise identified on the panel. Note that the floor span rating of “20” is intended for end-use spans of 19.2 inches.

2.5.1 Sheathing

The span rating on Sheathing grade panels appears as two numbers separated by a slash, such as 32/16, 48/24, etc. The left-hand number denotes the maximum recommended spacing of supports when the panel is used for roof sheathing with the strength axis of the panel across three or more supports (two or more spans). The right-hand number indicates the maximum recommended spacing of supports when the panel is used for subflooring with the strength axis of the panel across three or more supports. A panel marked 32/16, for example, may be used for roof sheathing over supports up to 32 inches on center or for subflooring over supports up to 16 inches on center.

Some roof sheathing maximum spans are dependent upon panel edge support. See Section 4.5.6. for more information. Sheathing panels rated for use only as wall sheathing are usually identified as Wall-24 or W24. The numerical index (24) corresponds to the maximum wall stud spacing in inches. Wall sheathing panels are performance tested with the secondary axis (usually the short dimension of panel) spanning across supports, or studs. For this reason, wall sheathing panels may be applied with either the strength axis or secondary axis across supports.

2.5.2 Single Floor (APA Sturd-I-Floor)

The span rating on Single Floor grade panels appears as a single number. Single Floor panels are designed specifically for single-floor (combined subfloor and underlayment) applications and are manufactured with span ratings of 16, 20, 24, 32 and 48 oc. The span ratings for Single Floor panels, like those for Sheathing grade, are based on application of the panel with the strength axis across three or more supports.

3. CODE PROVISIONS

Recommendations given in APA literature for construction applications are generally consistent with provisions given in the model building codes in the United States. However, most of the information herein has been expanded compared to the code provisions so as to be more useful to designers.

The general APA recommendations apply primarily to conventional or non-engineered construction but can also be considered conservative for engineered construction. On the other hand, for engineered construction, codes contain provisions for acceptance of engineering calculations and design capacities given herein may be used. In many cases, calculations using values in this document will lead to higher allowable design loads for Sheathing and Single Floor panels. This is because the general APA and code recommendations are based on the minimum structural requirements or criteria of the performance standards, while the design capacities are based on actual characteristics of panels qualified under the performance standards. Since it would be difficult to manufacture a truly “minimum” panel with regard to all properties, the actual panel characteristics meet or exceed requirements of the performance standards.

Regardless of any increase in allowable load based on calculations, always observe the maximum recommended span (i.e., span rating). Maximum span is established by test and is often controlled by the concentrated load performance requirements.

4. MECHANICAL PROPERTIES

Wood structural panels can typically be incorporated into construction projects without the need for engineering design of the panels themselves. They lend themselves to tabular and descriptive presentation of design recommendations and provisions. Occasionally, however, there is a need to engineer panel applications that call for panel properties or capacities or it may be necessary to evaluate specific panel constructions that yield superior mechanical properties compared to those that are the basis for general use recommendations.

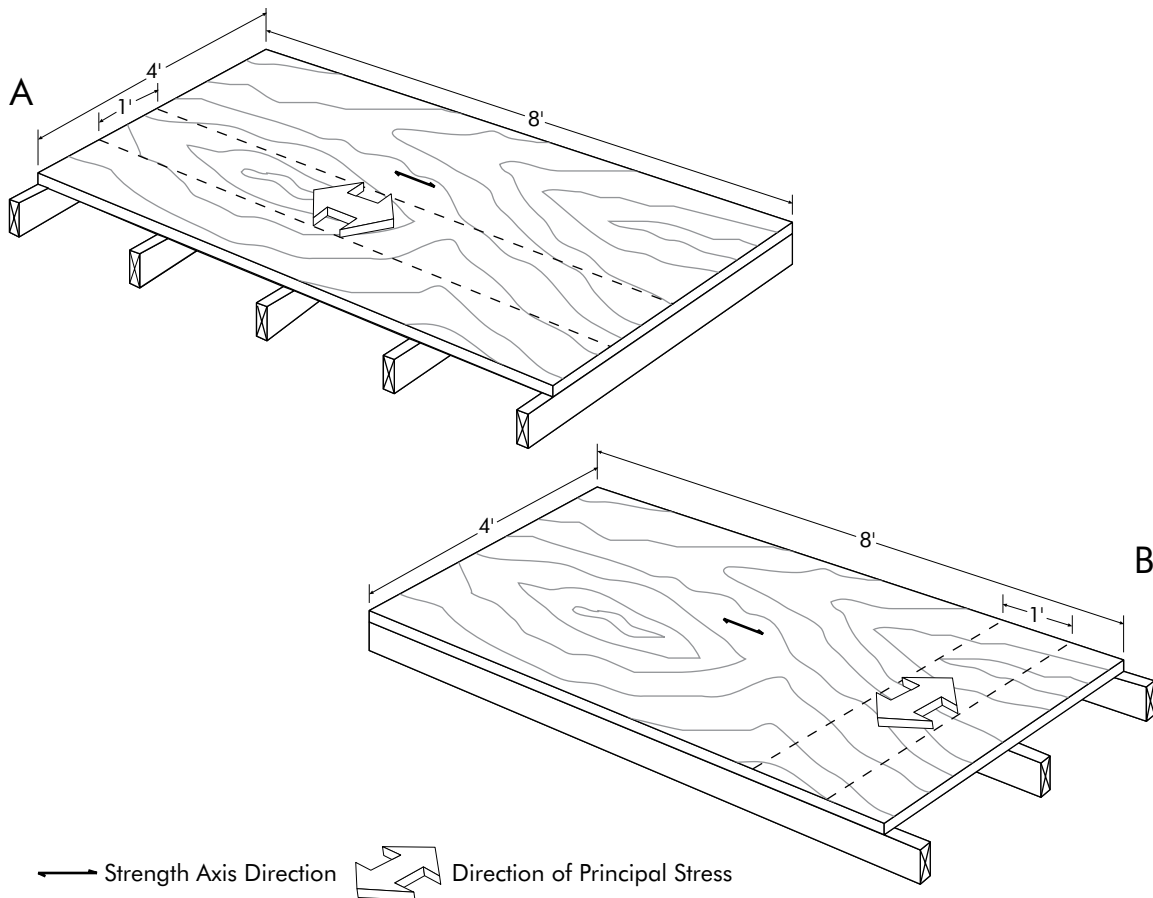
4.1 Strength Axis

A feature of most wood structural panel types, primarily plywood and OSB, is that there is a strength axis associated with their manufacture. The layered construction of both products, in which layers are oriented 90 degrees from one another, creates dissimilar properties in the two principal directions. This is illustrated in Figure 2. The orientation of the face and back layer determines the direction of the strength axis.

The panel strength axis is typically in the long panel direction; that is, the panel is typically stronger and stiffer along the panel length than across the panel width. Specification of panel orientation, then, can be stated as “strength axis is perpendicular (or parallel) to supports” or, sometimes, “stress is parallel (or perpendicular) to the strength axis.” In the case of plywood, the strength axis is sometimes referred to as the face grain direction.

FIGURE 2

TYPICAL WOOD STRUCTURAL PANEL WITH STRENGTH AXIS DIRECTION PERPENDICULAR TO OR ACROSS SUPPORTS (A) AND PARALLEL TO SUPPORTS (B). Note the standard 4' x 8' size, strength axis direction, and representative portion of panel used in calculation of capacities for stress parallel (A) or perpendicular (B) to the strength axis.



4.2 Panel Construction

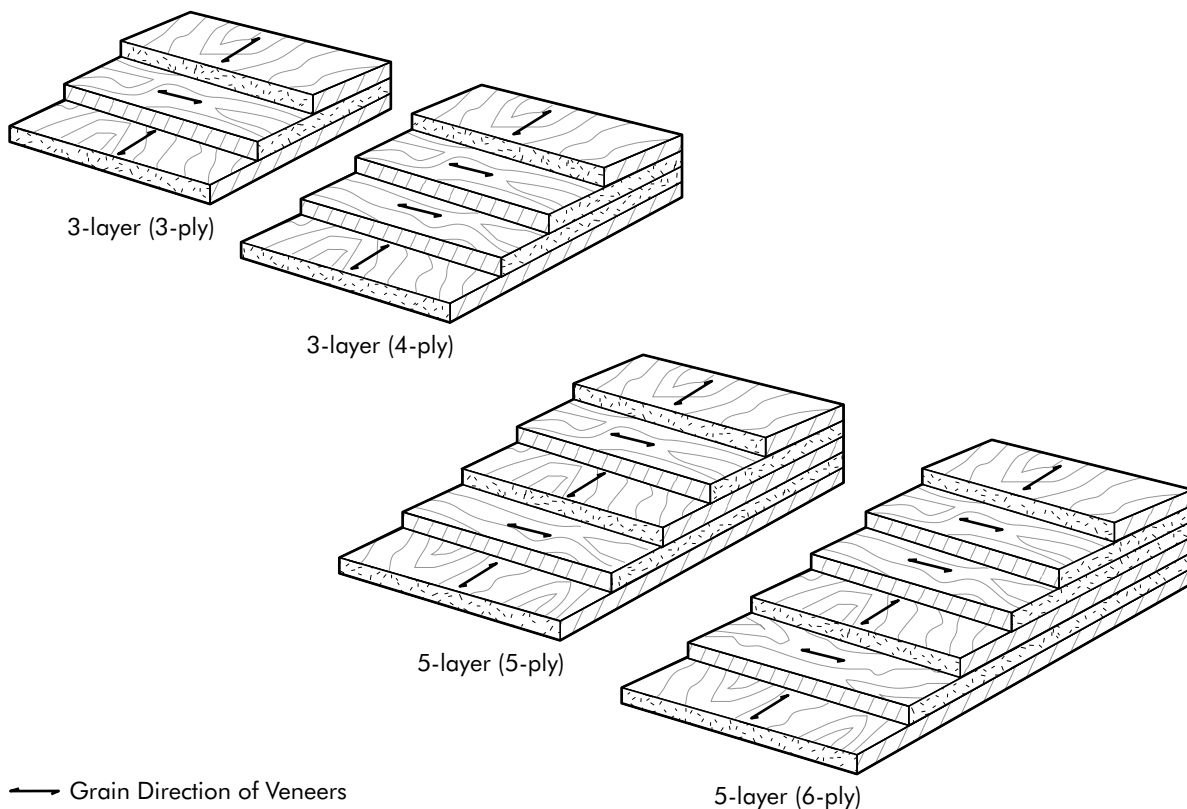
Plywood mills may use different layups for the same panel Performance Category and span rating to make optimum use of the raw material resources. Design calculations must consider the direction in which the stresses will be imposed in the panel. If stresses can be expected in both directions, both the parallel and perpendicular directions should be checked. For this reason, tabulated capacities are given for both directions.

Capacities parallel to the face grain of plywood are based on a panel construction that gives minimum values in that direction (see Figure 3). Capacities perpendicular to the face grain are usually based on a different panel construction that gives minimum values in that direction. Both values, therefore, are conservative. Capacities given for the two directions are not necessarily for the same panel construction.

Similar layers occur also in OSB manufacture. However, the layers are not defined and therefore cannot be specified. For this reason, ply-layer options are not tabulated for OSB.

FIGURE 3

TYPICAL WOOD STRUCTURAL PANEL WITH STRENGTH AXIS DIRECTION PERPENDICULAR TO OR ACROSS SUPPORTS (A) AND PARALLEL TO SUPPORTS (B). Note the standard 4' x 8' size, strength axis direction, and representative portion of panel used in calculation of capacities for stress parallel (A) or perpendicular (B) to the strength axis.



4.3 Properties and Stresses

Plywood properties have traditionally been separately tabulated as section properties and design stresses. These are multiplied together to obtain a capacity. In many cases the resulting capacity will be quite conservative. Design stresses are conservatively developed, taking into account grade factors and manufacturing factors, and then the data is statistically analyzed such that it represents the “low end” of possible values. The stress is then further adjusted by a load factor or, as some call it, a factor of safety.

At the same time, section properties are developed for virtually all possible layup combinations of veneer thickness and species. The lowest property value for a given panel thickness or span rating is then chosen for tabulation. The resulting capacity combines two already conservative values. In the 1990s, this procedure was largely replaced by direct publication of panel capacities, as discussed in Section 4.4. However, the section property and design stress technique are still used occasionally to analyze individual plywood layup variations.

4.4 Capacities

Panel design capacities listed in Tables 9 and 10 are minimum for grade and span rating or Performance Category. For Structural I panels, the tabulated capacities shall be permitted to be multiplied by the “Structural I Multiplier” factors given in the bottom of each property table. Since Table 10 gives capacities for sanded panels marked as Group 1, Table 11 provides multipliers for sanded panel capacities that are identified as Group 2, 3 or 4. The tabulated capacities are based on data from tests of panels bearing the APA trademark. To take advantage of these capacities and adjustments, the specifier must ensure that the correct panel is used in the final construction.

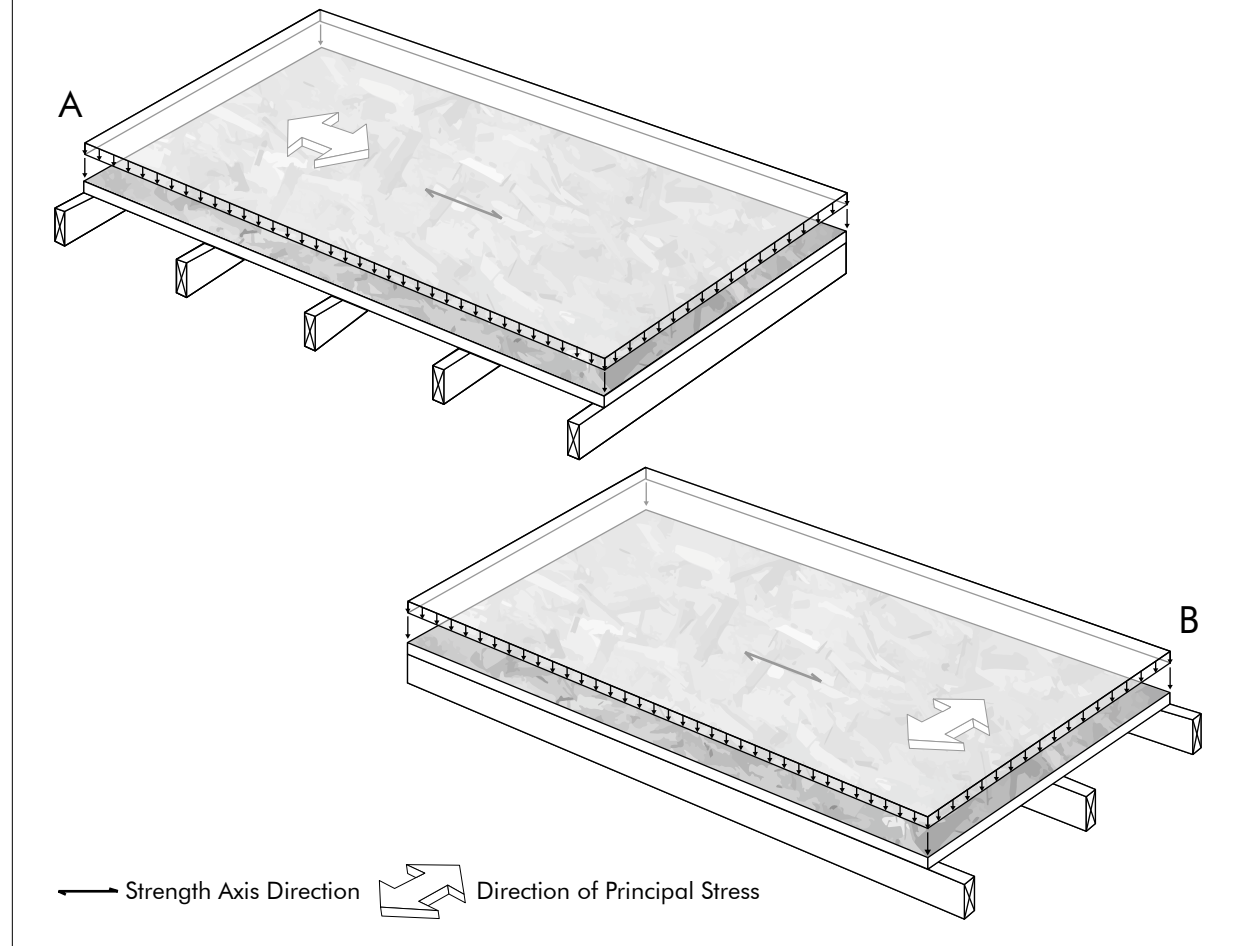
4.4.1 Panel Flexure (Flatwise Panel Bending)

Panel design capacities reported in Tables 9 and 10 are based on flatwise panel bending, as measured by testing in accordance with ASTM D3043, Method C (large panel testing). See Figure 4.

Bending Stiffness (EI) and Modulus of Elasticity (E)

Panel bending stiffness is the capacity to resist deflection and is represented in bending equations as EI. The E is the apparent modulus of elasticity of the material and the I is the moment of inertia of the cross section. Units of EI are lbf-in.² per foot of panel width. In this specification, the tabulated EI is determined by dividing the EI obtained directly from ASTM D3043 Method C tests by a factor of 1.1 to conservatively account for any shear deflection that usually represents a small fraction of the estimated total deflection in typical sheathing applications and an inherent difference in the tested EI obtained from various test configurations. The apparent panel modulus of elasticity (E) can be obtained by dividing the published panel EI by the moment of inertia (I) provided in Table 13.

FIGURE 4

STRUCTURAL PANEL IN BENDING – (A) STRESS PARALLEL TO STRENGTH AXIS AND (B) STRESS PERPENDICULAR TO STRENGTH AXIS**Bending Strength ($F_b S$) and Allowable Bending Stress (F_b)**

Allowable bending strength capacity is the design maximum moment, represented in bending equations as $F_b S$. Terms are the allowable extreme fiber stress of the material (F_b) and the section modulus (S). Units of $F_b S$ are lbf-in. per foot of panel width. The allowable panel bending stress (F_b) can be obtained by dividing the published panel $F_b S$ by the section modulus (S) provided in Table 13.

4.4.2 Panel Axial Strength**Tensile Capacity ($F_t A$) and Allowable Tensile Stress (F_t)**

Allowable tensile capacities are reported in Tables 9 and 10 based on testing in accordance with ASTM D3500, Method B. Tensile capacity is given as $F_t A$, where F_t is the allowable axial tensile stress of the material and A is the cross-sectional area. Units of $F_t A$ are lbf per foot of panel width. The allowable panel axial tensile stress (F_t) can be obtained by dividing the published panel $F_t A$ by the cross-sectional area (A) provided in Table 13.

Compressive Capacity ($F_c A$) and Allowable Compressive Stress (F_c)

Allowable compressive capacities are reported in Tables 9 and 10 based on testing in accordance with ASTM D3501, Method B. Compressive capacity is given as $F_c A$, where F_c is the allowable axial compressive stress of the material, and A is the cross-sectional area. Units of $F_c A$ are lbf per foot of panel width. The allowable panel axial compressive stress (F_c) can be obtained by dividing the published panel $F_c A$ by the cross-sectional area (A) provided in Table 13.

4.4.3 Panel Axial Stiffness (EA) and Axial Modulus of Elasticity (E)

Panel axial stiffness is reported in Tables 9 and 10 based on testing in accordance with ASTM D3501, Method B. Axial stiffness is the capacity to resist axial strain and is represented by EA . The E is the axial modulus of elasticity of the material and A is the cross-sectional area. Units of EA are lbf per foot of panel width. The panel axial modulus of elasticity (E) can be obtained by dividing the published panel EA by the cross-sectional area (A) provided in Table 13.

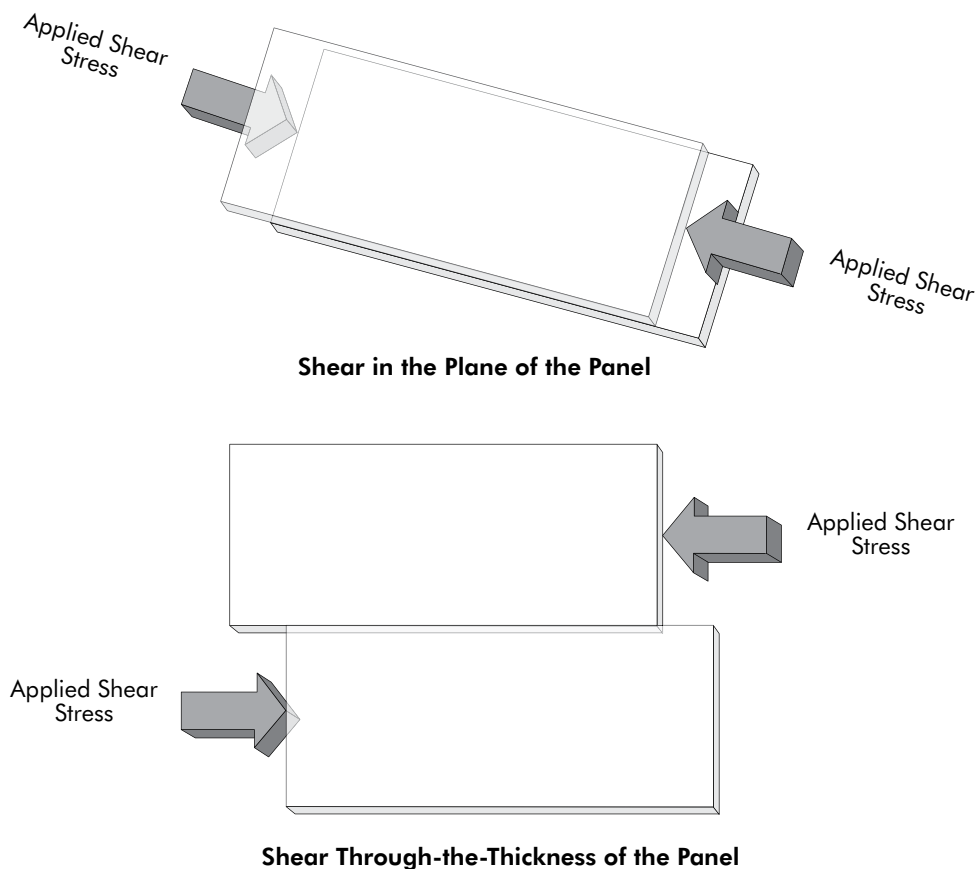
4.4.4 Shear Capacity ($F_s [lb/Q]$) and Allowable Shear Stress (F_s) in the Plane of the Panel

Allowable shear in the plane of the panel (or interlaminar shear, sometimes called “rolling shear” in plywood) is reported in Tables 9 and 10 based on testing in accordance with ASTM D2718. Shear strength in the plane of the panel is the capacity to resist horizontal shear forces when loads are applied or developed on opposite faces of the panel, as they are during flat panel bending. See Figure 5. The term F_s is the allowable interlaminar shear stress, while lb/Q is the panel cross sectional shear constant. Units of $F_s (lb/Q)$ are lbf per foot of panel width. The allowable panel shear in the plane of the panel (F_s) can be obtained by dividing the published panel $F_s (lb/Q)$ by the shear constant (lb/Q) provided in Table 13.

4.4.5 Shear Capacity ($F_v t_v$) and Allowable Shear Stress (F_v) Through the Panel Thickness

Allowable shear through the thickness (sometimes called “in-plane shear capacity” in lateral load design) is the capacity to resist horizontal shear forces when loads are applied or developed on opposite edges of the panel, such as they are in an I-joist web, and is reported in Tables 9 and 10. See Figure 5. Panel shear-through-the-thickness capacities are reported based on testing in accordance with ASTM D2719. Where additional support is not provided to prevent buckling, design capacities in Tables 9 and 10 are limited to sections 2 feet or less in depth. Deeper sections may require additional reductions. The term F_v is the allowable through-the-thickness shear stress, while t_v is the effective panel thickness for shear. Units of $F_v t_v$ are lbf per inch of shear-resisting panel length. The allowable shear through the panel thickness (F_v) can be obtained by dividing the published panel $F_v t_v$ by the panel thickness (t_v) provided in Table 13.

FIGURE 5

TWO TYPES OF PANEL SHEAR: SHEAR THROUGH THE THICKNESS AND SHEAR IN THE PLANE OF THE PANEL**4.4.6 Panel Shear Rigidity ($G_v t_v$) and Modulus of Rigidity (G_v) Through the Panel Thickness**

Panel shear rigidity through the panel thickness (sometimes called “in-plane shear rigidity” in lateral load design) is reported in Tables 9 and 10 and is the capacity to resist deformation when under shear-through-the-thickness stress. Rigidity is represented by $G_v t_v$, where G_v is the modulus of rigidity and t_v is the effective panel thickness for shear. The units of $G_v t_v$ are lbf per inch of panel depth (for vertical applications). Multiplication of $G_v t_v$ by panel depth gives GA , which is used by designers for some applications. The panel modulus of rigidity (G_v) can be obtained by dividing the published panel $G_v t_v$ by the panel thickness (t_v) provided in Table 13.

4.4.7 Panel Bearing Capacity ($F_{c\perp} A$) and Allowable Bearing Stress ($F_{c\perp}$)

As compression load is applied perpendicular to the plane of the plies or to the surface of the panel (such as by columns or by reactions at supports), bearing stress is induced through the bearing area. Panel bearing stress is based on testing in accordance with ASTM D143. The allowable bearing stress of wood structural panels ($F_{c\perp}$) is derived based on the load at a 0.04-in.

deformation limit. A design bearing stress of 360 psi shall be used for wood structural panels under dry-use conditions where the equilibrium moisture content is less than 16%. Multiplying the allowable bearing stress by the bearing area gives the bearing capacity, $F_{c\perp} A$, in lbf.

A reduced design bearing stress may be appropriate where bearing deformation could affect load distribution or where total deformation of members must be closely controlled. The allowable panel bearing stress value at 0.02-inch deformation ($F_{c\perp 0.02''}$) can be chosen as 210 psi.

4.4.8 Reference Fastener Design Values

The reference fastener design values for wood structural panels can be calculated using the provisions provided in the NDS in dry service conditions where the equilibrium moisture content is less than 16%. The equivalent specific gravity (ESG), as shown in Table 4, replaces the lumber specific gravity (G) in the NDS equations for the reference fastener withdrawal and reference fastener head pull-through design values for wood structural panels, as outlined in Sections 4.4.8.1 through 4.4.8.3. The reference fastener lateral design values are provided in Section 4.4.8.4.

TABLE 4

EQUIVALENT SPECIFIC GRAVITY (ESG) FOR WOOD STRUCTURAL PANELS IN DRY SERVICE CONDITIONS FOR FASTENER WITHDRAWAL AND FASTENER HEAD PULL-THROUGH^a

Loading Mode	Fasteners	Applicable Equation	Plywood		
			Group 1	Others	OSB
			Equivalent Specific Gravity (ESG)		
Fastener Withdrawal	Smooth- or Screw-Shank Carbon Steel Nails	1	0.40	0.40	0.40
	Ring-Shank Carbon Steel Nails	2	0.56	0.56	0.56
	Wood Screws	3	0.45	0.45	0.45
Fastener Head Pull-Through	Fasteners (Round Heads) ^b	4 or 5	0.50	0.42 ^c	0.50

a. For use with the equations shown in Sections 4.4.8.1 through 4.4.8.3 of this specification or the equations specified in the NDS.

b. For fasteners with a round-head diameter of 0.234 through 0.500 in.

c. Use 0.42 when species of the plies is not known. When species of the plies is known, specific gravity listed for the actual species may be used or the weighted average may be used for mixed species.

4.4.8.1 Reference Nail Withdrawal Design Values

Under certain circumstances, such as strong gusts of wind or hurricanes, sheathing or siding may be loosened or detached as nails are withdrawn from the wood structural panels. Based on extensive testing conducted by APA in accordance with ASTM D1761, the ESG values for nail withdrawal strength with wood structural panels are listed in Table 4, which can be used to estimate the reference nail withdrawal design values for wood structural panels using Eq. 1 for either smooth- or screw-shank carbon steel nails, or Eq. 2 for ring-shank carbon steel nails. The calculated reference nail withdrawal design values are tabulated in Appendix A, Tables A1 and A2, for ease of reference. The reference nail withdrawal design values shall be multiplied by all applicable adjustment factors (see Section 4.5 of this specification and Table 11.3.1 of the NDS) to obtain adjusted nail withdrawal design values.

$$W_{sn} = 1380 G^{5/2} D \quad [1]$$

$$W_{rn} = 1800 G^2 D \quad [2]$$

where:

W_{sn} = reference nail withdrawal design value for smooth- or screw-shank carbon steel nails, lbf/in. penetration,

W_{rn} = reference nail withdrawal design value for ring-shank carbon steel nails, lbf/in. penetration,

G = equivalent specific gravity (ESG) listed in Table 4, and

D = nail diameter, in.

4.4.8.2 Reference Wood Screw Withdrawal Design Values

Reference withdrawal design values for wood screws from wood structural panels depend on a number of variables, including the type and diameter of the screw, the specific gravity of the wood component and the effective penetration of the threaded portion of the screw. Wood screw withdrawal tests are conducted in accordance with ASTM D1761. The ESG values for wood screw withdrawal strength with wood structural panels are listed in Table 4, which can be used to estimate the reference wood screw withdrawal design values from wood structural panels using Eq. 3. The calculated reference wood screw withdrawal design values are tabulated in Appendix A, Table A3, for ease of reference. The reference wood screw withdrawal design values shall be multiplied by all applicable adjustment factors (see Section 4.5 of this specification and Table 11.3.1 of the NDS) to obtain adjusted wood screw withdrawal design values.

$$W_{ws} = 2850 G^2 D \quad [3]$$

where:

W_{ws} = reference wood screw withdrawal design value, lbf/in. thread penetration,

G = equivalent specific gravity (ESG) listed in Table 4, and

D = wood screw diameter, in.

4.4.8.3 Reference Fastener Head Pull-Through Design Values

A nailed or screwed sheathing-to-wood-framing joint may be limited by either fastener withdrawal from the lumber or by fastener head pull-through strength of the wood structural panel. Reference fastener head pull-through design values are affected by many factors, such as the fastener head diameter, and the thickness and specific gravity of the wood structural panel. Testing for the fastener head pull-through strength is based on ASTM D1037. The ESG values for fastener head pull-through strength with wood structural panels are listed in Table 4, which can be used to estimate the reference fastener head pull-through design values from wood structural panels using Eq. 4 or 5. The calculated reference nailhead pull-through design values are tabulated in Appendix B, Table B1, for ease of reference. The reference fastener head pull-through design values shall be multiplied by all applicable adjustment factors (see Section 4.5 of this specification and Table 11.3.1 of the NDS) to obtain adjusted fastener head pull-through design values.

$$\text{When } t \leq 2.5 D_H \quad W_H = 690 \pi D_H G^2 t \quad [4]$$

$$\text{When } t > 2.5 D_H \quad W_H = 1725 \pi D_H^2 G^2 \quad [5]$$

where:

t = wood structural panel thickness, in.,

W_H = reference fastener head pull-through design value, lbf,

D_H = fastener round head diameter, in. ($0.234 \text{ in.} \leq D_H \leq 0.500 \text{ in.}$), and

G = equivalent specific gravity (ESG) listed in Table 4

4.4.8.4 Reference Fastener Lateral Design Values

Reference lateral design values for dowel-type fasteners installed on structural panels shall be calculated using the dowel bearing strength of wood structural panels provided in Table 5 and the yield limit equations provided in Table 12.3.1A of the NDS. Chapter 12 of the NDS provides the calculated reference fastener lateral load design values for various fastener types, configurations and framing and sheathing materials. The reference fastener lateral design values shall be multiplied by all applicable adjustment factors (see Section 4.5 of this specification and Table 11.3.1 of the NDS) to obtain adjusted fastener lateral design values.

TABLE 5

DOWEL BEARING STRENGTH (F_e) AND ESG FOR WOOD STRUCTURAL PANELS IN DRY SERVICE CONDITIONS

Dowel-type fastener (nails, screws and bolts) diameter (D)	Plywood								OSB			
	Group 1				Others ^a							
	^b		⊥ ^c		^b		⊥ ^c		^b		⊥ ^c	
	F _e (psi)	ESG	F _{e⊥} (psi)	ESG	F _e (psi)	ESG	F _{e⊥} (psi)	ESG	F _e (psi)	ESG	F _{e⊥} (psi)	ESG
D ≤ 1/4 in.	4,650	0.50	4,650	0.50	3,350	0.42	3,350	0.42	4,650	0.50	4,650	0.50
1/4 in. < D ≤ 1/2 in. ^d	5,600	0.50	3,150	0.50	4,700	0.42	2,450	0.42	3,700	0.33	3,700	0.56

a. Use when species of the plies is not known. When species of the plies is known, specific gravity of the actual species and the corresponding dowel bearing strength may be used, or the weighted average may be used for mixed species.

b. Dowel bearing strength when the dowel is loaded parallel to the strength axis of the panel.

c. Dowel bearing strength when the dowel is loaded perpendicular to the strength axis of the panel.

d. Refer to APA Technical Note: Fastener Loads for Plywood – Bolts, Form E825, for $1/2$ in. $< D \leq 3/4$ in. used with plywood.

4.5 Adjustments

Panel design capacities may be adjusted as required under the following provisions.

4.5.1 Duration of Load (DOL)

Design capacities listed are based on “normal duration of load” as traditionally used for solid wood in accordance with USDA Forest Products Laboratory Report R-1916, and successfully used for plywood for approximately 70 years. Adjustment factors for strength capacities (C_D) are shown in Table 6.

TABLE 6

LOAD DURATION FACTORS FOR WOOD STRUCTURAL PANELS

Time Under Load	DOL Adjustment Factor* (C_D)
Permanent	0.90
Normal	1.00
Two Months	1.15
Seven Days	1.25
Wind or Earthquake	1.60

*Adjustment for impact load does not apply to wood structural panels.

Creep

Wood structural panels under constant load will creep (deflection will increase) over time. For typical construction applications, panels are not normally under constant load and, accordingly, creep need not be considered in design. When panels will sustain permanent loads that will stress the product to one-half or more of its design strength capacity, allowance should be made for creep. Limited data indicates that under such conditions, creep may be taken into account in deflection calculations by applying the applicable creep adjustment factor (C_C) to panel bending stiffness, EI, as shown in Table 7.

TABLE 7

CREEP ADJUSTMENT FACTORS FOR WOOD STRUCTURAL PANELS

Creep Adjustment Factor (C_C) for Permanent Loads		
Moisture Content	Plywood	OSB
Dry ($< 16\%$ MC)	1/2	1/2
16% MC or greater	1/2	1/6

See Section 4.5.2 for additional adjustments related to service moisture conditions, which for EI is cumulative with the creep adjustment.

4.5.2 Service Moisture Conditions

Design capacities apply to panels under moisture conditions that are continuously dry in service; that is, where equilibrium moisture content is less than 16%. Adjustment factors for conditions where the panel moisture content in service is expected to be 16% or greater are shown in Table 8.

4.5.3 Elevated Temperature

Capacities in Tables 9 and 10 apply at temperatures of 70°F and lower. Wood structural panel parts of buildings should not be exposed to temperatures above 200°F for more than very brief periods. However, between 70°F and 200°F, adjustments to capacity generally do not need to be made, because the need for adjustment of dry capacities depends upon whether moisture content will remain in the 12 to 15% range or whether the panel will dry to lower moisture contents as a result of the increase in temperature. If drying occurs, as is usually the case, the increase in strength due to drying can offset the loss in strength due to elevated temperature. For instance, temperatures of up to 150°F or higher do occur under roof coverings of buildings on hot days, but they are accompanied by moisture content reductions which offset the strength loss so that high temperatures are not considered in the design of roof structures. To maintain a moisture content of 12% at 150°F, sustained relative humidity of around 80% would be required. The designer needs to exercise judgment in determining whether high temperature and moisture content occur simultaneously and the corresponding need for temperature adjustment of allowable design capacities.

4.5.4 Pressure Treatment

Preservative treatment

Capacities given in this specification apply, without adjustment, to plywood pressure-impregnated with preservative chemicals and redried in accordance with American Wood Protection Association (AWPA) Standard U1. OSB panels are currently recommended only for non-pressure applications of preservative treating in accordance with AWPA Standard T1.

TABLE 8

MOISTURE CONTENT ADJUSTMENT FACTORS FOR WOOD STRUCTURAL PANELS

Allowable Design Capacity	Moisture Content Adjustment Factor (C_M)
Strength (F_b , S , F_t , F_c , F_v , F_s [lb/Q], F_v [t_v])	0.75
Stiffness (EI, EA, G_v [t_v])	0.85
Bearing ($F_{c\perp}$, A) Plywood OSB	0.50 0.20
Nail withdrawal strength	0.75
Wood screw withdrawal strength and lateral strength for dowel-type fasteners (nails, screws and bolts) of 1/2 inch or less in diameter	NDS Table 11.3.3

Fire-retardant treatment

Discussion in this specification does not apply to fire-retardant-treated structural panels. However, some general information on fire-retardant treated plywood roof sheathing is available in *APA Technical Note: Fire-Retardant-Treated (FRT) Plywood*, Form K320. For fire-retardant-treated plywood, all capacities and end-use conditions shall be in accordance with the recommendations and model code evaluation reports of the company providing the treating and redrying service.

4.5.5 Panel Size

Strength capacities in bending and tension are appropriate for panels 24 inches or greater in width. For panels less than 24 inches in width used in applications where failure could result in life safety concerns, the panel size adjustment shall be made to the bending and tensile strength capacities (b is the panel width or dimension perpendicular to the applied stress, in inches):

$$b \geq 24 \text{ inches, } C_s = 1.00 \quad [6]$$

$$24 > b \geq 8 \text{ in., } C_s = 0.25 + 0.0313 b \quad [7]$$

$$b < 8 \text{ in., } C_s = 0.50 \quad [8]$$

Single strips less than 8 inches wide used in stressed applications shall be chosen such that they are relatively free of surface defects.

4.5.6 Panel Edge Support

For roof sheathing applications, additional panel edge support is recommended when panel widths are less than 24 inches. Specific recommendations are in *APA Technical Note: Panel Edge Support for Narrow-Width Roof Sheathing*, Form R275. Note that panel edge support may be required or recommended in applications subject to walking loads, such as floor or roof sheathing. See *APA Engineered Wood Construction Guide*, Form E30, for more information.

TABLE 9

PANEL DESIGN CAPACITIES

Span Rating	Stress Parallel to Strength Axis				Stress Perpendicular to Strength Axis			
	Plywood				Plywood			
	3-ply	4-ply	5-ply	OSB	3-ply	4-ply	5-ply	OSB
PANEL BENDING STIFFNESS, EI (lbf-in.²/ft of panel width)								
W24	NA	NA	NA	52,000	NA	NA	NA	11,000
24/0	66,000	66,000	66,000	60,000	3,600	7,900	11,000	11,000
24/16	86,000	86,000	86,000	78,000	5,200	11,500	16,000	16,000
32/16	125,000	125,000	125,000	115,000	8,100	18,000	25,000	25,000
40/20	250,000	250,000	250,000	200,000	18,000	39,500	56,000	56,000
48/24	NA	440,000	440,000	350,000	NA	65,000	91,500	91,500
16 oc	165,000	165,000	165,000	150,000	11,000	24,000	34,000	34,000
20 oc	230,000	230,000	230,000	195,000	13,000	28,500	40,500	40,500
24 oc	NA	330,000	330,000	300,000	NA	57,000	80,500	80,500
32 oc	NA	NA	715,000	650,000	NA	NA	235,000	235,000
48 oc	NA	NA	1,265,000	1,150,000	NA	NA	495,000	495,000
Structural I Multiplier								
	1.0	1.0	1.0	1.0	1.5	1.5	1.6	1.6
PANEL BENDING STRENGTH, F_bS (lbf-in./ft of panel width)								
W24	NA	NA	NA	300	NA	NA	NA	97
24/0	250	275	300	300	54	65	97	97
24/16	320	350	385	385	64	77	115	115
32/16	370	405	445	445	92	110	165	165
40/20	625	690	750	750	150	180	270	270
48/24	NA	930	1,000	1,000	NA	270	405	405
16 oc	415	455	500	500	100	120	180	180
20 oc	480	530	575	575	140	170	250	250
24 oc	NA	705	770	770	NA	260	385	385
32 oc	NA	NA	1,050	1,050	NA	NA	685	685
48 oc	NA	NA	1,900	1,900	NA	NA	1,200	1,200
Structural I Multiplier								
	1.0	1.0	1.0	1.0	1.3	1.4	1.5	1.5
PANEL AXIAL TENSION, F_A (lbf/ft of panel width)								
W24	NA	NA	NA	2,300	NA	NA	NA	780
24/0	2,300	2,300	3,000	2,300	600	600	780	780
24/16	2,600	2,600	3,400	2,600	990	990	1,300	1,300
32/16	2,800	2,800	3,650	2,800	1,250	1,250	1,650	1,650
40/20	2,900	2,900	3,750	2,900	1,600	1,600	2,100	2,100
48/24	NA	4,000	5,200	4,000	NA	1,950	2,550	2,550
16 oc	2,600	2,600	3,400	2,600	1,450	1,450	1,900	1,900
20 oc	2,900	2,900	3,750	2,900	1,600	1,600	2,100	2,100
24 oc	NA	3,350	4,350	3,350	NA	1,950	2,550	2,550
32 oc	NA	NA	5,200	4,000	NA	NA	3,250	3,250
48 oc	NA	NA	7,300	5,600	NA	NA	4,750	4,750
Structural I Multiplier								
	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Shaded cells are atypical APA panel constructions, as shown in Table 14.

Continued on next page

TABLE 9 (Continued)

PANEL DESIGN CAPACITIES

Span Rating	Stress Parallel to Strength Axis				Stress Perpendicular to Strength Axis			
	Plywood				Plywood			
	3-ply	4-ply	5-ply	OSB	3-ply	4-ply	5-ply	OSB
PANEL AXIAL COMPRESSION, F_A (lb/ft of panel width)								
W24	NA	NA	NA	2,850	NA	NA	NA	2,500
24/0	2,850	4,300	4,300	2,850	2,500	3,750	3,750	2,500
24/16	3,250	4,900	4,900	3,250	2,500	3,750	3,750	2,500
32/16	3,550	5,350	5,350	3,550	3,100	4,650	4,650	3,100
40/20	4,200	6,300	6,300	4,200	4,000	6,000	6,000	4,000
48/24	NA	7,500	7,500	5,000	NA	7,200	7,200	4,300
16 oc	4,000	6,000	6,000	4,000	3,600	5,400	5,400	3,600
20 oc	4,200	6,300	6,300	4,200	4,000	6,000	6,000	4,000
24 oc	NA	7,500	7,500	5,000	NA	7,200	7,200	4,300
32 oc	NA	NA	9,450	6,300	NA	NA	9,300	6,200
48 oc	NA	NA	12,150	8,100	NA	NA	10,800	6,750
Structural I Multiplier								
	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
PANEL AXIAL STIFFNESS, E_A (lb/ft of panel width)								
W24	NA	NA	NA	3,350,000	NA	NA	NA	2,000,000
24/0	3,350,000	3,350,000	3,350,000	3,350,000	2,900,000	2,900,000	2,900,000	2,500,000 ^a
24/16	3,800,000	3,800,000	3,800,000	3,800,000	2,900,000	2,900,000	2,900,000	2,700,000 ^a
32/16	4,150,000	4,150,000	4,150,000	4,150,000	3,600,000	3,600,000	3,600,000	2,700,000 ^a
40/20	5,000,000	5,000,000	5,000,000	5,000,000	4,500,000	4,500,000	4,500,000	2,900,000 ^b
48/24	NA	5,850,000	5,850,000	5,850,000	NA	5,000,000	5,000,000	3,300,000 ^b
16 oc	4,500,000	4,500,000	4,500,000	4,500,000	4,200,000	4,200,000	4,200,000	2,700,000 ^a
20 oc	5,000,000	5,000,000	5,000,000	5,000,000	4,500,000	4,500,000	4,500,000	2,900,000 ^b
24 oc	NA	5,850,000	5,850,000	5,850,000	NA	5,000,000	5,000,000	3,300,000 ^b
32 oc	NA	NA	7,500,000	7,500,000	NA	NA	7,300,000	4,200,000 ^b
48 oc	NA	NA	8,200,000	8,200,000	NA	NA	7,300,000	4,600,000
Structural I Multiplier								
	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
PANEL SHEAR IN THE PLANE, F_s (lb/Q) (lb/ft of panel width)								
W24	NA	NA	NA	130	NA	NA	NA	130
24/0	155	155	170	130	275	375	130	130
24/16	180	180	195	150	315	435	150	150
32/16	200	200	215	165	345	480	165	165
40/20	245	245	265	205	430	595	205	205
48/24	NA	300	325	250	NA	725	250	250
16 oc	245	245	265	205	430	595	205	205
20 oc	245	245	265	205	430	595	205	205
24 oc	NA	300	325	250	NA	725	250	250
32 oc	NA	NA	390	300	NA	NA	300	300
48 oc	NA	NA	500	385	NA	NA	385	385
Structural I Multiplier								
	1.4	1.4	1.4	1.0	1.4	1.4	1.0	1.0

Shaded cells are atypical APA panel constructions, as shown in Table 14.

- a. The value shall be permitted to be increased to 2,900,000 lb/ft for the calculation of the bending stiffness (EI_{joist}) of prefabricated wood I-joists.
- b. The value shall be permitted to be increased to 4,500,000 lb/ft for the calculation of the composite floor bending stiffness ($EI_{composite}$) of prefabricated wood I-joists.

Continued on next page

TABLE 9 (Continued)

PANEL DESIGN CAPACITIES

Span Rating	Stress Parallel to Strength Axis				Stress Perpendicular to Strength Axis			
	Plywood			OSB	Plywood			OSB
	3-ply	4-ply	5-ply		3-ply	4-ply	5-ply	
PANEL RIGIDITY THROUGH THE THICKNESS, G_{v_v} (lbf/in. of panel depth)								
W24	NA	NA	NA	69,000	NA	NA	NA	69,000
24/0	25,000	32,500	37,500	77,500	25,000	32,500	37,500	77,500
24/16	27,000	35,000	40,500	83,500	27,000	35,000	40,500	83,500
32/16	27,000	35,000	40,500	83,500	27,000	35,000	40,500	83,500
40/20	28,500	37,000	43,000	88,500	28,500	37,000	43,000	88,500
48/24	NA	40,500	46,500	96,000	NA	40,500	46,500	96,000
16 oc	27,000	35,000	40,500	83,500	27,000	35,000	40,500	83,500
20 oc	28,000	36,500	42,000	87,000	28,000	36,500	42,000	87,000
24 oc	NA	39,000	45,000	93,000	NA	39,000	45,000	93,000
32 oc	NA	NA	54,000	110,000	NA	NA	54,000	110,000
48 oc	NA	NA	76,000	155,000	NA	NA	76,000	155,000
Structural I Multiplier								
	1.3	1.3	1.1	1.0	1.3	1.3	1.1	1.0
PANEL SHEAR THROUGH THE THICKNESS, F_{v_v} (lbf/in. of shear-resisting panel length)								
W24	NA	NA	NA	145	NA	NA	NA	145
24/0	53	69	80	155	53	69	80	155
24/16	57	74	86	165	57	74	86	165
32/16	62	81	93	180	62	81	93	180
40/20	68	88	100	195	68	88	100	195
48/24	NA	98	115	220	NA	98	115	220
16 oc	58	75	87	170	58	75	87	170
20 oc	67	87	100	195	67	87	100	195
24 oc	NA	96	110	215	NA	96	110	215
32 oc	NA	NA	120	230	NA	NA	120	230
48 oc	NA	NA	160	305	NA	NA	160	305
Structural I Multiplier								
	1.3	1.3	1.1	1.0	1.3	1.3	1.1	1.0

Shaded cells are atypical APA panel constructions, as shown in Table 14.

TABLE 10

SANDED GROUP 1^a PLYWOOD DESIGN CAPACITIES

Performance Category	Stress Parallel to Strength Axis			Stress Perpendicular to Strength Axis		
	A-A, A-C	Marine	Other	A-A, A-C	Marine	Other
PANEL BENDING STIFFNESS, EI (lb-in.²/ft of panel width)						
1/4	15,000	15,000	15,000	700	980	700
11/32	34,000	34,000	34,000	1,750	2,450	1,750
3/8	49,000	49,000	49,000	2,750	3,850	2,750
15/32	120,000	120,000	120,000	11,000	15,500	11,000
1/2	140,000	140,000	140,000	15,500	21,500	15,500
19/32	205,000	205,000	205,000	37,500	52,500	37,500
5/8	230,000	230,000	230,000	48,500	68,000	48,500
23/32	320,000	320,000	320,000	90,500	125,000	90,500
3/4	355,000	355,000	355,000	115,000	160,000	115,000
7/8	500,000	500,000	500,000	185,000	260,000	185,000
1	760,000	760,000	760,000	330,000	460,000	330,000
1-1/8	985,000	985,000	985,000	490,000	685,000	490,000
Structural I Multiplier						
	1.0	1.0	1.0	1.4	1.0	1.4
PANEL BENDING STRENGTH, F_bS (lb-in./ft of panel width)						
1/4	115	105	95	17	20	14
11/32	185	170	155	31	36	26
3/8	245	225	205	44	52	37
15/32	425	390	355	130	150	110
1/2	470	430	390	175	205	145
19/32	625	570	520	270	315	225
5/8	670	615	560	325	380	270
23/32	775	710	645	455	530	380
3/4	815	750	680	565	660	470
7/8	1,000	935	850	780	910	650
1	1,300	1,200	1,100	1,150	1,350	975
1-1/8	1,600	1,500	1,350	1,500	1,750	1,250
Structural I Multiplier						
	1.0	1.0	1.1	1.4	1.0	1.4
PANEL AXIAL TENSION, F_tA (lb/ft of panel width)						
1/4	1,800	1,650	1,650	660	990	550
11/32	1,800	1,650	1,650	840	1,250	700
3/8	2,350	2,150	2,150	1,250	1,900	1,050
15/32	3,500	3,200	3,200	2,400	3,600	2,000
1/2	3,500	3,200	3,200	2,450	3,700	2,050
19/32	4,400	4,000	4,000	2,750	4,150	2,300
5/8	4,500	4,100	4,100	3,000	4,500	2,500
23/32	5,100	4,650	4,650	3,400	5,150	2,850
3/4	5,250	4,750	4,750	4,150	6,200	3,450
7/8	5,350	4,850	4,850	5,200	7,850	4,350
1	6,750	6,150	6,150	6,250	9,350	5,200
1-1/8	7,000	6,350	6,350	6,300	9,450	5,250
Structural I Multiplier						
	1.0	1.0	1.0	1.7	1.0	1.8

a. See Table 11 for multipliers for other species Groups

TABLE 10 (Continued)

SANDED GROUP 1^a PLYWOOD DESIGN CAPACITIES

Performance Category	Stress Parallel to Strength Axis			Stress Perpendicular to Strength Axis		
	A-A, A-C	Marine	Other	A-A, A-C	Marine	Other
PANEL AXIAL COMPRESSION F_c (lbf/ft of panel width)						
1/4	1,710	1,550	1,550	605	990	550
11/32	1,710	1,550	1,550	715	1,150	650
3/8	2,200	2,000	2,000	1,050	1,700	950
15/32	3,300	3,000	3,000	2,050	3,350	1,850
1/2	3,300	3,000	3,000	2,100	3,400	1,900
19/32	4,150	3,750	3,750	2,350	3,850	2,150
5/8	4,200	3,800	3,800	2,600	4,250	2,350
23/32	4,800	4,350	4,350	2,900	4,750	2,650
3/4	4,900	4,450	4,450	3,500	5,750	3,200
7/8	5,000	4,550	4,550	4,500	7,400	4,100
1	6,350	5,750	5,750	5,350	8,750	4,850
1-1/8	6,550	5,950	5,950	5,400	8,800	4,900
Structural I Multiplier						
	1.0	1.0	1.0	1.8	1.0	1.8
PANEL AXIAL STIFFNESS, $E A$ (lbf/ft of panel width)						
1/4	1,800,000	1,800,000	1,800,000	625,000	1,150,000	625,000
11/32	1,800,000	1,800,000	1,800,000	750,000	1,350,000	750,000
3/8	2,350,000	2,350,000	2,350,000	1,150,000	2,050,000	1,150,000
15/32	3,500,000	3,500,000	3,500,000	2,150,000	3,850,000	2,150,000
1/2	3,500,000	3,500,000	3,500,000	2,250,000	4,050,000	2,250,000
19/32	4,350,000	4,350,000	4,350,000	2,500,000	4,500,000	2,500,000
5/8	4,450,000	4,450,000	4,450,000	2,750,000	4,950,000	2,750,000
23/32	5,100,000	5,100,000	5,100,000	3,150,000	5,650,000	3,150,000
3/4	5,200,000	5,200,000	5,200,000	3,750,000	6,750,000	3,750,000
7/8	5,300,000	5,300,000	5,300,000	4,750,000	8,550,000	4,750,000
1	6,700,000	6,700,000	6,700,000	5,700,000	10,500,000	5,700,000
1-1/8	6,950,000	6,950,000	6,950,000	5,700,000	10,500,000	5,700,000
Structural I Multiplier						
	1.0	1.0	1.0	1.8	1.0	1.8
PANEL SHEAR IN THE PLANE, F_s (lb/Q) (lbf/ft of panel width)						
1/4	105	135	105	105	135	105
11/32	145	190	145	145	190	145
3/8	165	215	165	165	215	165
15/32	220	285	220	220	285	220
1/2	235	305	235	235	305	235
19/32	290	375	290	290	375	290
5/8	310	405	310	310	405	310
23/32	350	455	350	350	455	350
3/4	360	470	360	360	470	360
7/8	425	555	425	425	555	425
1	470	610	470	470	610	470
1-1/8	525	685	525	525	685	525
Structural I Multiplier						
	1.3	1.0	1.3	1.4	1.0	1.4

a. See Table 11 for multipliers for other species Groups

TABLE 10 (Continued)

SANDED GROUP 1^a PLYWOOD DESIGN CAPACITIES

Performance Category	Stress Parallel to Strength Axis			Stress Perpendicular to Strength Axis		
	A-A, A-C	Marine	Other	A-A, A-C	Marine	Other
PANEL RIGIDITY THROUGH THE THICKNESS $G_v t_v$ (lbf/in. of panel depth)						
1/4	24,000	31,000	24,000	24,000	31,000	24,000
11/32	25,500	33,000	25,500	25,500	33,000	25,500
3/8	26,000	34,000	26,000	26,000	34,000	26,000
15/32	38,000	49,500	38,000	38,000	49,500	38,000
1/2	38,500	50,000	38,500	38,500	50,000	38,500
19/32	49,000	63,500	49,000	49,000	63,500	49,000
5/8	49,500	64,500	49,500	49,500	64,500	49,500
23/32	50,500	65,500	50,500	50,500	65,500	50,500
3/4	51,000	66,500	51,000	51,000	66,500	51,000
7/8	52,500	68,500	52,500	52,500	68,500	52,500
1	73,500	95,500	73,500	73,500	95,500	73,500
1-1/8	75,000	97,500	75,000	75,000	97,500	75,000
Structural I Multiplier						
	1.3	1.0	1.3	1.3	1.0	1.3
PANEL SHEAR THROUGH THE THICKNESS, $F_v t_v$ (lbf/in. of shear-resisting panel length)						
1/4	51	66	51	51	66	51
11/32	54	70	54	54	70	54
3/8	55	72	55	55	72	55
15/32	80	105	80	80	105	80
1/2	81	105	81	81	105	81
19/32	105	135	105	105	135	105
5/8	105	135	105	105	135	105
23/32	105	135	105	105	135	105
3/4	110	145	110	110	145	110
7/8	110	145	110	110	145	110
1	155	200	155	155	200	155
1-1/8	160	210	160	160	210	160
Structural I Multiplier						
	1.3	1.0	1.3	1.3	1.0	1.3

a. See Table 11 for multipliers for other species Groups.

4.6 Section Properties

Where required, geometric cross-sectional properties may be calculated by assuming a uniform rectangular cross section in conjunction with typical Performance Categories for a given span rating, as shown in Table 12. Computed rectangular (geometric) properties on a per-foot-of-panel-width basis are provided in Table 13.

4.7 Uniform Load Computations

Computation of uniform load capacity of wood structural panels shall be as outlined in this section for such applications as roofs, floors and walls. The design capacities are subject to adjustment as specified in Section 4.5.

4.7.1 Number of Spans

Three basic span conditions are presented for computing uniform load capacities of wood structural panels. For normal framing practice and a standard panel size (4 x 8 ft), the following assumptions are recommended for computing load-span tables:

When the panel strength axis is across (perpendicular to) the supports, the three-span condition is assumed for support spacing up to and including 32 inches. The two-span condition is assumed for support spacing greater than 32 inches.

When the panel strength axis is placed parallel to the supports, the three-span condition is assumed for support spacing up to and including 16 inches, the two-span condition is assumed when the support spacing is greater than 16 inches up to 24 inches, and a single span is assumed for spans greater than 24 inches.

TABLE 11

MULTIPLIERS FOR SANDED GROUP 2, 3 AND 4 PLYWOOD DESIGN CAPACITIES

Species Group	A-A, A-C	Marine	Other
PANEL BENDING STIFFNESS, EI (lb-in.²/ft of panel width)			
2	0.83	NA	0.83
3	0.67	NA	0.67
4	0.56	NA	0.56
PANEL BENDING STRENGTH, $F_b S$ (lb-in./ft of panel width)			
2	0.70	NA	0.73
3	0.70	NA	0.73
4	0.67	NA	0.67
PANEL AXIAL TENSION, $F_t A$ (lb/ft of panel width)			
2	0.70	NA	0.73
3	0.70	NA	0.73
4	0.67	NA	0.67
PANEL AXIAL COMPRESSION, $F_c A$ (lb/ft of panel width)			
2	0.73	NA	0.71
3	0.65	NA	0.64
4	0.61	NA	0.62
PANEL AXIAL STIFFNESS, EA (lb/ft of panel width)			
2	0.83	NA	0.83
3	0.67	NA	0.67
4	0.56	NA	0.56
PANEL SHEAR IN THE PLANE, $F_v (lb/Q)$ (lb/ft of panel width)			
2	1.00	NA	1.00
3	1.00	NA	1.00
4	1.00	NA	1.00
PANEL RIGIDITY THROUGH THE THICKNESS, $G_v t_v$ (lb/in. of panel depth)			
2	0.83	NA	0.83
3	0.67	NA	0.67
4	0.56	NA	0.56
PANEL SHEAR THROUGH THE THICKNESS, $F_v t_v$ (lb/in. of shear-resisting panel length)			
2	0.74	NA	0.74
3	0.74	NA	0.74
4	0.68	NA	0.68

TABLE 12

PERFORMANCE CATEGORY AND NOMINAL THICKNESS (in.) BY SPAN RATING(The predominant Performance Category for each span rating is highlighted in **bold** type.)

Span Rating	Performance Category										
	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	7/8	1	1-1/8
Sheathing											
W24	0.375	0.437	0.469								
24/0	0.375	0.437	0.469	0.500							
24/16		0.437	0.469	0.500							
32/16			0.469	0.500	0.594	0.625					
40/20					0.594	0.625	0.719	0.750			
48/24							0.719	0.750	0.875		
Single Floor											
16 oc					0.594	0.625					
20 oc					0.594	0.625					
24 oc							0.719	0.750			
32 oc									0.875	1.000	
48 oc											1.125

TABLE 13

PANEL SECTION PROPERTIES^a

Performance Category	Approximate Weight ^b (psf)		Nominal Thickness † (in.)	Area A (in. ² /ft)	Moment of Inertia I (in. ⁴ /ft)	Section Modulus S (in. ³ /ft)	Statical Moment Q (in. ³ /ft)	Shear Constant lb/Q (in. ² /ft)
	Plywood	OSB						
3/8	1.1	1.2	0.375	4.500	0.053	0.281	0.211	3.000
7/16	1.3	1.4	0.437	5.250	0.084	0.383	0.287	3.500
15/32	1.4	1.5	0.469	5.625	0.103	0.440	0.330	3.750
1/2	1.5	1.7	0.500	6.000	0.125	0.500	0.375	4.000
19/32	1.8	2.0	0.594	7.125	0.209	0.705	0.529	4.750
5/8	1.9	2.1	0.625	7.500	0.244	0.781	0.586	5.000
23/32	2.2	2.4	0.719	8.625	0.371	1.033	0.775	5.750
3/4	2.3	2.5	0.750	9.000	0.422	1.125	0.844	6.000
7/8	2.6	2.9	0.875	10.500	0.670	1.531	1.148	7.000
1	3.0	3.3	1.000	12.000	1.000	2.000	1.500	8.000
1-1/8	3.4	3.7	1.125	13.500	1.424	2.531	1.898	9.000

See Section 6 for conversion factors.

a. Properties based on rectangular cross section of 1-ft width.

b. Approximate weight for calculating actual dead loads of the panel.

4.7.2 Support Widths

To include the effects of support width in deflection and shear strength calculations, two-inch-nominal lumber framing is assumed for support spacings less than 48 inches. Four-inch-nominal lumber framing is assumed for support spacing of 48 inches or greater.

4.7.3 Other Considerations

The equations presented in this section are standard beam formulas altered to accept the mixed units noted. These formulas are provided for computing uniform loads on wood structural panels over conventional lumber framing. Because it is assumed that no blocking is used, the formulas are for one-way “beam” action, rather than two-way “plate” action. The resulting loads are assumed to be applied to full-sized panels in standard sheathing-type applications. Loads are for the panels only, and in no way account for the design of the framing supports. Further consideration shall be given to concentrated loads, in compliance with local building codes and with maximum span recommendations of APA.

4.7.4 Loads Based on Bending Strength

The following formulas shall be used for computing loads based on design bending strength capacity ($F_b S$).

For a single span:

$$w_b = \frac{96 F_b S}{\ell_1^2} \quad [9]$$

For a two-span condition:

$$w_b = \frac{96 F_b S}{\ell_1^2} \quad [10]$$

For a three-span condition:

$$w_b = \frac{120 F_b S}{\ell_1^2} \quad [11]$$

where:

w_b = uniform load based on bending strength, psf,

$F_b S$ = design bending strength capacity, lbf-in./ft, and

ℓ_1 = span (center to center of supports), in.

4.7.5 Uniform Loads Based on Shear Strength

The following formulas shall be used for computing loads based on design shear strength capacity (F_s [lb/Q]).

For a single span:

$$w_s = \frac{24 F_s (\text{lb/Q})}{\ell_2} \quad [12]$$

For a two-span condition:

$$w_s = \frac{19.2 F_s (\text{lb/Q})}{\ell_2} \quad [13]$$

For a three-span condition:

$$w_s = \frac{20 F_s (\text{lb/Q})}{\ell_2} \quad [14]$$

where:

w_s = uniform load based on shear strength, psf,

F_s (lb/Q) = design shear strength capacity, lbf/ft, and

ℓ_2 = clear span (center to center of supports minus support width), in.

4.7.6 Uniform Loads Based on Deflection Requirements

The following formulas shall be used for computing deflection under uniform load, or allowable loads based on deflection requirements.

For a single span:

$$\Delta = \frac{w \ell_3^4}{921.6 EI} \quad [15]$$

For a two-span condition:

$$\Delta = \frac{w \ell_3^4}{2220 EI} \quad [16]$$

For a three-span condition:

$$\Delta = \frac{w \ell_3^4}{1743 EI} \quad [17]$$

where:

Δ = deflection, in.,

w = uniform load, psf,

EI = design apparent bending stiffness (see Section 4.4.1), lbf-in.²/ft,

ℓ_3 = clear span + SW, in., and

SW = support-width factor, equal to 0.25 inch for two-inch-nominal lumber framing and 0.625 inch for four-inch-nominal lumber framing (see Section 4.7.2 for support widths).

4.7.7 Allowable Uniform Load Based on the Allowable Deflection Requirement

For allowable uniform load based on a deflection requirement, compute bending deflection (Δ) with a uniform load (w) equal to one psf. The allowable uniform load based on the allowable deflection is then computed as:

$$w_d = \frac{\Delta_{all.}}{\Delta} \quad [18]$$

where:

w_d = uniform load based on the allowable deflection, psf, and

$\Delta_{all.}$ = allowable deflection, in.

4.8 Design Examples Showing Use of Capacity Tables

Note: In these examples, panel type and construction are selected for illustrative purposes. Normally specification is by grade and span rating, irrespective of the panel type, and calculations should assume the lowest capacities applicable to available types and constructions as given in Table 14 for the specified span rating.

TABLE 14

TYPICAL APA PANEL CONSTRUCTIONS^a

Span Rating	Plywood			OSB
	3-ply	4-ply	5-ply ^b	
Sheathing				
W24				X
24/0	X			X
24/16				X
32/16	X	X	X	X
40/20	X	X	X	X
48/24		X	X	X
Single floor				
16 oc				
20 oc		X	X	X
24 oc		X	X	X
32 oc			X	X
48 oc			X	X

a. Constructions listed may not be available in every area. Check with suppliers concerning availability.

b. Applies to plywood with 5 or more layers.

4.8.1 Example 1 — Conventional Roof

A 4-ply plywood panel trademarked Single Floor 24 oc with tongue-and-groove edges was inadvertently installed over 4-in.-nominal roof supports 48 in. on center. The long dimension (strength axis) of the panel was placed perpendicular to supports. The local building code requires that the panel support a 25-psf snow load.

Bending Strength

From Table 9, a 4-ply plywood Single Floor 24 oc panel with stress applied parallel to the strength axis (long panel dimension perpendicular to supports) has a bending strength capacity ($F_b S$) of 705 lbf-in./ft. This capacity is adjusted by a duration-of-load factor (C_D) of 1.15 (see Section 4.5.1). This duration-of-load factor is normally associated with snow loads for roof structures. From Eq. 10, a two-span condition is assumed.

$$w_b = \frac{96 F_b S}{\ell_1^2} = \frac{96 (705 \times 1.15)}{48^2} = 34 \text{ psf}$$

Shear Strength in the Plane

From Table 9, a 4-ply plywood Single Floor 24 oc panel with stress applied parallel to the strength axis has shear strength in the plane (F_s [lb/Q]) of 300 lbf/ft. This capacity is adjusted by a duration-of-load factor (C_D) of 1.15 (see Section 4.5.1). From Eq. 13,

$$w_s = \frac{19.2 F_s (\text{lb/Q})}{\ell_2} = \frac{19.2 (300 \times 1.15)}{(48 - 3.5)} = 149 \text{ psf}$$

Bending Stiffness

From Table 9, a Single Floor 24 oc panel with stress applied parallel to the strength axis has a dry stiffness capacity (EI) of 330,000 lbf-in.²/ft. The deflection limit for live load is $\ell/240$. From Eq. 16,

$$\Delta = \frac{w \ell_3^4}{2220 EI} = \frac{1.0 (48 - 3.5 + 0.625)^4}{2220 \times 330000} = 5.66 \times 10^{-3} \text{ in.}$$

From Eq. 18,

$$w_d = \frac{\Delta_{\text{all.}}}{\Delta} = \frac{48/240}{5.66 \times 10^{-3}} = 35 \text{ psf}$$

Bending strength controls (provides the lowest capacity) for this application. The bending strength capacity of 34 psf represents total load, from which dead load is subtracted to arrive at live load capacity. The bending stiffness capacity of 35 psf represents live load only. Here, if dead load (panel weight plus roofing) is no more than 9 psf, the 25 psf snow load capacity is achieved. The tongue-and-groove edges provide required edge supports.

4.8.2 Example 2 — Panelized Roof

An OSB panel trademarked as Structural I Sheathing 32/16 is to be used in a panelized roof system over 2-in.-nominal framing members 24 in. on center under roof snow load. The long panel dimension (strength axis) of the panel will be placed parallel to supports.

Bending Strength

From Table 9, an OSB Sheathing 32/16 panel with stress applied perpendicular to strength axis (long panel dimension parallel to supports) has a bending strength capacity ($F_b S$) equal to 165 lbf-in./ft. This capacity is adjusted by a multiplier of 1.5 for OSB Structural I, and by a duration-of-load factor (C_D) of 1.15 (see Section 4.5.1). From Eq. 10, a two-span condition is assumed.

$$w_b = \frac{96 F_b S}{\ell_1^2} = \frac{96 (165 \times 1.5 \times 1.15)}{24^2} = 47 \text{ psf}$$

Shear Strength in the Plane

From Table 9, an OSB Sheathing 32/16 panel with stress applied perpendicular to strength axis has shear strength in the plane (F_s [lb/Q]) of 165 lbf/ft. This capacity is adjusted by a multiplier of 1.0 for OSB Structural I, and by a duration-of-load factor (C_D) of 1.15 (see Section 4.5.1). From Eq. 13,

$$w_s = \frac{19.2 F_s (\text{lb/Q})}{\ell_2} = \frac{19.2 (165 \times 1.0 \times 1.15)}{(24 - 1.5)} = 162 \text{ psf}$$

Bending Stiffness

From Table 9, an OSB Sheathing 32/16 panel with stress applied perpendicular to strength axis has a dry stiffness capacity (EI) of 25,000 lbf-in.²/ft. This capacity is adjusted by a multiplier of 1.6 for OSB Structural I. The deflection limit for live load is $\ell/240$. From Eq. 16,

$$\Delta = \frac{w \ell_3^4}{2220 EI} = \frac{1.0 (24 - 1.5 + 0.25)^4}{2220 \times (25000 \times 1.6)} = 3.02 \times 10^{-3} \text{ in.}$$

From Eq. 18,

$$w_d = \frac{\Delta_{\text{all.}}}{\Delta} = \frac{24/240}{3.02 \times 10^{-3}} = 33 \text{ psf}$$

4.8.3 Example 3 — Floor

A 5-ply plywood panel trademarked as Single Floor 24 oc is to be used in a floor system over supports 24 inches on center. The panels will be placed with the long panel dimension (strength axis) perpendicular to supports. Supports are 2-in.-nominal framing members. The capacity of the panel will be computed based on bending strength, shear strength in the plane and bending stiffness.

Bending Strength

From Table 9, a 5-ply plywood Single Floor 24 oc panel with stress applied parallel to the strength axis (long panel dimension perpendicular to supports) has a bending strength capacity ($F_b S$) of 770 lbf-in./ft. From Eq. 11, a three-span condition is assumed.

$$w_b = \frac{120 F_b S}{\ell_1^2} = \frac{120 \times 770}{24^2} = 160 \text{ psf}$$

Shear Strength in the Plane

From Table 9, a 5-ply plywood Single Floor 24 oc panel with stress applied parallel to the strength axis has shear strength in the plane (F_s [lb/Q]) equal to 325 lbf/ft. From Eq. 14,

$$w_s = \frac{20 F_s (\text{lb/Q})}{\ell_2} = \frac{20 \times 325}{(24 - 1.5)} = 289 \text{ psf}$$

Bending Stiffness

From Table 9, a 5-ply plywood Single Floor 24 oc panel with stress applied parallel to the strength axis has a dry stiffness capacity (EI) of 330,000 lbf-in.²/ft. The deflection limit for live load is $\ell/360$. From Eq. 17,

$$\Delta = \frac{w \ell_3^4}{1743 EI} = \frac{1.0 (24 - 1.5 + 0.25)^4}{1743 \times 330000} = 4.66 \times 10^{-4} \text{ in.}$$

From Eq. 18,

$$w_d = \frac{\Delta_{\text{all.}}}{\Delta} = \frac{24/360}{4.66 \times 10^{-4}} = 143 \text{ psf}$$

While the above calculations would indicate that this Single Floor construction has a live load capacity of 143 psf (limited by bending stiffness), it is important to note that some structural panel applications are not controlled by uniform load. Residential floors, commonly designed for 40-psf live load, are a good example. The calculated allowable load is greatly in excess of the typical design load. This excess does not mean that floor spans for Single Floor panels can be increased, but only that there is considerable reserve strength and stiffness for uniform loads. Recommended maximum spans for wood structural panel floors are based on deflection under concentrated loads, how the floor “feels” to passing foot traffic and other subjective factors which relate to user acceptance. Always check the maximum floor and roof spans for wood structural panels before making a final selection for these applications.

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6. CONVERSION FACTORS

Quantity	Multiply	By	To Obtain
Pressure/Stress	psi	6.895	kPa
	psi	6,895	N/m ²
Length	in.	25.40	mm
Panel Bending Stiffness Capacity (EI)	lbf-in. ² /ft of panel width	9.415	N-m ² /m
Panel Bending Strength Capacity (F _b S)	lbf-in./ft of panel width	0.3707	N-m/m
Panel Axial Tension Capacity (F _t A)	lbf/ft of panel width	14.59	N/m
Panel Axial Compression (F _c A)	lbf/ft of panel width	14.59	N/m
Panel Axial Stiffness Capacity (EA)	lbf/ft of panel width	14.59	N/m
Panel Shear in the Plane Capacity (F _s [lb/Q])	lbf/ft of panel depth	14.59	N/m
Panel Rigidity Through the Thickness (G _v T _v)	lbf/in. of panel length	1.216	N/m
Approximate Weight of Panel	psf	4.88	kg/m ²
Panel Area (A)	in. ² /ft of panel width	2.117	10 ³ mm ² /m
Panel Section Modulus (S)	in. ³ /ft of panel width	53.76	10 ³ mm ³ /m
Panel Moment of Inertia (I)	in. ⁴ /ft of panel width	1.366	10 ⁶ mm ⁴ /m
Temperature	°F	t°C = (t°F – 32)/1.8	°C

APPENDIX A. Calculated Reference Withdrawal Design Values for Nails and Wood Screws

TABLE A1

SMOOTH- OR SCREW-SHANK CARBON STEEL NAIL REFERENCE WITHDRAWAL DESIGN VALUES (W_{sn}) IN NORMAL (10-YEAR) LOAD DURATION ($C_D = 1.0$)

Sheathing ESG	Nail Dia., D (in.)	Nail Type			W_{sn} (lbf) for Sheathing Thickness, t (in.) of											
		Common	Box	Sinker	5/16	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	1	1 1/8	1 1/4
0.40 (Plywood and OSB)	0.092			6d	4	5	6	6	6	8	8	9	10	13	14	16
	0.099		6d & 7d	7d	4	5	6	6	7	8	9	10	10	14	16	17
	0.113	6d & 7d	8d	8d	5	6	7	7	8	9	10	11	12	16	18	20
	0.120			10d	5	6	7	8	8	10	10	12	13	17	19	21
	0.128		10d & 12d		6	7	8	8	9	11	11	13	13	18	20	22
	0.131	8d			6	7	8	9	9	11	11	13	14	18	21	23
	0.135		16d	12d	6	7	8	9	9	11	12	14	14	19	21	24
	0.148	10d & 12d	20d & 30d	16d	6	8	9	10	10	12	13	15	16	21	23	26
	0.162	16d	40d		7	8	10	11	11	13	14	16	17	23	25	28
	0.177			20d	8	9	11	12	12	15	15	18	19	25	28	31
	0.192	20d		30d	8	10	12	13	13	16	17	19	20	27	30	34
	0.207	30d		40d	9	11	13	14	14	17	18	21	22	29	33	36
	0.225	40d			10	12	14	15	16	19	20	23	24	31	35	39
	0.244	50d		60d	11	13	15	16	17	20	21	24	26	34	38	43
	0.263	60d			11	14	16	17	18	22	23	26	28	37	41	46

TABLE A2

RING-SHANK CARBON STEEL NAIL REFERENCE WITHDRAWAL DESIGN VALUES (W_{rn}) IN NORMAL (10-YEAR) LOAD DURATION ($C_D = 1.0$)

Sheathing ESG	Nail Dia., D (in.)	Nail Type			W_{rn} (lbf) for Sheathing Thickness, t (in.) of											
		Common	Box	Cooler	5/16	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	1	1 1/8	1 1/4
0.56 (Plywood and OSB)	0.092			6d	16	19	23	24	26	31	32	37	39	52	58	65
	0.099		6d		17	21	24	26	28	33	35	40	42	56	63	70
	0.113	6d	8d	8d	20	24	28	30	32	38	40	46	48	64	72	80
	0.120			10d	21	25	30	32	34	40	42	49	51	68	76	85
	0.131	8d			23	28	32	35	37	44	46	53	55	74	83	92
	0.135		16d		24	29	33	36	38	45	48	55	57	76	86	95
	0.148	10d & 12d			26	31	37	39	42	50	52	60	63	84	94	104
	0.162	16d			29	34	40	43	46	54	57	66	69	91	103	114

TABLE A3

CARBON STEEL WOOD SCREWS REFERENCE WITHDRAWAL DESIGN VALUES^a (W_{ws}) IN NORMAL (10-YEAR) LOAD DURATION ($C_D = 1.0$)

Sheathing ESG	Screw Dia., D (in.)	Screw Number	W_{ws} (lbf) for Sheathing Thickness, t (in.) of											
			5/16	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	1	1 1/8	1 1/4
0.45 (Plywood and OSB)	0.138	6	25	30	35	37	40	47	50	57	60	80	90	100
	0.151	7	27	33	38	41	44	52	54	63	65	87	98	109
	0.164	8	30	35	41	44	47	56	59	68	71	95	106	118
	0.177	9	32	38	45	48	51	61	64	73	77	102	115	128
	0.190	10	34	41	48	51	55	65	69	79	82	110	123	137
	0.216	12	39	47	55	58	62	74	78	90	93	125	140	156
	0.242	14	44	52	61	65	70	83	87	100	105	140	157	175
	0.268	16	48	58	68	73	77	92	97	111	116	155	174	193
	0.294	18	53	64	74	80	85	101	106	122	127	170	191	212
	0.320	20	58	69	81	87	92	110	115	133	139	185	208	231
	0.372	24	67	81	94	101	107	127	134	154	161	215	242	268

a. The tabulated values assume a full-penetration of the wood screw thread into the sheathing thickness.

APPENDIX B. Calculated Reference Nail-Head Pull-Through Design Values

TABLE B1

REFERENCE NAIL-HEAD PULL-THROUGH DESIGN VALUES (W_H) IN NORMAL (10-YEAR) LOAD DURATION ($C_D = 1.0$)

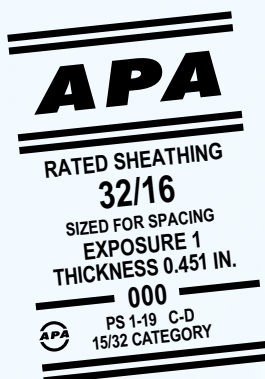
Sheathing ESG	Nail Head Dia., D_H (in.)	Nail Type			W_H (lbf) for Sheathing Thickness, t (in.) of											
		Common	Box	Sinker	5/16	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	1	1 1/8	1 1/4
0.50 (Group 1 Plywood and OSB)	0.234			6d	40	48	55	59	63	74	74	74	74	74	74	74
	0.250			7d	42	51	59	64	68	80	85	85	85	85	85	85
	0.266	6d & 7d	6d & 7d	8d	45	54	63	68	72	86	90	96	96	96	96	96
	0.281	8d		10d	48	57	67	71	76	90	95	107	107	107	107	107
	0.297		8d		50	60	70	75	80	96	101	116	120	120	120	120
	0.312	10d & 12d	10d & 12d	12d	53	63	74	79	85	100	106	122	127	132	132	132
	0.344	16d	16d	16d	58	70	82	87	93	111	117	134	140	160	160	160
	0.375		20d & 30d	20d	64	76	89	95	102	121	127	146	152	191	191	191
	0.406	20d		30d	69	83	96	103	110	131	138	158	165	220	223	223
	0.438	30d	40d	40d	74	89	104	111	119	141	148	171	178	237	260	260
	0.469	40d			79	95	111	119	127	151	159	183	191	254	286	298
	0.500	50d		60d	85	102	119	127	135	161	169	195	203	271	305	339
0.42 (Other Plywood Groups) ^a	0.234			6d	28	34	39	42	45	52	52	52	52	52	52	52
	0.250			7d	30	36	42	45	48	57	60	60	60	60	60	60
	0.266	6d & 7d	6d & 7d	8d	32	38	44	48	51	60	64	68	68	68	68	68
	0.281	8d		10d	34	40	47	50	54	64	67	75	75	75	75	75
	0.297		8d		35	43	50	53	57	67	71	82	84	84	84	84
	0.312	10d & 12d	10d & 12d	12d	37	45	52	56	60	71	75	86	89	93	93	93
	0.344	16d	16d	16d	41	49	58	62	66	78	82	95	99	113	113	113
	0.375		20d & 30d	20d	45	54	63	67	72	85	90	103	108	134	134	134
	0.406	20d		30d	49	58	68	73	78	92	97	112	116	155	158	158
	0.438	30d	40d	40d	52	63	73	79	84	99	105	120	126	167	183	183
	0.469	40d			56	67	78	84	90	106	112	129	135	179	202	210
	0.500	50d		60d	60	72	84	90	96	114	119	137	143	191	215	239

a. When species of the plies is not known. When species of the plies is known, specific gravity listed for the actual species may be used or the weighted average may be used for mixed species, and the nail head pull-through design values may be calculated in accordance with Eq. 4 or 5, as applicable.

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