

1. Introduction

Decks and balconies are a way of expanding living areas into the outdoors for many households. There are over 30 million residential decks in the US (Shook et al., 2001). Over 85% of new single-family detached homes are being built with decks, patios, porches or balconies and, each year, 4.2% of homeowners add a deck to their existing home (Shook et al., 2001). Overall, 46% of households have built decks onto existing homes as do-it-yourself projects (Shook et al., 2001).

A deck is defined in building codes as “an exterior floor projecting from and supported by an adjacent structure, posts, piers, or other independent structures” (BOCA, 1996). A balcony is “an exterior floor projecting from and supported by a structure without additional independent supports” (BOCA, 1996). Usually, each is surrounded by guardrails: “A system of building components located near the open sides of elevated walking surfaces for the purpose of minimizing the possibility of an accidental fall from the walking surface to a lower level” (BOCA, 1996).

1.1 Justification

Reported deck failures are becoming more common. Failures can be prevented with a proper full inspection of the structure and proper maintenance. They happen without warning, especially when fastener failure is the cause, and when injuries are most likely to occur, such as when a large number of people gathers on the deck or balcony (Bohnhoff, 2002; Cushman, 2002). In many

cases, the failures have resulted in lawsuits being filed for Homeowner Negligence (Bohnhoff, 2002).

Construction requirements for decks and balconies vary by locality. All building codes cover certain requirements, but the enforcement of these requirements varies widely from locality to locality. Few building codes give detailed guidance for the building of decks or balconies (Cushman, 2002). North Carolina has added an appendix to their building code on deck construction, but the typical building code gives only design loads, some decay prevention guidelines, and the standards for railings and stairways.

With the increased failures, the recommended methods for deck connection have changed. Publications from 1972 and 2001 detail the connection to a house without removing siding, providing flashing, or checking for the material that the deck is attached to (Anderson, 1972; Cory, 2001). Fairfax County, Virginia, developed deck details for new decks in the late 1980's or early 1990's. They are given to homeowners interested in adding a deck to their home. The information covers everything from footings to attachment to house to railing details (Fairfax County, 1998). They developed the deck details, not because of failures, but because the deck permits are the most requested type of building permits. The county began giving out deck details to show the proper building techniques to the inexperienced builders (Foley, 2002). After construction is complete, a building inspector is sent to the site for a full inspection. Since they became available in Fairfax County, other localities such as Ocean City, MD and Manitoba, Canada have adopted similar regulations.

These regulations and inspections for the areas previously mentioned are only for new construction. The average deck is upgraded, expanded, repaired, or replaced after 11 years (Shook et al., 2001). Building codes call for safe construction and keeping the structure in sound condition (BOCA, 1978), but without proper maintenance and inspection, there is no way to know what condition the deck is in.

2. Objectives

1. To devise a methodology for residential wood deck and balcony inspection for conventional dimension lumber (2x_ material) construction that is accessible to a building inspector or consultant.
2. To complete a draft manual for possible publication that will contain methods for wood deck and balcony inspection.
3. To collect information that can be subsequently used to prepare articles for appropriate magazines and trade journals to increase awareness about the potential problem of wood deck and balcony failure.

3. Literature Review

3.1 Issues that need to be addressed

There are several resources for inspecting wood structures, both protected and exposed to the environment. The following is a compilation of several issues and facts that need to be considered during the inspection of wood decks or balconies.

3.1.1 Installation

A deck or balcony inspector should not assume that a deck or balcony was built following the applicable local building code. Therefore, it is necessary to check the structure's conformance to the building code at the time it was built. However, if repairs are required, the 1996 BOCA National Building Code requires that any inadequate members must "be made to conform to the requirements for new structures" (BOCA, 1996)

Identifying the wood species, grade and preservatives used in construction is essential for determining the strength properties and decay resistance of the material. Lumber treated with CCA was introduced in early 1970s (Dannenberg, 1988). By 1989, 80% of residential decks in US were pressure treated (Shook et al., 2001). Grade stamps (species and grade) and quality marks (preservatives) provide this information on new material.

3.1.2 Moisture Content

After drying, the moisture level in wood comes into equilibrium with the environment. This moisture content is known as the Equilibrium Moisture Content (EMC). The EMC of wood is dependent on relative humidity and temperature of the environment and fluctuates while the wood is in service (Forest Products Laboratory, 1999). The three common methods used in determining the moisture content of wood are the oven-dry method, the electrical method and the electromagnetic method.

3.1.2.1 *Oven-Drying*

The oven-dry method is mostly used in a laboratory setting, it requires cutting and weighing the wood (Forest Products Laboratory, 1999). The sample is cut and weighed, then dried for a period of time. It is then weighed a second time. The following equation is used to find moisture content on a dry-basis (Forest Products Laboratory, 1999):

$$MC(\%) = \frac{\text{Weight when cut} - \text{Ovendry weight}}{\text{Ovendry weight}} \times 100$$

3.1.2.2 *Electrical*

The electrical method is a faster way to measure moisture content. It is based on the relationships between moisture content and several electrical properties of wood: electrical conductivity, resistivity, dielectric constant and the powerloss factor (Forest Products Laboratory, 1999). The meters depend on calibration curves to find the moisture content so the correlations between these

properties and moisture content are not perfect (ASTM, 1992). Measurements are limited to moisture contents from about 6% to the fiber saturation point (approximately 30%), but can be accurate to 1% when used correctly in appropriate ranges (Forest Products Laboratory, 1999). Readings must be made in defect free areas to lower the variation due to natural defects in the wood (ASTM, 1992).

Conductance type meters are electrical moisture meters that use needles (electrodes) driven into the wood to determine moisture content (Forest Products Laboratory, 1999). Their reliability ranges from 6 to 27% moisture content. The moisture content is based on the ionic conductance between the two electrodes. Ionic conductance is influenced by moisture content, wood variables, environment, probe geometry, and meter design, but is independent of specific gravity. Also, readings are affected by the possibility of wet pockets in the wood and preservative treatments. Using an insulated electrode driven to below the penetration depth of the preservative avoids this problem (ASTM, 1992). Electrodes are both insulated and uninsulated. The uninsulated electrodes will sense the highest moisture content between the electrodes, usually at the surface (Forest Products Laboratory, 1999). The insulated type will sense the moisture content only at the driven depth. Lumber has a moisture gradient, meaning the moisture content is not uniform throughout the entire cross section. To find the average value, readings may be taken at several depths. To find the average moisture content using a single point, the electrodes should be driven to one-quarter to one-fifth of the thickness in a rectangular section ($1/6$ to $1/7$ the

diameter of circular section) (ASTM, 1992; Forest Products Laboratory, 1999). Readings should be taken so that the current flow is parallel to the grain and read as soon as the pins are driven in, as the readings tend to drift toward a lower moisture content (ASTM, 1992).

Dielectric meters are divided into two types: capacitance and power loss. Both utilize surface contact electrodes with an electric field that penetrates the wood (Forest Products Laboratory, 1999). Capacitance meters (also known as admittance meters) are based on the dielectric constant and power loss meters are based on the power loss factor or the resistance of material (Forest Products Laboratory, 1999; ASTM, 1992). They are reliable from 0% moisture content to the fiber saturation point. Readings are influenced by moisture gradient, electric field penetration, specific gravity, material thickness, surface condition/roughness, and contact pressure. Because an electric field is used, an air gap is required below the sample to prevent interference of another material (ASTM, 1992). Also, the electric field decreases with depth, so the reading is biased to the surface moisture content (Forest Products Laboratory, 1999; ASTM, 1992). Problems arise with warped members; readings must be taken on opposite sides and the highest reading is used. When preservative treatment is present, the “reading should be considered qualitative or semi-quantitative at best” (ASTM, 1992).

Electric moisture meters are factory calibrated for a certain species and temperature. The readings must be corrected as described in the manufacturer’s instructions (Forest Products Laboratory, 1999). According to ASTM standard

D4444-92, the temperature correction should be made before species correction. Differences may occur between the two types of meters, even in the same conditions. Also, heartwood and sapwood could give different readings, and should not be mixed in one reading (ASTM, 1992).

3.1.2.3 *Electromagnetic*

The third type of moisture meters is the electromagnetic meter. According to a product brochure, the electromagnetic moisture meters are not affected by temperature. It uses a three dimensional electric field to find an average MC over the entire area. This technology is accurate for 5 to 30% moisture contents (Wagner, 1993).

3.1.3 Weathering

Weathering of wood is caused by light, water and heat. It causes raised grain, loose grain, checks, pulling away from fasteners, color changes, dirt and mildew, splinters, and fragments that separate from the surface. Water causes rapid changes in moisture content, which, in turn, creates an internal moisture gradient that causes warping, cup, checks and raised grain. Sunlight causes color changes. Heat causes little change as compared to the effects of light and water. The physical changes caused by weathering are weakening of surface cell walls and erosion, which occurs at the rate of about 1/4 in. per century (Freas, 1982; Verrall et al., 1980). Weathering has little effect on strength properties, such as modulus of elasticity, modulus of rigidity and compressive strength. Toughness is slightly affected by thermal effects (Freas, 1982).

3.1.4 Fungal Decay

In untreated and non-decay resistant wood species, any wood can become unsafe in as little as three years, (Gaby et al., 1978). Decay fungi is natural in the environment. If conditions are favorable, it penetrates wood and can be established in a few weeks (Esllyn et al., 1979). The fungus permeates the wood in strands (Forest Products Laboratory, 1999) and uses enzymes to degrade cellulose (Verrall et al., 1980). As fungus penetrates wood, the porosity increases, increasing the ability of the wood to hold water. As the amount of fungal decay increases, the rate of deterioration also increases. (Esllyn et al., 1979)

Once decay is visible, it has reached an advanced stage; incipient, early and intermediate decay surround the visible area, reaching 6 to 12 inches in the grain. In some cases, the decay is severe below a thin layer of intact wood (Esllyn et al., 1979). Extreme visual evidence of decay are growths of the decay fungi on the surface of the wood. Mycelium is a light colored papery growth on the surface (DeBonis, 1999). Fruiting bodies are mushroom like structures that develop in early or late stages, depending on the species of the fungi. They may be outside, or deep within, wood and usually appear out of direct sun exposure in moist areas with high local humidity. The bodies are white to dark brown in color, and darken with age, unless they are eaten by animals (Esllyn et al., 1979).

3.1.4.1 *Types of Decay*

A specific threat to softwood decks is brown rot (McDonald et al., 1996). In its incipient stage, there is initial colonization and release of enzymes, but no visual damage. The early stage marks slight color and texture changes. In the intermediate stage, there is an obvious change in color and texture but the structure of wood is still intact. By the advanced stage, the wood has a brown color, it crumbles when touched, the surface has a cubical appearance, and the cell structure is affected (Clausen et al., 2001). Brown rot attacks the cellulose, which makes up approximately 50% of wood by weight and is a major component of the cell wall (Forest Products Laboratory, 1999). Eslyn gave a somewhat vague estimation of the strength loss caused by brown rot: 50 to 70% with just a 3% weight loss (Eslyn et al., 1979).

White rot fungi are common in hardwoods. It attacks both the lignin, the glue that holds the wood cells together, and cellulose. With white rot, the wood loses color, becomes whiter, and has a spongy feeling (Forest Products Laboratory, 1999).

Soft rot is caused by mold fungi. It severely degrades wood, but is very shallow. Immediately below the rot zone, the wood is in fine condition. Soft rot causes damage to thin pieces that are alternately wet and dry, like deck boards, and is common in weathered wood (Forest Products Laboratory, 1999).

Decay is still a threat to dry wood in some cases. Water conducting fungi has strands, called rhizomorphs, that carry water from the soil to lumber that would normally be dry (Forest Products Laboratory, 1999; Verrall et al., 1980).

Molds and stains are less of a threat than rot fungi. They cannot breakdown cellulose, therefore do not reduce strength properties, and feed on the starches in sugars stored in sapwood cells (Verrall et al., 1980). Molds and stains appear as specks, spots, streaks, or patches on the wood surface. Colors range from blue to blue-black, gray to brown, yellow, orange, purple and red, depending on the organism and moisture content. They follow the rays in wood and on a cross section, show up as pie-shaped. The effects may be deep reaching, even if the color comes off easily, and molds reach deeper than stains (Forest Products Laboratory, 1999). Molds and stains have effects on shock resistance, toughness, absorbency, and their presence indicates the presence of incipient decay (Verrall et al., 1980; Forest Products Laboratory, 1999).

3.1.4.2 *Conditions*

Decay fungi need certain conditions to grow. They are air, temperature, water, and food. The air requirement is oxygen, but very little is needed, especially when the fungi is in a dormant stage (Eslyn et al., 1979). For optimum growth, the fungus prefers temperatures between 70 and 90°F. It grows slowly at temperatures below 50°F and above 90°F (Verrall et al., 1980) and dies at temperatures above 100°F (Eslyn et al., 1979). When it is below freezing, the fungi will become dormant (Eslyn et al., 1979; Verrall et al., 1980; Forest Products Laboratory, 1999).

The Climate Index Map, Figure 3.1, is a resource to determine the amount of decay protection needed in an area. With a higher threat of decay, more

protection is needed. The map shows areas with climate indexes less than 35, which have little threat, area that are greater than 35 and less than 70, which have a moderate threat, and areas listed above 70, which indicates a severe threat of decay (Forest Products Laboratory, 1999).

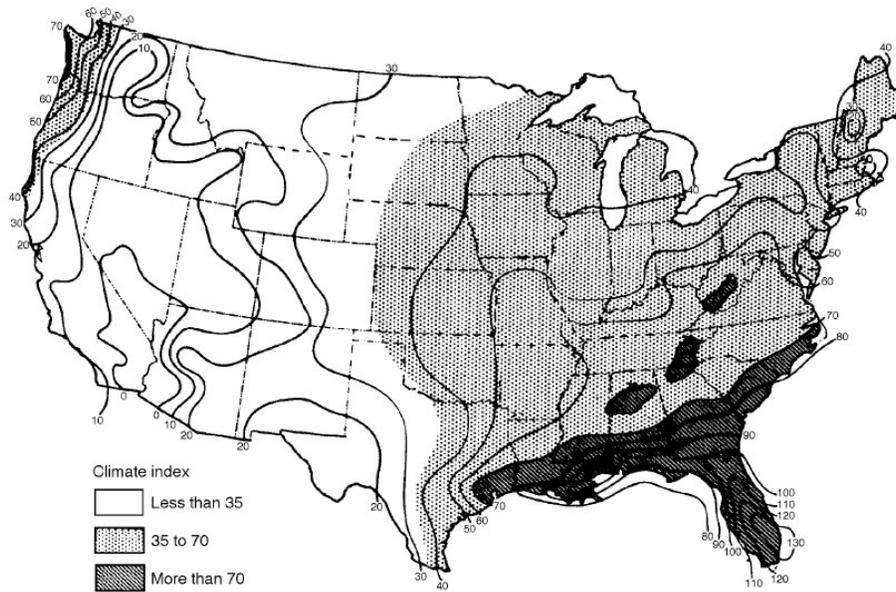


Figure 3.1. The climate index is a measure of the decay threat. Higher numbers indicate more decay protection is needed (Forest Products Laboratory, 1999).

The source of food for fungus is the wood. Decay attacks sapwood and non decay resistant heartwood (Eslyn et al., 1979). Destructive decay fungi, such as brown and white rot, feed on the cellulose and lignin while nondestructive forms rely on stored food within the cell (Forest Products Laboratory, 1999). Decay fungi need free water to grow, and therefore require moisture contents above the fiber saturation point at some locations (Eslyn et al., 1979; Forest Products Laboratory, 1999). Water vapor in humidity (condensation) is not usually enough for substantial decay (Forest Products Laboratory, 1999). Decay fungi is only affected by MC in contact with the fungal growth area and since the moisture content changes over time, fungal decay may be present in areas that are currently dry (Eslyn et al., 1979).

Many locations in wood construction are conducive to fungal growth. Some of the more common ones are fastener holes, joints, horizontal checks,

end grain, and points of soil contact. Also important are areas with plant growth, the presence of moss or vegetation means there have been periods of sustained wetting (Eslyn et al., 1979).

3.1.4.3 *Decay Resistance*

Resistance to decay can be natural or induced. Natural resistance varies within the same species and even within the same tree. Some species, such as cedar and old growth redwood, have naturally decay resistant heartwood (Forest Products Laboratory, 1999), however, wood exposed to the weather often loses its resistance due to leaching of the fungitoxic compounds (Eslyn et al., 1979). Man-made preservative treatments include, but are not limited to, Chromated Copper Arsenate, waterborne (CCA), Ammoniacal copper quaternary ammonium chloride, waterborne (ACQ) and Ammoniacal copper zinc arsenate, waterborne (ACZA) (McDonald et al., 1996). “Factory-applied” preservatives treat to specific retentions and penetration depths, creating a treated shell around the core (McDonald et al., 1996). This penetration may be as little as 1/4 in. (Eslyn et al., 1979). The American Wood-Preserver’s Association standard C15-02 is a table of minimum retentions of preservatives for commercial and residential construction (AWPA, 2002). Quality marks are required for the treating process, and are the only way to be assured of the depth of penetration and retention without coring into the wood. Field applied preservatives are for cuts and drilled holes; the penetration is not as deep as the factory-applied preservatives (McDonald et al., 1996).

3.1.4.4 *Strength Properties*

Fungal decay reduces the strength properties of wood. Work to maximum load, toughness, and impact bending are most sensitive to early decay (Clausen et al., 2001). Toughness is the “energy required to cause rapid complete failure in a centrally loaded bending specimen,” in other words, the ability to withstand impacts. With a 1% weight loss, the loss of toughness can range from 6 to 50%. With a 10% weight loss, greater than 50% of the toughness is lost. Static bending is second, followed by the reduction of all strength properties. Once decay is visible, it is safe to assume that there has been considerable loss of strength (Forest Products Laboratory, 1999). A summary of several researchers’ work was presented by Wilcox (1978) and is shown in Table 3.1.

Table 3.1. Summary of strength losses caused by brown-rot fungi in softwoods from a review of literature by Wilcox (1978). Losses are expressed as a percentage of the expected value in non-decayed wood.

Strength Property	Weight Loss	Strength Loss
Toughness	1 - 2%	50- 55%
	8 - 10%	60-85%
Work to Maximum Load	2%	27%
MOE	2%	4 - 55%
	6%	66%
MOR	2%	13 - 50%
	6%	61%
Bending	2 - 5%	5 - 16%
	9%	36%
Compression Perpendicular-to-grain	2%	18%
	8%	48%
Compression Parallel-to-grain	2%	10%
	9%	42%
Tension Parallel-to-grain	1 - 2%	23 - 40%
	5 - 8%	50 - 60%
Shear Parallel-to-grain	1%	2%
	7%	13%

3.1.5 Decay Identification

Identifying decay in the early stages is difficult. Nondestructive evaluation detects voids and discontinuities, which are natural in wood (Ross et al., 1991).

The challenge is relating the results to the strength properties.

3.1.5.1 *Physical Methods*

The visual method includes locating areas of localized depressions, staining, insects, plant growth and fruiting bodies. As stated above, once decay

becomes visual, it is safe to assume that the strength is greatly reduced (Ross et al., 1999).

Sounding is a way to locate severe decay that began on wood utility poles. The wood is hit with a hammer and a dull or hollow sound indicates decay (Eslyn et al., 1979). Locating the decay is based on experience, and is difficult to do without training. Sounding is only reliable with the wood is less than 4" thick (Ross et al., 1999) and boring/coring is required for verification and finding the extent of decay (Eslyn et al., 1979).

Boring and coring into the wood is used to find the limits of degradation. Core samples are intact and depth of decay is visible (Eslyn et al., 1979), as well as the preservatives (Ross et al., 1999). In boring, the samples are shavings, so the amount of resistance and quality of the shavings are the important factors (Ross et al., 1999). Sharp tools must be used, as they will not crush wood fibers. Common defects that may be mistaken for decay are resin pockets, shakes, abnormal grain and knots. When boring or coring a sample, it is recommended that penetration should be parallel to the fasteners and the holes created should be treated with preservatives (Eslyn et al., 1979).

The use of probes for checking splinters, also known as the "Pick Test", is based on toughness (Wilcox, 1983; Anderson et al., 2002). The presence of decay near the surface is based on breakage pattern of the splinter and comparing the softness and resistance to sound wood (Eslyn et al., 1979; Ross et al., 1999). The pick test has been proven to detect decay at as little as 5 to 10% weight loss (Wilcox, 1983). Using a moderately pointed tool (Eslyn et al.,

1979), the surface of the wood is penetrated into the latewood parallel-to-the-grain and a splinter is pried out (Wilcox, 1983). A brash break occurring directly over or near the tool with few splinters indicates decay. The decay also causes the break to be in a line across the grain (Ross et al., 1999; Wilcox, 1983; Verrall et al., 1980). Non-decayed wood will fail in either of two ways: a splintered break or fibrous failure. The splintering break occurs directly over the tool with numerous splinters. A fibrous failure has long splinters that separate far from the tool. The “pick test” is a subjective test and the results are influenced by operator experience, grain angle, and amount of latewood (Wilcox, 1983).

The Pilodyn test is quantitative and involves a spring loaded hardened steel pin driven into the wood. The depth of penetration is correlated with the amount of degradation (Ross et al., 1991). An investigation by Squirrell and Clarke (1987) used the Pilodyn instrument to test the quality of waterlogged timbers. Penetration depths of the Pilodyn instrument were compared to penetration depths into sound wood at a similar moisture content.

Other tests are screw withdrawal, static bending and compression. Static bending tests measure modulus of elasticity using a load-deflection relationship. It is a difficult test to conduct in the field, because it is hard to maintain constant boundary conditions (Ross et al., 1991)

3.1.5.2 *Electrical Methods*

Shigometers may only be used when the wood is at or above fiber saturation point. A small hole is drilled and the shigometer measures the resistance to a pulsed current. Lower resistance to the current indicates

increased cation concentrations. The cation concentrations increase with fungal decay and fungal stains (Eslyn et al., 1979).

Use of an electric moisture meter, as described earlier, identifies areas with conditions conducive to decay. X-rays are used to detect voids in wood that may be caused by decay (Ross et al., 1991).

3.1.5.3 *Acoustic Methods*

Acoustic Emissions uses the “elastic energy that is spontaneously released by materials undergoing deformation” (Lemaster et al., 1997). A piezoelectric sensor converts the energy to an electric signal. This method is used to detect termites more than it is used for fungal decay (Lemaster et al., 1997).

Transverse vibrations analyze the behavior of a vibrating beam. This is another test that is not used in the field due to the difficulty in maintaining boundary conditions (Ross et al., 1991).

Stress waves are used to detect areas of decay by their speed and dissipation properties. Impact and stopping devices are linked to a timer and time for the wave to pass through the wood and reach sensors is displayed (Ross et al., 1999). The speed of the wave is constant through the wood; dissipation of energy is determined using particle movement (Ross et al., 1991). Areas with decay have slower wave speeds, and times greater than expected values are indicative of decay. Speeds are faster along the grain; slower speeds perpendicular-to-the-grain are due to the density changes between latewood and earlywood (Lanius et al., 1981) however, the perpendicular to grain transmission

times are noticeably increased with decay (Ross et al., 1999). A study by Emerson et al. (2002) found that wave velocity decreased 50% in areas of moderate to severe decay, but was not a “poor indicator of incipient or early decay”. Voids will not transmit waves (Ross et al., 1999), so the stress wave will ignore holes and partial failures i.e., splits. The times are also used to calculate the dynamic modulus of elasticity (Lanius et al., 1981) and compressive strength parallel-to-grain. It is not a useful technique for detecting termites, they attack mostly early wood and the stress waves will still be transmitted by the latewood (Ross et al., 1997).

3.1.5.4 Laboratory Methods

The immunodiagnostic methods target a hemicellulase enzyme that is released while brown rot is breaking down hemicellulose during incipient stages. Sawdust from the wood is placed into a solution which gives a positive or negative response. Weathered wood has contaminants that may give false positives (Clausen et al., 2001).

Serology, like the immunodiagnostic methods, is the science that deals with properties and reactions of serums. The serums are derived from animal antibodies. There are two types of seriological methods. Fluorescent antibody microscopy (FA) is not quantitative and difficult to analyze results because of cross-reactions. Enzyme Linked Immunosorbent Assay (ELISA) is quantitative and accepted (Goodell et al., 1988).

Agglutination Assay has been deemed promising for building inspectors. It is a rapid method, using latex particles that are covered in antibodies that react

with decay, the antigen. Agglutination occurs within 30 seconds and can recognize brown rot at 0% weight loss (Clausen, 1997).

Compression perpendicular-to-grain is another method that has been mentioned in the literature, but not written about in enough detail to report here.

3.1.6 Chemical Decay

Enzymatic oxidation occurs when chemicals in wood react with air and make a new dark-colored chemical that appears as a brown stain in sapwood. Iron stain is the interaction of iron with tannins in wood. It mostly occurs in hardwoods, but has been seen in Douglas-fir (Forest Products Laboratory, 1999).

3.1.7 Bacteria

Bacteria require free water to grow (Freas, 1982). It is present when the wood has been wet for a long time (Forest Products Laboratory, 1999). Bacteria increase the permeability of wood, increasing the possibility of decay (Freas, 1982)

3.1.8 Insects

At least 73 insect species exist that damage wood and they cause about \$2 billion in damage per year. The banning of certain pesticides may have increased termite damage by 30% (Lemaster et al., 1997).

3.1.8.1 *Beetles*

Beetles, such as roundheaded and flatheaded borers and powderpost beetles prefer fresh cut wood, seasoned lumber, and rustic structures, attacking both hardwood and softwoods (Forest Products Laboratory, 1999). The wood they infest has moisture contents between 15 and 20% (Forest Products Laboratory, 1999; McDonald et al., 1996). Fine powder falls from the wood when the adult makes a hole for the egg. The larvae will develop and emerge, even if the wood has been painted or varnished, leaving holes in the surface (Forest Products Laboratory, 1999).

3.1.8.2 *Old house borers*

Old house borers prefer seasoned, coniferous wood. They attack sapwood, and when they emerge, up to 10 to 15 years later, leave an oval hole 1/4 in. in diameter and are known to emit a ticking sound while inside the wood (Forest Products Laboratory, 1999).

3.1.8.3 *Termites*

Subterranean termites need wood that is in close proximity to the ground. The colony lives in the ground, and uses the wood for their food. These termites build earthen tubes from the ground to the wood and will tunnel right below the surface of the wood, preventing them from being found. Their only indications are the tunnels (mud tubes) and piles of wings left from swarming (Forest Products Laboratory, 1999).

Nonsubterranean termites move with infested items (Forest Products Laboratory, 1999). There are three types: Dry wood termites, which can infest wood at moisture contents as low as 13%, Formosan termites, which are larger and more aggressive than subterranean termites, and dampwood termites which require very wet wood (McDonald et al., 1996).

3.1.8.4 *Carpenter ants*

Carpenter ants are black or brown and are larger than regular ants. For them, the wood is shelter, not food, so they prefer soft wood or wood that has been softened by decay. They keep neat nests and debris accumulates beneath them (Forest Products Laboratory, 1999).

3.1.8.5 *Carpenter bees*

Carpenter bees look like large bumblebees with shiny abdomens. They prefer wood that is not exposed to direct sunlight. The bees make large tunnels into softwoods, treated or untreated (Forest Products Laboratory, 1999).

3.1.9 Connection to house

Residential decks can fail because of an inadequate connection to the house. These failures are caused by decay, improper fastening or failure of the house framing (McDonald et al., 1996). According to DeBlois (1996), the spacing of the fasteners at the connection to the house is a major problem when decks collapse. With improper flashing, water may enter around connections and expose the untreated wood within the house (Roberston, 2002). Figure 3.2 gives

a conceptual detail for connecting the deck ledger to the band joist as recommended by Anderson et al. (2003). In this connection, the reaction force of the deck joist is transmitted directly to the band joist by lag screws acting in shear. Bright aluminum flashing has been shown to corrode within five years when in contact with CCA treated lumber, therefore no longer preventing water from entering the structure (Roberston, 2002).

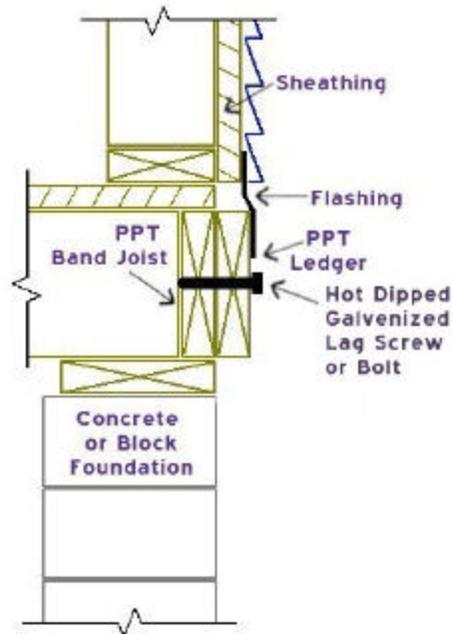


Figure 3.2. A deck detail of the deck-to-house connection shows a lag screw supporting the gravity load from the deck.

Bolts or lag screws are required for the connection, not nails (McDonald et al., 1996). DeBlois (1996) discussed the spacing required when using bolts, lag screws, or nails at the connection to the house, but recommended bolts or lag screws over nails. Self-supporting decks are required by one building department in Virginia when the quality of the connection cannot be verified in inspection. However, if the deck is an exit required by code, it must be attached to the house in some way because of the possibility of a seismic event (IRC, 2000). Failure of the house framing is often due to engineered floor systems, which do not have a perimeter band. For this case, the deck structure may be attached to only plywood or OSB sheathing. With Trus-Joist products, the band joist shown in their literature is engineered 1-1/4 in. Timberstrand rimboard (Robertson, 2002).

3.1.10 Support and Bracing

If the deck is not attached to the house, lateral support or bracing is required. The recommended sizes of brace members are 2x4 if less than 8 ft long and 2x6 when greater than 8 ft long. They must be attached to the deck with at least 3/8 in. bolts (McDonald et al., 1996).

Another alternative is attaching the deck to the house foundation as lateral support. The posts near the house are attached with 5/8 in. galvanized thru bolts or threaded rods, which penetrate the concrete or reinforced masonry foundation (Anderson et al, 2003).

3.1.11 Fasteners

3.1.11.1 *Adequacy*

Connections are just as important as proper sizing of members. They are subject to shrink/swell, rusting and chemical reactions. Normally, nails, screws, lag screws, bolts, metal straps and hangers are used in decks (Falk et al., 1993). Where nails are acceptable, the recommendation is a hot dipped galvanized with hardness range of 32 to 39 and an ultimate tensile strength of 1000 to 1210 MPa (CSA standard 086), (Baker, 1980). In preservative treated wood, stainless steel AISI 304 and 316, copper, or silicon bronze are the least corrodible materials (Baker, 1988; DeGroot et al., 1981). Hot dipped galvanized and spiral shanked nails resist rust and have high holding capacities (Dannenberg, 1988). The wet service factor, C_M , is applied to all connections (except toe-nails in withdrawal) that are exposed to wet service conditions. Wet service conditions are defined

when the moisture content will be above 19% for extended periods of time. The factor is 0.25 for smooth shanked nails in withdrawal (AF&PA, 1997).

The end distance and edge distance (distinguished by Figure 11G in the NDS-01) have minimums that are described in the NDS-01, Section 11.5.1. The distances are dependant on the penetration and diameter of the fastener (AF & PA, 2001).

3.1.11.2 Materials

Stainless steel is resistant to corrosion. Nails made from stainless steel are expensive, but they are more durable. They have less staining around the fastener than common nails. AISI grades 302,303, 304 and 316 are appropriate for outdoor use (Falk et al., 1993).

Copper is not normally used (Falk et al., 1993).

Aluminum is adequate in untreated wood and wood treated with oil-type preservatives. However, it will corrode rapidly with copper and waterborne preservatives (Falk et al., 1993).

3.1.11.3 Nails

For the NDS-01 requirements to be met, all nails used in the design equations must be manufactured with the specifications of ASTM F1667.

The delayed holding power of nails is more important than immediate holding power. The free water in wood cells or large amounts of water in the cell wall causes nails to lose holding power (Stern, 1952). Smooth shanked nails loose withdrawal resistance with wetting and drying cycles, causing them to pop-

up. The NDS wet service factor accounts for wood shrinkage. Deformed shank nails with spiral or annular grooves have better initial withdrawal resistance (Falk et al., 1993). In general, details that put nails in withdrawal should be avoided (Hoyle et al., 1989). Also, the deformed shanks have been shown to increase their holding power as wood dries, when driven into green lumber (Stern, 1952).

3.1.11.4 Screws

Wood screws and multipurpose screws are good because they can be easily withdrawn for replacement and can be used to flatten boards that cup or twist (Falk et al., 1993). Wood screws used with the NDS-01 design equations must be manufactured according to ANSI/ASME B18.6.1.

3.1.11.5 Lag Screws and Bolts

Lag screws are typically used to fasten one member to a thicker member when through bolts are not possible (Falk et al., 1993). Lag screws and bolts used in the NDS-01 design equations must be manufactured according to the specifications in ANSI/ASME B18.2.1. Tables in the NDS-01 give design values for lag screws, but are based on the assumption that the non-threaded portion is in the shear plane. When the threads are located in the shear plane, the strength reduction can reach 20% (McLain, 1992). The NDS-01 requires washers for bolts but does not mention washers as required for lag screws. Washers with lag screws are accounted for in the lateral load design equations and have been recommended for good practice (AF&PA, 1997; McDonald et al., 1996). At least half the length of the lag screw must penetrate thicker member (Falk et al.,

1993). The penetration of the lag screw must be at least 4D (four diameters) according to the NDS-01. Research described in McLain (1992) found that penetrations of 7D (dense woods) and 11D (less dense woods) are required for the connection to reach full strength.

When lag screw and bolts are installed, the holes must be predrilled according to NDS-01 requirements, and preservative, such as copper naphthenate, must be sprayed into the holes. Lag screws, as well as bolts, need to be retightened regularly (Falk et al., 1993).

3.1.11.6 Hangers and Straps

Hangers and straps are intended for indoor use (Falk et al., 1993), they are usually electroplated (McDonald et al., 1996), however using stainless steel or galvanizing will increase the life (Falk et al., 1993).

Manufacturers of hangers provide literature on the properties of their products. Allowable loads for hangers are based on particular nail sizes and adjustment factors, provided in the literature, must be applied when different nail sizes are used (Simpson, 1997).

3.1.11.7 Coatings

Adhesive coatings are chromate paint and have been known to flake off during installation. Galvanized coatings are “sacrificial” coatings, once they are gone, the fastener has nothing left to protect it from corroding. The thicker the coating, the more protection there is. Minimum coverage is 0.85 oz/ft², and for long-term high humidity environments 1.0 oz/ft² is recommended. The

galvanizing is done by electroplating, mechanical plating or dipping.

Electroplated fasteners are the most commonly used, especially since they are available for nail guns (Falk et al., 1993).

A study reported by Baker (1992) compared the percent weight loss of hot-dipped galvanized, mechanically galvanized, and electrolytically galvanized nails in treated wood. The hot dipped nails lost weight at the lowest rate, losing 16% of their weight after 17 years in CCA I and 8% of their weight after 17 years in CCA II. After 17 years, both the mechanically and electrolytically coated nails had disintegrated (Baker, 1992).

3.1.11.8 Corrosion

In the investigation described by Bohnhoff (2002), the deck nails had corroded to the point where they left only ferric oxide and small parts of the nail shank. The lag bolts were at 50% of their original cross sectional area (Bohnhoff, 2002). Corrosion is an electrochemical process depending on the type of metal, the electrical conductivity of wood, the length of time the wood is wet, temperature of wood, the wood species, the presence of contaminants and preservatives and the overall condition of the wood (Baker, 1988; Falk et al., 1993). It requires only two things: moisture and oxygen (Bohnhoff, 2002). When the wood surrounding the metal is at 18% moisture content, conditions are good for corrosion (Baker, 1988; Falk et al., 1993). A lower moisture contents, the electrical resistance of the wood is high enough to inhibit the current flow required for corrosion (Freas, 1982). High moisture contents may be reached when there is condensation around the nail (Baker, 1988). The first sign of

corrosion is iron stain (Falk et al., 1993). Damp wood is acidic, especially Douglas-fir and western red cedar (pH of 4 – 6) and this condition accelerates corrosion of metals (Baker, 1980).

Corrosion resistant nails do not prevent decay of wood around them (Yang, 2001). The wood will deteriorate around the corroding metal. The acidic conditions oxidize the cellulose (Baker, 1988) with the corroding iron as the catalyst in the reaction. The chemical deterioration of wood causes embrittlement and a loss of tensile strength (Bohnhoff, 2002).

Crevice corrosion occurs with a single fastener. Hydroxyl ions form on the exposed ends if they are not washed away and attract chlorides to the nail, where they cause corrosion and weaken the wood. The nail head is considered the cathode and the shank is the anode. Iron ions on the anode react with hydroxyl ions in crevice to create iron hydroxide, which creates a more acidic solution. With these reactions, the pH may reach as low as 2 to 3. This process starts slow, but the rate increases with decreasing pH (Baker, 1980).

When two dissimilar metals are in contact in a corrosive environment, the less resistant metal becomes the anode and has a high rate of corrosion while the more resistant metal becomes the cathode and has very little corrosion (Baker, 1980). For this reason, the washer, nut and bolts should be made of the same material (Falk et al., 1993).

Corrosion in preservative treated wood depends on the type of preservative. With oil-type preservative, the heavy oils inhibit corrosion by increasing electrical resistance (Falk et al., 1993; Freas, 1982), but there is

corrosion in areas without treatment (Falk et al., 1993). In water-borne preservative treated wood, if the moisture content is greater than 18%, the electrical conductivity is high enough for corrosion to take place (Baker, 1988). The high chromate content in CCA treated lumber inhibits corrosion (Baker, 1980). When the formula is made up of potassium dichromate, copper sulfate and arsenic acid, the byproduct is potassium sulfate, which causes increased electrical conductivity. With chromium trioxide, copper carbonate and arsenic acid, the byproduct is carbon dioxide, which dissipates. A test performed by Baker (1988) with hot dipped galvanized nails at 100% relative humidity and 80°F for 14 years showed considerably reduced cross sections in CCA I and somewhat reduced cross sections in CCA II. In the same conditions, stainless steel (AISI type 316 and 304) nails showed little to no corrosion (Baker, 1988). In copper salt preservatives, the copper ions are soluble and cause corrosion in metals (Baker, 1988; Falk et al., 1993).

3.1.12 Railings and Stairways

All railings have required load capacities according to the model building codes. They are required for certain distances above the ground and have minimum heights (McDonald et al., 1996). The specific requirements are given in a later section. The dimensions of stairways are also described in the building codes and have their own railing requirements.

For the attachment of the railing to the deck, good practice calls for avoiding notches, which cause splitting (McDonald et al., 1996). Also, through bolts are recommended to resist the resultant forces from building code-required

loads that may be over 2,000 lbs (Randall, 1994). Options offered by Randall (1994) include bolting (with two 1/2 in. through bolts or lag screws) the rail posts to the rim joists, using a custom-made steel pipe connector, extending the deck posts up through the deck, or attaching the railing to a wall whenever possible.

Infill is the pickets of the railing system and the structural elements between the rails. They provide protection from falling and resistance to horizontal thrust. It is also recommended that they deter climbing (ASTM, 1993).

On structures with solid railing systems, proper drainage must be provided to ensure water is removed from the deck or balcony without endangering the structural elements (Smith, 2002).

3.1.13 Overhangs/Cantilevers

Overhangs should be limited to 25% of first interior bay joist span (McDonald et al., 1996). Fairfax County, Virginia, limits joist overhangs to 3 ft and beam overhangs to 2 ft (Fairfax County, 1998). They produce uplift on first interior post/joist and require steel twist strap. The joist supporting the overhang or cantilever must be continuous over support (McDonald et al., 1996).

3.1.14 Redundancy

The deck design and failure described by Bohnhoff (2002) had no redundancy, when one fastener failed, the whole system failed. The hung balcony was attached to the house using two triangular supports, one on each side, that were attached to framing with lag bolts (Bohnhoff, 2002).

3.1.15 Footings

Footings must reach below frost penetration depth (McDonald et al., 1996)

3.1.16 Landscape

Under decks, the grass will die and erosion will become a problem. The water must be diverted from flowing into this area. Also, the presence of weeds will cause high local humidities (McDonald et al., 1996).

3.2 Standards/Codes

Building codes are legal documents accepted by localities (Freas, 1982). As the codes have evolved, the requirements for decks and balconies have become more stringent, going from nearly nothing in the 1976 Southern Building Code to a strict guide provided by officials in Fairfax County, Virginia. The differences between the model and other codes are shown in Tables 3.2 and 3.3.

Table 3.2. Load and other requirements for residential (one and two family) decks from the 1976 Southern Building Code, the 1992 and 1995 CABO codes, the 1995 ASCE Minimum Design Loads, the 1998 Fairfax County Deck Details and the 1998 and 2000 International Code Council.

Requirement	Southern Building Code, 1976	CABO, 1992	CABO, 1995	ASCE 7, 1995	Fairfax County, 1998	International Code Council, 1998	IRC, 2000
Live Load (psf)		40	40	40	40	40	40
Dead Load (psf)*					10		
Railing Load (plf)				20	50		
Concentrated Load for railing (lb)			200	200	200	200	200
Infill Load, Over 1 ft ² area				50			
Maximum Opening, In railing. Will not allow a sphere of this diameter to pass through		6" 4" for horizontal rails	4"		4"	4"	4"
Maximum Opening, In triangle at stairway			6"		6"	6"	6"
Height above grade at which guards are required			30"			30"	30"
Minimum height of guards, railing			36"		36"	36"	36"
Minimum Rise of stairs required for guard	4 risers	30"	30"			30"	30"
Minimum height of guards, stairway**		34"	34"		36"	34"	36"

Table 3.3. Load and other requirements for balconies from the 1995 ASCE Minimum Design Loads, 1996 BOCA, ASTM E985-00e1, the 1997 Southern Building Code and the 2000 International Building Code.

Requirement	ASCE 7, 1995	BOCA, 1996	ASTM, 1996	Southern Building Code, 1997	IBC, 2000
Live Load (psf)	100 psf 60 psf (less than 100 ft ²)	100 psf 60 psf (less than 100 ft ²)			100 psf 60 psf (less than 100 ft ²)
Dead Load (psf)*					
Railing Load (plf)	50	ASCE 7	50 (public access) 60 (public area)	50 horizontal, 100 vertical	50
Concentrated Load for railing (lb)	200	ASCE 7	300 (public access) 365 (public area)	200	200
Infill Load, Over 1 ft ² area	50	ASCE 7	50	200	50
Maximum Opening, In railing. Will not allow a sphere of this diameter to pass through		4"	5.5" (4" if children are present)	4" (2" below bottom rail)	4" (0 to 34" above surface) 8" (34 to 42" above surface)
Maximum Opening, In triangle at stairway		6"		6"	6"
Height above grade at which guards are required		15.5" 30" when screened		30"	30"
Minimum height of guards, railing		42" 36" If less than 30" above grade	42"	42"	42***
Minimum Rise of stairs required for guard					30"

* Dead load is generally the weight of the materials themselves.

** If the top rail also serves as a handrail (R-2: apartments, two-dwelling units and R-3: permanent) height must be $\geq 34"$ and $\leq 38"$ vertically from the nose of the leading tread

Recently, decay protection has become required, depending on the location. Preservative treated wood is necessary when used as structural elements that will be exposed to the weather or supporting permeable floors. The building codes also require quality marks (or tags) for preservative treatments (CABO, 1995).

The 2002 edition of the state of North Carolina residential building code (International Code Council, 2001) includes a section dedicated to wood decks. Requirements include a treated band joist where the deck is attached or corrosion resistant flashing (aluminum flashing is not allowed). Siding must be removed before the deck is attached. Fastener schedules are included for attaching the ledger board to the house with bolts or nails. Other specifications are the deck post sizes, beam attachment to the posts, lateral stability and freestanding decks.

3.3 Do-it-yourself books

Homeowners interested in adding a deck to their home as a do-it-yourself project often seek help from a local hardware store or from books. Almost all of the do-it-yourself literature provides information on lumber sizes, but few, if any, offer any information on fastener sizing. The following is a summary of information from several books and pamphlets obtained at area hardware stores or requested by mail.

3.3.1 Materials

Using screws instead of nails in the decking is recommended because they are easy to remove, help prevent twisting and mistakes can be fixed without having to pry out a nail (Cory, 2001). For attaching structural members, use of lag screws or bolts is recommended. Lag screws should penetrate into main member two times the thickness of the side member. Bolts should be 1 in. longer than the total thickness of all members. All fasteners should be corrosion resistant, either galvanized or stainless steel (Staub, 2001).

Preservative treated lumber is rated by the amount of preservative retained in the wood. The amounts of preservative available vary from 0.25 lbs/ft³ to 0.60 lbs/ft³. For lumber used above ground, 0.25 lbs/ft³ is adequate. 0.40 lbs/ft³ is adequate for ground contact, but 0.60 lbs/ft³ is recommended for deck posts. Finishing is required to maintain preservative treated lumber, just as it is required for non-treated lumber (Marshall, 2002).

Specially manufactured lumber, such as the product advertised as Wolmanized® Lumber claims to be resistant to fungal decay and termites (Wolmanized®, 1998).

3.3.2 Drainage

The homeowner must realize that after the deck is built, the same amount of water as before will reach under the deck, but it will not evaporate as readily (Cory, 2001). Therefore, the ground under the deck must slope away from house 1/4 in. per foot (Staub, 2001) or a drainage ditch with a perforated drainpipe

should be installed. Also, gutters emptying under the deck should be moved to another location (Cory, 2001).

3.3.3 Attachment at ledger

Local codes dictate exactly how the ledger board should be attached to the house and should be checked first. The authors' recommendations for ledger attachment include flashing use, attachment methods and fastener sizes and spacings.

Flashing should go under the siding and over the ledger (Straub, 2001). Aluminum flashing will last, but temperature changes cause it to expand and contract, which, in turn, causes nails to pull out and expose areas for water to pass through. Aluminum nails should always be used with the aluminum flashing. Galvanized flashing will develop rust spots, but will not expand or contract as much as aluminum with changes in temperature (Cory, 2001).

The ledger board should be made from the same material as the deck joists (Cory, 2001) and be pressure treated (Marshall, 2002). It should be attached to the band joist with thru bolts, through the plywood sheathing and with spacers for drainage (Cory, 2001). If the band joist is not accessible, the ledger should be attached to concrete block, solid concrete or the wall studs (Marshall, 2002). When the homeowner expects the deck to support heavy loads, two fasteners (at top and bottom) spaced every 16 in. are recommended. For lighter loads, alternating top and bottom fasteners every 24 to 32 inches is acceptable. When the house has stucco siding, the fasteners should be spaced every 16 in. With masonry, a threaded rod with epoxy in the holes is recommended to bond

the rod into the hole and secure the connection (Staub, 2001). If bolts are used, they should be 5 in. long; lag screws should be 3/8 in. (Marshall, 2002) or 1/2 in. diameter (Cory, 2001) and at least 4-1/2 in. long to penetrate into sheathing and framing (Staub, 2001).

According to Staub (2001), the ledger will crush siding if it is attached over it. Marshall (2002) strongly recommends against attachment to the siding. The Wolmanized® (1998) pamphlet only recommends removing siding if it is aluminum or vinyl. Cory (2001) provides several methods for attaching the ledger, including techniques for attachment over the siding. With flat siding or masonry, the ledger should be joined tightly and flashing is only needed if it is required by code. Also, for a flat surface, several washers or pressure treated shims could be used to create enough space for water to run off and easily dry out. With sloped siding, boards could be cut and installed to create a flat surface for ledger attachment. If the siding is stucco or masonry, a channel must be cut for the flashing. Finally, if the siding is cut away, flashing should be used (Cory, 2001).

Other points emphasized for the ledger board attachment are to always avoid trapped moisture (Cory, 2001) and to create a small step down from the house floor to the deck to prevent rain and snow from entering the house (Cory, 2001; Marshall, 2002).

3.3.4 Posts

Sizing for the posts depends on the deck height. If the deck is 6 ft. or less off the ground, 4x4s can be used, otherwise, 6x6s should be used. The posts

could be sunk into footing holes and secured with concrete or held in place with concrete piers and post anchors, which keep the post a few inches above ground and safe from rot (Cory, 2001). Footings should be a minimum of 24 in. deep or below the frost line (Wolmanized®, 1998). If the deck is over a slope, the bottom of the footing should be 7 ft horizontally from the incline (Staub, 2001; Marshall, 2002).

Bracing, either in Y, K or X configurations, should be used on the posts if they are above 4 ft high (with 4x4s) or 8 ft high (6x6s) (Cory, 2001). If the deck is attached to the house, bracing is needed if it is more than 8 ft above the ground. Freestanding decks need to be braced if they are more than 3 ft high (Staub, 2001). The posts may be sunk into the concrete footings to provide some resistance to movement (Cory, 2001).

3.3.5 Beams

Beams can be attached to posts by stacking, sandwiching, attaching to side of using lag screws or notching. Stacked beams are more resistant to downward forces, but side-attached beams have more resistance to twisting. When the post is notched, it should be a 6x6 and the top of the beam should be flush with the top of the post. Stacking should not be done on top of a 6x6 because the post end grain is exposed unless the beam is also a 6x_ (Cory, 2001).

When a built up beam is used, three screws should be used every 2 ft, and should not be located less than 1-1/2 in. from the edge. Marshall states that beams should be bonded with exterior construction adhesive and 2-3/4 in.

galvanized screw every 6 in. alternating top and bottom. Also, beams should be cantilevered no more than 1/4 of the total span length for a solid feel (Cory, 2001).

3.3.6 Joists

Joists may overhang the beams one-quarter to one-third their total span length, leaving three-quarters to two-thirds of the length supported (Cory, 2001). However, Staub (2001) and Marshall (2002) recommend that joists be cantilevered no more than 1/4 of the allowable span between the supports (Staub, 2001).

3.3.7 Railings

Cory (2001) recommends notching deck boards around post rather than post itself and states that notching the rail post near one-half of its thickness can cause it to crack when pressure is applied (Cory, 2001). Straub gives detailed instructions on how to notch railing posts when attaching them to deck (2001).

3.4 Inspections

Generally, inspection of wood structures is needed when there is a change of use or there has been some kind of damage, collapse, unservicability or deterioration. Also, it is recommended whenever there are important changes in the building codes (Freas, 1982). Routine inspections identify potential problems. Their timing should be based on local situations, accidents, biological activity, the nature of the material and the quality of the design and construction

(Forest Products Laboratory, 1985). McDonald (1996) recommended annual inspections.

3.4.1 Preparation

Before inspection, information should be gathered on the deck itself. The age, size, design, previous inspections, damage, repair, replacements and modifications should all be known before the inspection takes place (Eslyn et al., 1979). Necessary materials include a flashlight, ladder, measuring tape, fishing line (to see deflections), a pointed tool for probing, a moisture meter, camera and a hammer (Forest Products Laboratory, 1985). A systematic approach must be prepared to be sure nothing is missed (Eslyn et al., 1979). This plan will also show areas where there is a need to expose hidden members (Pneuman, 1991).

3.4.2 Original Structural Design

The species and grades used must be known to determine the original strength values (Forest Products Laboratory, 1985). Grade marks show the supervising agency, mill identification, grade, species or species group, and seasoning (McDonald et al., 1996).

3.4.2.1 *Proper installation*

There is a possibility that pieces may be mixed during construction, and a defective piece could be in a critical location. The inspector should locate spacing deviations and modifications to original plans, including notching or

removal (Grossthaner et al., 1991). Butt joints in the beam spans should not be permitted, even in built up beams (McDonald et al., 1996).

3.4.2.2 *Quality*

Knots, grain angle, checks, and splits could degrade the quality of the structure (Forest Products Laboratory, 1985) as the associated openings could accommodate more rapid decay.

3.4.2.3 *Connections*

The connections distribute the load through all members down to the ground (Grossthaner et al., 1991). It is good practice for the structure to not be totally dependant on the fasteners (McDonald et al., 1996). The inspector should check for edge distances, tightness (Grossthaner et al., 1991) and washers under every head of lag screws and bolts. At the house attachment, check for water trapping at the joint between deck and house (McDonald et al., 1996).

Signs of early problems in the connections are rust and iron stain. To identify the difference between iron stain and mildew, Williams et al. (2002) proposed a method of using a saturated solution of oxalic acid in water to remove the discoloration. If the solution cleans the wood, then cause was iron stain. If bleach removes the stain, it was mildew.

3.4.3 Serviceability

Knowledge of the design will help the inspector locate overstressed members (Forest Products Laboratory, 1985). A load test is necessary when

there is a question for the safety for the intended use (ASCE, 1995). ASTM has produced methods for testing steel guardrail performance (ASTM, 1993). They also provide standards for the performance and deflection limits that are based on the height and length of guardrail, including the residual effects (ASTM, 2000).

3.4.4 Physical Signs of problems

Visual inspections may not reveal all problems, but there are several that may be exposed with a close look.

3.4.4.1 *Moisture Content*

Areas of sustained high moisture content are wide cap rails, horizontal rails at the bottom of vertical balusters, low ends of sloping rails and wide deck boards (Gaby et al., 1978). They are characterized by paint failure (blistering and peeling), buckling and nail pulling out (Verrall et al., 1980).

3.4.4.2 *Failures*

Excessive deflections, crushing, and fractures show evidence of member failure. Connection failures are caused by looseness, shearing and rust. The behavior of a connection that is hit with a hammer shows its tightness. Loose connections have excessive vibration and hollow sound if loose. The ring is solid if the connection is tight (Forest Products Laboratory, 1985). Connections should be tightened four to five months after installation then every five years following (Freas, 1982). Inspection locates sheared off bolts (Pneuman, 1991). By the

time rust is visible on surface, the corrosion is probably extreme (Verrall et al., 1980).

Splits are not critical if they are outside connector area and parallel to the length axis of the piece (Forest Products Laboratory, 1985).

On balconies with waterproofing products over sheathing (which usually require a 1/4 in. per foot slope away from the building), fractures or delaminations in the surface occur over joints in the sheathing when the support system below it is water damaged (Smith, 2002).

Other signs of problems are: raised grain (the latewood rises above early wood), shelling (separation of latewood and earlywood), fractures and delaminations, low spots and soft spots, sagging and shrink/swell (McDonald et al., 1996; Pneuman, 1991; Forest Products Laboratory, 1985).

3.4.4.3 *Exposure*

Areas that are exposed to weather are likely to have problems. Other problem areas are end grain and areas in ground contact (Forest Products Laboratory, 1985). When untreated lumber is used, there should be at least 8 in. between the soil and framing members, in areas with frequent heavy rains, the distance should be 12 to 18 in. (Forest Products Laboratory, 1999).

3.4.4.4 *Decay*

It is conservative to estimate that visibly decayed areas have no strength and will continue to deteriorate. The selections of sites for probing/coring/boring should be near watermarks, rust stains, plant growth, joints and water trapping

areas (Eslyn et al., 1979). Decayed areas have a musty or stale odor (Forest Products Laboratory, 1985). Eslyn recommended putting the decay conditions in three categories: (1) Existing decay with severe strength loss, which requires immediate repair and restricted use (2) Existing decay without limiting service, where the conditions will worsen over time and (3) Conditions conducive to decay, where preventative maintenance is required (Eslyn et al., 1979). DeBonis (1999) stated that once advanced decay is located, further examination of that area is not necessary; the wood has undergone extreme strength loss and must be repaired or replaced.

3.4.5 Repair

The 1996 BOCA code states, “When repairs are made to structural elements of an existing structure and uncovered structural elements are found to be unsound or otherwise structurally deficient, such elements shall be made to conform to the requirements for new structures.” According to the 2000 International Building Code, “Additions, alterations or repairs to any building or structure shall conform with the requirements of the code for new construction.”

In making repairs, it is important to note that replacement members will be exposed to the same conditions (Eslyn et al., 1979). Surface preservative treatments have little effect, and all affected members should be replaced (Verrall et al., 1980).

3.5 Conclusion

With an increase in reported deck failures, the need for a deck and balcony inspection manual to be used by engineers and home inspection professionals has become evident. The purpose of this project is to not only complete an inspection manual, but also to spread the word about the significance of regular deck inspections to protect the users.

4. Investigations

4.1 Fairfax County Typical Deck Details

4.1.1 Introduction

Fairfax County, Virginia, provided typical deck details as a guide for homeowners attaching decks to existing structures. The history of the deck details is described in Section 1.1. To be able to understand and/or endorse the guidance given in the deck details, the following is presented as a review summary of engineering design criteria and analyses are presented.

4.1.2 Objectives

To independently verify the analysis and design data used in the development of the Fairfax County Typical Deck Details.

4.1.3 Table 1 in the Fairfax County Deck Details: Maximum Joist Spans and Beam Sizes

Table 1 of the Fairfax County Deck Details is reproduced in Table 4.1.

Table 4.1. Maximum joist spans and beam sizes as recommended by the Fairfax County Deck Details (Table 1, Fairfax County, 1998)

Joist Size	Joist Spacing	Maximum Joist Span (all conditions)	Beam Size for Simple Span Joists	Beam Size for Overhang Joists
2x6	16"	9'-7"	2-2x8	2-2x10
2x6	24"	7'-10"	2-2x8	2-2x10
2x8	16"	12'-2"	2-2x10	2-2x12
2x8	24"	10'-1"	2-2x8	2-2x12
2x10	16"	14'-10"	2-2x10	3-2x12
2x10	24"	12'-1"	2-2x10	2-2x12
2x12	16"	18'-9"	2-2x12	3-2x12
2x12	24"	15'-4"	2-2x12	3-2x12

4.1.3.1 Design Methods

The following equations are the design criteria for bending members.

Equation 4.1 is the requirement for bending strength, Equation 4.2 is the requirement for shear strength, Equation 4.3 is the requirement for compressive strength perpendicular to the grain, and Equation 4.4 is the requirement for deflection.

$$f_b \leq F_b' \quad [4.1]$$

$$f_v \leq F_v' \quad [4.2]$$

$$f_{c-perp} \leq F_{c-perp} \quad [4.3]$$

$$\Delta_{max} \leq \Delta_{allowable} \quad [4.4]$$

f_b = bending stress, psi

F_b, F_b' = tabulated and allowable bending design value, psi, respectively

f_v = shear stress parallel to grain, psi

F_v, F_v' = tabulated and allowable shear design value, psi, respectively

f_{c-perp} = actual compressive stress perpendicular-to-grain, psi

F_{c-perp} , F_{c-perp}' = tabulated and allowable compression design value
perpendicular to grain, psi, respectively

$\Delta_{allowable}$ = allowable live load deflection, in.

Δ_{max} = maximum live load deflection, in.

The NDS-01 (NDS-01, Table 4.3.1) requires that certain factors be applied to the tabulated design values. The factors applicable to deck design are as follows:

$$F_b' = F_b C_F C_D C_r C_L C_M \quad [4.5]$$

$$F_v' = F_v C_F C_D C_M \quad [4.6]$$

$$F_{c-perp}' = F_{c-perp} C_M C_b \quad [4.7]$$

The factors are: the size factor, C_F , the load duration factor, C_D , the repetitive member factor, C_r , the beam stability factor, C_L , the bearing area factor, C_b , and the wet service factor, C_M . The tabulated values for bending strength, shear strength and compressive strength perpendicular to grain for the lumber grade and size required by the Fairfax County Deck Details are shown in Table 4.2.

Table 4.2. The tabulated bending, shear and compression perpendicular to grain design values for sizes of No. 2 Southern Pine dimension lumber.

Size	F _b (psi)	F _v (psi)	F _{c,perp} (psi)
2x6	1250	175	565
2x8	1200	175	565
2x10	1050	175	565
2x12	975	175	565

4.1.3.2 Bending Design

Actual bending stress in a member is found using Equation 4.8 and the maximum moment in a simple span bending member with a code-specified uniform load is given by Equation 4.9.

$$f_b = \frac{M_{\max}}{S} \quad [4.8]$$

$$M_{\max} = \frac{wL^2}{8} \quad [4.9]$$

To determine the maximum joist spans, f_b was set equal to F_b' using the design criterion from Equation 4.1. Solving Equations 4.1, 4.8 and 4.9 for the maximum span yields Equation 4.10.

$$L = \sqrt{\frac{8F_b' S}{w_j}} \quad [4.10]$$

M = maximum bending moment, in.-lb

S = section modulus, in.³

w_j = uniformly distributed load on joist, pli

L = maximum allowable span, in.

The Fairfax County Deck Details allow three span cases (Case 1: simple span, Case 2: overhangs on both sides and Case 3: overhang on one side). On joists, the allowable overhang is 3 ft on each side of the deck (Figure 4.1).

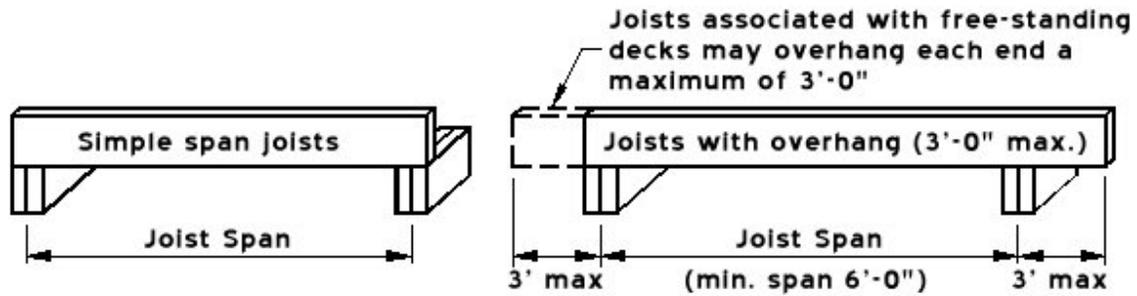


Figure 4.1. A simple joist span is from the center of one bearing to the center of the other. Joist spans with may have a maximum of 3'-0" overhang on each side.

This does not affect the calculations for maximum span, as the moment in a simple span joist under a uniform load is greater than the maximum moments for spans with overhangs. For example, a 2x6 No. 2 Southern Pine joist with 16 in. on-center spacing and a span of 9'-7" has a maximum moment of 468 ft-lb for two overhangs, 623 ft-lb for one overhang and 767 ft-lb for a simple span. Therefore, the maximum moment in a simple span joist was used to find all maximum joist span values in Table 1 of the Fairfax County Deck Details (Table 4.1 in this text).

Beam (or girder) sizes were found by solving Equations 4.1, 4.8 and 4.9 for a required section modulus [Equation 4.11].

$$S_{req} = \frac{w_b (L_b + a_b)^2 (L_b - a_b)^2}{8L_b^2 F_b'} \quad [4.11]$$

w_b = uniformly distributed load on beam, lb/in.

a_b = length of beam overhang, in.

All beam designs allow for a single overhang of a maximum 2 ft. The beam spans are shown in Figure 4.2.

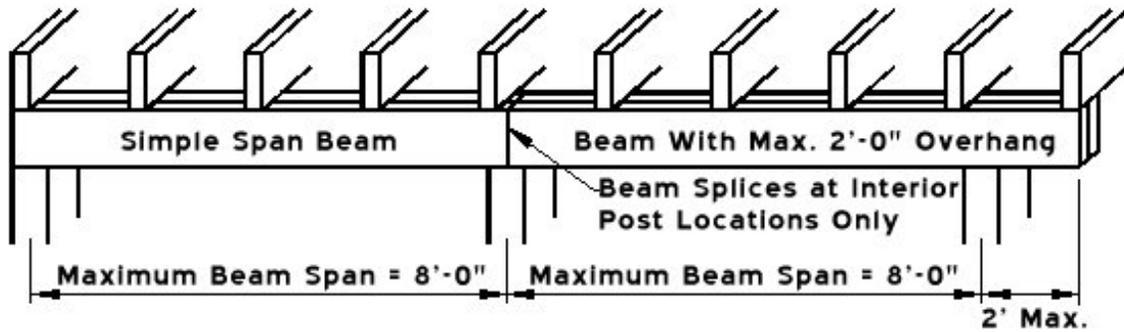


Figure 4.2. The beam spans are shown with the joists stacked on top and the maximum overhang of 2'-0".

The distributed load per ply on the beam, w_b , was found using the reactions from each joist and the joist spacing. The maximum reaction force occurred when the joist had an overhang on one side [Equation 4.12].

$$w_b = \frac{w_j(L_j + a_j)^2}{2L_j * joist\ spacing * plies} \quad [4.12]$$

When the joists do not have overhangs, the distributed load per ply on the beam is shown by Equation 4.13.

$$w_b = \frac{w_j L_j}{2 * joistspacing * plies} \quad [4.13]$$

4.1.3.3 Shear Stress

The maximum shear stress in a simple span joist is shown in Equation 4.14.

$$f_v = \frac{3V_{max}}{2A} \quad [4.14]$$

Maximum shear in a simple span is at the bearing points [Equation 4.15].

$$V_{max} = \frac{wL}{2} \quad [4.15]$$

V_{\max} = shear force in joist or beam, lb

A = cross-sectional area of joist or beam, in.²

In the three span cases allowed by the Fairfax County Deck Details, the overhang on one side produces the highest shear in the joist. This shear value is found using Equation 4.16.

$$V_{\max} = \frac{w_j(L_j^2 + a_j^2)}{2L_j} \quad [4.16]$$

The joist was checked for adequacy using Equations 4.2, 4.6 and 4.14.

For checking the shear strength of the beams, Equations 4.2, 4.14 and 4.16 were used using values for beams.

4.1.3.4 *Compressive Strength Perpendicular-to-Grain*

Assuming the joists are stacked on the beams, the compressive stress perpendicular-to-grain was calculated by Equation 4.17.

$$f_{c,perp} = \frac{R}{A_{bearing}} \quad [4.17]$$

R = reaction force at bearing, lb

$A_{bearing}$ = area of bearing, in.²

The compressive strength perpendicular-to-grain was checked for the longest joist span assuming a two-ply girder support and it was adequate. Also, assuming a 6x6 post and using an 8 ft span beam, the compressive perpendicular-to-grain stress was less than the allowable for No. 2 Southern Pine.

4.1.3.5 Deflection

Allowable deflection of both the beams and the joists was L/360. The maximum deflection of a simple span was found using Equation 4.18.

$$\Delta_{\max} = \frac{5wL^4}{384EI} \quad [4.18]$$

E = tabulated modulus of elasticity, psi

I = moment of inertia, in.⁴

For joist and beams with one overhang, the maximum deflection occurred at midspan and was found by Equation 4.19 where x is the midspan point.

$$\Delta_{\max} = \frac{wx}{24EIL} (L^4 - 2L^2x^2 + Lx^3 - 2a^2L^2 + 2a^2x^2) \quad [4.19]$$

The deflection in the overhang was checked [Equation 4.20], also, with x_1 as the point at the end of the overhang.

$$\Delta_1 = \frac{wx_1}{24EI} (4a^2L - L^3 + 6a^2x_1 - 4ax_1^2 + x_1^3) \quad [4.20]$$

4.1.3.6 Adjustment Factors

The size factor, C_F , is not applicable for Southern Pine dimension lumber (12" or less) and was set equal to 1.0 for these calculations. Southern Pine does not require the use of the size factor (NDS-01, Section 4.3.6) because design values are published for the different nominal lumber sizes. The Fairfax County Deck Details used "normal" loading duration ($C_D = 1.0$). The NDS-01 recommends a C_r value of 1.15 when the members are used as joists, spaced

less than 24 in. on-center, with three or more members in the system and joined by a load distributing element.

The wet service factor, C_M , is applied to applications where the moisture content of the lumber will exceed 19% in service. For bending stress, C_M is 0.85 when the tabulated bending stress with the size factor applied is greater than 1150 psi. When the bending stress is less than or equal to 1150 psi, the wet service factor is equal to 1.0. For shear strength, the wet service factor is 0.97 and for compressive strength perpendicular to the grain, C_M is 0.67.

The beam stability factor, C_L , was assumed to be 1.0 for the beams and the joists. The joists were assumed to be laterally supported by the deck boards for their entire length, except for overhang spans when present. At the overhang, the laterally unsupported length was the length of the overhang (maximum 3 ft for joists and maximum 2 ft for beams). Using NDS-01 Table 3.3.3 and the method described in NDS-01 Section 3.3.3.8, the minimum C_L was 0.99. For the beams, the laterally unsupported lengths are 16 in. and 24 in., depending on the joist spacing. At the overhangs, the laterally unsupported length was 24 in. Using the methods described in NDS-01 Section 3.3.3, the value of C_L , including values at the overhangs, is nearly 1.0. Using a C_L equal to 1.0 required nailing as per the Fairfax County Deck Details with a recommended threaded hardened-steel (annular or spiral) nails (0.135"x 3.25"), with two rows (top and bottom), at 16 in. on center.

4.1.3.7 *Fairfax County Methods*

Using Equations 4.1, 4.5 and 4.10 and the Fairfax County Deck Details, the maximum spans for No. 2 Southern Pine are summarized in Table 4.3.

Sample calculations are included in Section 9.1.1 of Appendix A.

Table 4.3. Maximum spans of No. 2 Southern Pine joists

Joist Size	Joist Spacing	Max. Joist Span	
		Fairfax County	Thesis
2x6	16"	9'-7"	9'-7"
2x6	24"	7'-10"	7'-10"
2x8	16"	12'-2"	12'-4"
2x8	24"	10'-1"	10'-1"
2x10	16"	14'-10"	15'-11"
2x10	24"	12'-1"	13'-1"
2x12	16"	18'-9"	18'-9"
2x12	24"	15'-4"	15'-4"

Shaded cells indicate a difference in the results from the two sources.

The differences between the allowable spans in the Fairfax County Deck Details and the methods described herein stemmed from the use of the wet service factor. The NDS-01 allows C_M equal to 1.0 when the value of $(F_b)(C_F) \leq 1150$ psi. For a 2x10 No. 2 Southern Pine, the tabulated bending strength was 1050 psi and the size factor was 1.0. Using a C_M of 0.85 will produce the spans given by Fairfax County.

Table 4.4 is a comparison of the beam sizes given by Fairfax County and the beam sizes arrived at by using the methods described in this thesis. The maximum beam span for using the table was 8'-0". Sample calculations are shown in Appendix A (Section 9.1.1)

Table 4.4. Beam sizes for No. 2 Southern Pine beams with simple span joists and overhanging joists.

Joist Size	Joist Spacing	Beam Size, Simple Span Joists		Beam Size, Overhangs on Joists	
		Fairfax County	Thesis	Fairfax County	Thesis
2x6	16"	2-2x8	2-2x8	2-2x10	2-2x10
2x6	24"	2-2x8	2-2x8	2-2x10	2-2x10
2x8	16"	2-2x10	2-2x8	2-2x12	2-2x10
2x8	24"	2-2x8	2-2x8	2-2x12	2-2x10
2x10	16"	2-2x10	2-2x10	3-2x12	2-2x12
2x10	24"	2-2x10	2-2x8	2-2x12	2-2x10
2x12	16"	2-2x12	2-2x10	3-2x12	2-2x12
2x12	24"	2-2x12	2-2x10	3-2x12	2-2x12

Shaded cells indicate a difference in the results from the two sources

Bending was the controlling factor in all of the calculated beam sizes. The methods used are more conservative than Fairfax County. The reason for these discrepancies is the concern for safety by Fairfax County, rather than designing for absolute minimums.

4.1.4 Table 2 of the Fairfax County Deck Details: Fastener Spacing at Ledger Board

The use of lag screws to attach the ledger board to the house is permitted by the Fairfax County Deck Details. The minimum requirements are 1/2 in. diameter, galvanized lag screw with standard cut washers and placement as per Figure 4.3. A lag screw length or minimum penetration is not part of the requirements.

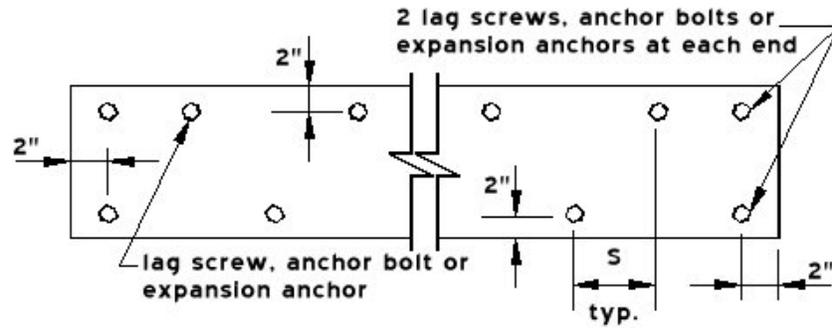


Figure 4.3. Spacing of fasteners and edge clearances as required by the Fairfax County Deck Details

The strength of lag screws is discussed in Section 4.2. The spacings shown in Table 2 of the Fairfax County Deck Details are shown in Table 4.5. For these spacings, each fastener is assumed to carry a certain load, also shown in Table 4.5.

Table 4.5. Spacing of fasteners at the ledger board as shown in Figure 4 and the resulting load per fastener for tabulated spans.

Joist Span	Spacing, S	Load per fastener
0 to 10'-0"	14" on center	292 lb
greater than 10'-0" to 14'-2"	10" on center	295 lb
greater than 14'-2" to 18'-9"	8" on center	313 lb

The loads per fastener were calculated [Equation 4.21] using the supported area and the live and dead loads (40 psf and 10 psf).

$$Load / fastener = \frac{LS}{2}(LL + DL) \quad [4.21]$$

L = maximum joist span, in.

S = fastener Spacing, in.

LL = Live load, psi

DL = dead load, psi

Assuming the a 2x Southern Pine ledger board attached to a 2x SPF band joist, the design value for a 1/2 in. bolt is 126 lbs; for a 1/2"x3.5" lag screw, the design value is 250 lbs without application of the penetration depth factor ($C_d \leq 1.0$) as discussed in Section 4.2.2.

4.1.5 Guardrail requirements

The rail cap of a deck guardrail is required by model codes (CABO, 1995; Southern Building Code, 1997; IRC, 1998) to be designed to withstand a 200 lb concentrated load at any point along its length. This load must be transferred to the post through the top rail, and the bolts carry the reaction force. Figure 4.4 is the Fairfax County required detail of a guardrail showing a 2x6 rail cap supported by a 2x4 top and bottom rail.

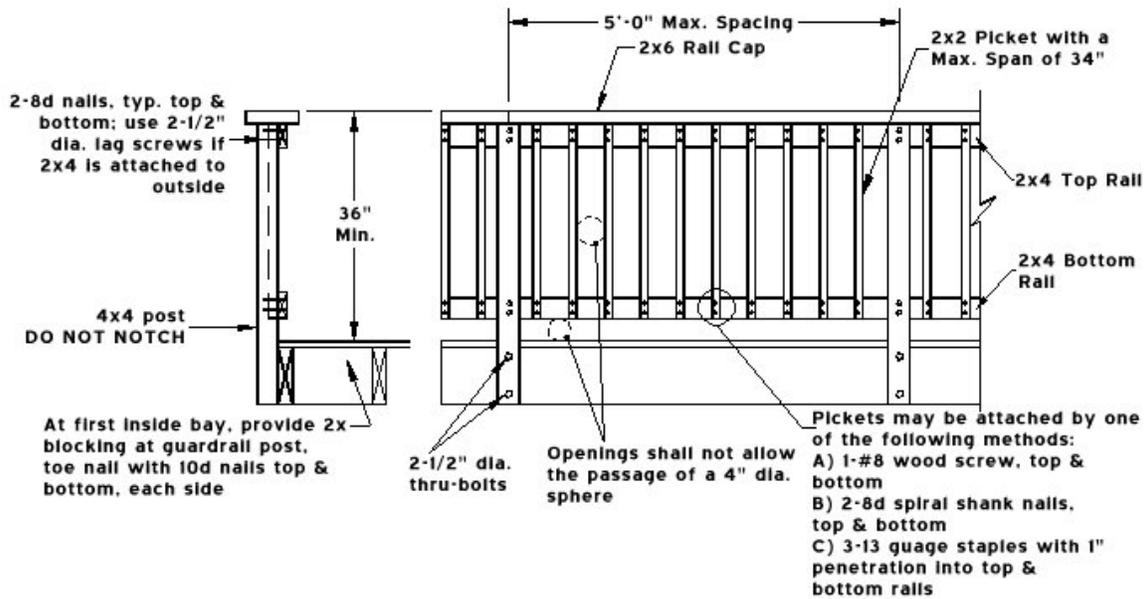


Figure 4.4. This typical guardrail detail is required by the Fairfax County Deck Details. The rail cap must withstand a 200 lb concentrated load and the infill must withstand 50 lbs over a 1 ft² area.

A free body diagram of a guardrail post is shown in Figure 4.5. The 200 lb concentrated lateral load produces 1,705 lbs and 1,505 lbs reactions in the top and bottom 1/2 in. diameter bolts (assuming a 2x10 rim joist and 2x_ decking which produces a maximum (conservative) force when 5/4 decking is used). These reaction forces cause withdrawal in the deck post-to-deck connections, which is why thru bolts are required. Washers are always required on both ends when bolts are used for wood connections (NDS-01, Section 11.1.2.3).

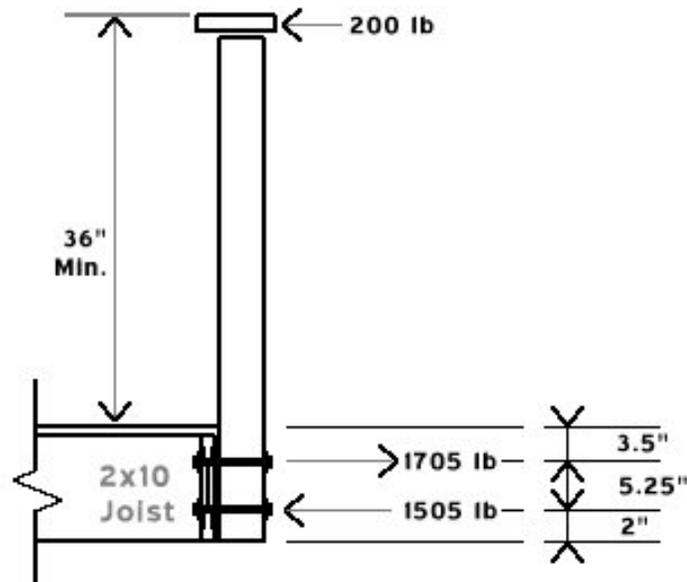


Figure 4.5. This free body diagram of a guardrail post shows the reactions in the 1/2 in. diameter thru bolts caused by a 200 lb force applied to the cap rail.

When the 200 lb force is applied vertically, the shear force in each bolt is 100 lbs. Using an excel program for bolts in single shear based on the NDS-01, the allowable shear in each bolt is 182 lb (260 lb without the wet service factor of 0.7). The group action factor, C_g , with two fasteners is 1.0. It can thus be concluded that the vertical force is safely resisted.

Allowable wet-service bending stress in a 4x4 No. 2 Southern Pine post is 1,275 psi. Assuming a 36 in. high post attached to a 2x10 joist, as shown in Figure 4.5, the maximum moment 7,900 in-lbs, which produces a maximum bending stress of 1,106 psi.

4.1.6 Posts

The Fairfax County Deck Details require 6x6 posts with maximum height limit of 14'-0". The maximum beam span is 8'-0" and the maximum joist span is

18'-9", which, in theory, could produce a 150 ft² area to be supported by a single post. The design compressive stress, found by Equation 4.23, in the post is 248 psi and the allowable compressive strength is 300 psi. Sample calculations are shown in Appendix A, Section 9.1.1.6.

$$f_c \leq F_c' \quad [4.22]$$

$$f_c = \frac{P}{A} \quad [4.23]$$

$$F_c' = F_c C_D C_F C_M C_P \quad [4.24]$$

From NDS-01:

$$C_P = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left(\frac{1 + (F_{cE}/F_c^*)}{2c}\right)^2 - \frac{(F_{cE}/F_c^*)}{c}} \quad [4.25]$$

$$F_{cE} = \frac{K_{cE} E}{(l_e/d)^2} \quad [4.26]$$

$$F_c^* = F_c C_D C_F C_M \quad [4.27]$$

F_c, F_c' = tabulated and allowable compression design value parallel to grain, psi, respectively

f_c = actual compression parallel to grain, psi

P = reaction force on column, lb

A = cross-sectional area of column, in.²

C_p = column stability factor

F_{cE} = critical buckling design value for compression members, psi

K_{cE} = Euler buckling coefficient for columns

c = 0.8 for sawn lumber

l_e = effective span length, in.

d = depth of bending member, in.

4.1.7 Conclusion

Reasons for all the differences in the deck joist and beam results from the methods described above and the requirements given by Fairfax County (Table 1 in the Typical Deck Details) have been determined. The reason for the discrepancy in spans for 2x10s was determined to be the use of the wet service factor. Table 4C (page 39, NDS-01 Supplement) in the NDS-01 requires that when $F_b \leq 1,150$ psi, C_M is equal to 1.0. If the calculations are redone using C_M equal to 0.85, then the results are the same as the Fairfax County Details. After a discussion of the methods used, it was determined that Fairfax County's concern for safety over minimum design values was the reason for all over-designing in Table 1 of the Fairfax County Typical Deck Details.

Loads carried by each bolt spaced as per Table 2 in the Fairfax County Deck Details were found, and the load carrying ability of lag screws is discussed in section 4.2. Forces in the bolts at the guardrail posts were also found and the allowable loads in these bolts were checked.

The maximum axial load carried by a 6x6 post was verified to be adequate per the provisions of NDS-01.

4.2 Lag Screw Design Values

4.2.1 Introduction

Lag screw design values in the NDS-01 are based on two member connections, with both members coming from the same species (or group) and the assumption that the threaded portion of the lag screw is not in the shear plane. For any other combination, the European Yield Mode (EYM) equations given by the NDS-01 (Section 11.3.1) must be used. They depend on the dowel bearing strength of the members, the bending yield strength of the screws, penetration of the screw in the main member and shank diameter of the screws.

Penetration minimums have been determined by the NDS-01, and reduced design values are required when the penetration into the main member is less than $8D$. The lowest allowable penetration is $4D$ and below this level, tabulated lag screw values are not allowed. Penetration is the length of the screw, minus the thickness of side member, minus the thickness of the washer (if present) and minus the tapered tip.

The objective of these calculations was to compare the design values of several sizes of lag screws installed in dimension lumber and an engineered lumber product used as main members.

4.2.2 Materials and methods

Four 3.5 in. lag screws were purchased at a local building supply store and the measurements are tabulated in Table 4.6. Their effective length was defined as the actual length of the screw minus the tapered part. The shank and

root diameters were measured for the sample of 3.5 in. screws and are reported in Table 4.6. The dimensions of 3 in. screws, given in Table 4.6, were determined using Appendix L of the NDS-01. The threaded portion of both screw sizes was in the shear plane of the connection; so all calculations in this section were conducted using the root diameter. These measurements are compared to the manufacturing standards set by ANSI/ASME B18.2.1 -1996.

Assumed yield strengths for the different size screws are given in the footnote to NDS-01 Table 11J. Table 11J design values are based on the following assumed steel properties: For diameters of 1/4 in., the bending yield strength (F_{yb}) is 70,000 psi, for 5/16 in. diameters, F_{yb} is 60,000 psi, and for diameters greater than 3/8 in. the F_{yb} is 45,000 psi.

The following equations (all used as per the NDS-01) were applied to the case of a band joist and ledger connection with a side member of PPT 2x_ Southern Pine ledger board ($G = 0.55$) and main members of 1 -1/4 in. Timber-Strand[®] band joist (equivalent $G = 0.58$) or 2x_ SPF ($G = 0.42$). The equivalent specific gravity of Timber-Strand[®] was obtained from Johnson and Woeste (1999).

Equations 4.27 through 4.29 give allowable loads corresponding to the three different yield modes of lag screws. When finding an allowable load for a connection, the minimum of these three values is used. Equation 4.27 represents Mode I_b failure, which is a bearing failure of the side member.

$$Z_1 = \frac{Dt_s F_{es}}{4K_q} \quad [4.27]$$

Equation 4.28 represents a mode III_s failure, which is fastener yielding in bending and bearing failure of the side member.

$$Z_2 = \frac{kDt_s F_{em}}{2.8(2 + R_e)K_q} \quad [4.28]$$

Equation 4.29 represents a mode IV failure, which is fastener yield in bending at shear plane without a bearing failure of the members. The yield modes for this case and others are depicted in Appendix I of the NDS-01.

$$Z_3 = \frac{D^2}{3K_q} \sqrt{\frac{1.75 F_{em} F_{yb}}{3(1 + R_e)}} \quad [4.29]$$

$$k = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{F_{yb}(2 + R_e)D^2}{2F_{em}t_s^2}} \quad [4.30]$$

$$R_e = \frac{F_{em}}{F_{es}} \quad [4.31]$$

$$F_{e(\text{parallel})} = 11200G \quad [4.32]$$

$$F_{e(\text{perpendicular})} = \frac{6100G^{1.45}}{\sqrt{D}} \quad [4.33]$$

$$K_q = 1 + \left(\frac{q_{\max}}{360^\circ} \right) \quad [4.34]$$

$$C_d = \frac{p}{8D} \leq 1.0 \quad [4.35]$$

$$Z' = Z_{\min} C_d \quad [4.36]$$

F_{em} = dowel bearing strength of main member, psi

F_{es} = dowel bearing strength of side member, psi

G = specific gravity of member

- p = penetration of lag screw into main member, in.
- C_d = penetration depth factor
- D = unthreaded shank diameter or root diameter of threaded portion when the threads extend into the shear plane, in.
- t_s = thickness of side member, in.
- F_{yb} = bending yield strength of lag screw, psi
- K_q = angle to grain coefficient
- θ_{max} = maximum angle of load to grain for any member in the connection, degrees ($0^\circ \leq \theta \leq 90^\circ$)
- Z' = reduced lateral design value, lb

Equation 4.33 was used to find the dowel bearing strength for both the side and main members. The maximum angle of the load to the grain was taken to be 90° . Equations 4.27 through 4.29 were used to calculate the allowable design value for the lag screw sizes tabulated in Table 4.6 when installed in the study connections. The penetration depth factor was calculated [Equation 4.35] and applied to the design value to find the reduced design value.

4.2.3 Results and Discussion

ASME standards for three lag screw nominal diameters are shown in Table 4.5. The dimensions of the purchased 5/16 in. and 3/8 in. screws conformed to the standards, but both 1/2 in. screws had shank diameters that were below the specified limits. Table 4.6 summarizes the results of the above calculations.

Table 4.6. Dimensions of lag screws as described in ASME B18.2.1-1996

Nominal Size	Shank		Root
	D_{min} , in	D_{max} , in	D_r , in
5/16"	0.298	0.324	0.227
3/8"	0.360	0.388	0.265
1/2"	0.482	0.515	0.371

Table 4.7. Lateral design values of lag screws with main member as shown and side member 2x_ southern pine ($G = 0.55$)

	Nominal Size	Effective Length L, in.	Shank Diameter D, in.	Root Diameter D_r , in.	Design Value, Z, lb (a),(b)			
					1-1/4" Timber-Strand, $G = 0.58$		2x_ SPF, $G = 0.42$	
					Without C_d	With C_d	Without C_d	With C_d
Actual	5/16 x 3.5"	3-1/4	0.300	0.227	136	94	120	99
	3/8 x 3.5"	3-1/4	0.364	0.278	168	94	148	100
	1/2 x 3.5"	3-1/8	0.481	0.376	282	(c)	251	(c)
	1/2 x 3.5"	3-1/8	0.480	0.383	287	(c)	259	(c)
NDS	5/16 x 3"	2-13/16	0.313	0.227	136	94	120	87
	3/8 x 3"	2-25/32	0.375	0.265	154	91	136	82
	1/2 x 3"	2-9/16	0.500	0.371	278	(c)	245	(c)

- (a) Values are based on the assumption that the ledger board (side member) is 2x_ Southern Pine ($G = 0.55$).
- (b) The Z value is the lowest of the three yield modes in NDS-01 equations 11.3-1 through 11.3-3
- (c) Penetration is below the limit of 4D and therefore is not allowed by the NDS-01

Without using the penetration factor, the lag screws in Timber-Strand® had higher design values, but once the factor was applied, the thicker SPF had higher values using 3.5 in. screws. The Timber-Strand® had a higher equivalent specific gravity than the SPF, therefore had a higher dowel bearing strength and a higher design value. However, the SPF was 1/4 in. thicker than the Timber-Strand® and therefore the SPF had a larger penetration depth factor. The 3.5 in. lag screws penetrated the entire thickness of both the Timber-Strand and the SPF, so the penetration factor reduced the value of the SPF less. With 3 in. screws, the entire effective length was embedded in the main member for SPF, but not for the Timber-Strand®. The penetration factors were closer for the 3 in. screws (for 3/8"x3" screws: $C_d = 0.59$ for Timber-Strand®, $C_d = 0.60$ for SPF) than for the 3.5 in. screws (for 5/16"x3.5" screws: $C_d = 0.69$ for Timber-Strand®, $C_d = 0.83$ for SPF). SPF was still reduced less, but the final result was that the screws had higher design values in Timber-Strand®.

The wet service factor, C_m , was also a concern. A wet service factor of 0.7 for lag screws applies to in-service lumber with a moisture content above 19% (NDS-01, Table 10.3.3). Application of C_m would reduce all design values in Table 4.6 by 30%.

4.2.4 Lag Screw Summary

The 3/8"x3.5" lag screws in SPF had the highest design value (100 lbs). The lowest design value was the 3/8"x3" screws in SPF (82.lbs). Trus-Joist literature recommended using 1/2 in. lag screws when attaching a 2x_ ledger

board to 1-1/4 in. Timber-Strand[®] LSL, and report a design value of 325 lbs/screw.

4.2.5 Recommendations

Based on a consultation with Drs. Dolan, Pollock, and Mr. Brad Douglas, it was concluded that the EYM equations for a bolted joint in single shear would be applicable to a lag screw connection that does not meet the 4D penetration limit of the lag screw provisions. When comparing the 1/2 in. lag screw allowable loads to allowable loads for 16d common nails, the nails come out with higher allowable loads, which means that, according to the lag screw equations, the nails are better than the lag screws. With the bolt equations, the allowable load is approximately 1.5 times the allowable load for the nails.

The Fairfax County, VA deck details required 1/2 in. lag screws. To meet the NDS requirements, these screws would need to penetrate into the main member at least 2 in.. This is not possible in the above situations.

4.3 Deck Attachment Issues

4.3.1 Introduction

Problems with the attachment of residential decks and balconies to houses are common. The purpose of this section is to analyze a common connection detail between the ledger board and band joist and to offer alternate details that will meet accepted structural design criteria for wood construction that would likely be imposed by a professional engineer.

4.3.2 Typical Deck Details

The Fairfax County, Virginia, Building Department has published Typical Deck Details at

<http://www.fairfaxcounty.gov/gov/dpwes/publications/deckdet.htm>.

The building code load requirements for residential decks are 40 psf live load and a dead load that accounts for the weight of the materials (about 10 psf). The live load required for a residential balcony is generally 60 psf. Unfortunately for the professional deck designer, the codes do not specify a lateral load requirement for lateral stability of the decking support system (joists, beams, and posts).

As an alternative to the details shown in many deck books and magazines, Figure 4.6 is a conceptual connection detail between the band joist and ledger board. In this connection, the reaction force of the deck joist is transmitted *directly* to the band joist by lag screws or bolts acting in shear. Note that the band joist and ledger board are in direct contact, separated only by the flashing. If insulation board is between the members, the strength of the lag (or bolt) connections is significantly reduced. If structural sheathing is between the two members, the strength is reduced to a lesser extent.

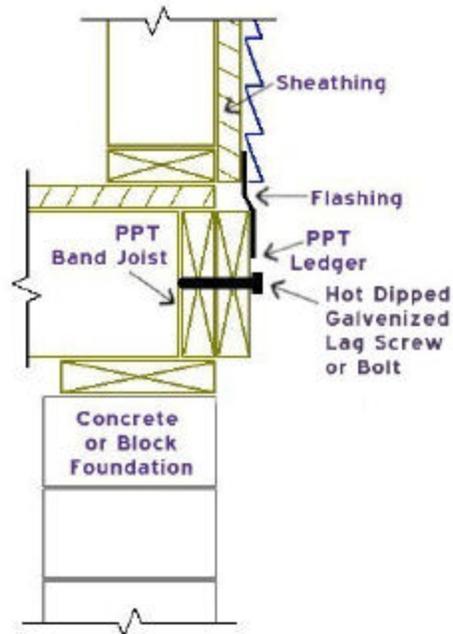


Figure 4.6. A deck detail of the deck-to-house connection shows a lag screw supporting the gravity load from the deck.

The flashing and preservative pressure treatment (PPT) band joist are the result of field studies of existing decks by Mr. Roger Robertson of the Chesterfield County (VA) Building Department. His field studies revealed decay of untreated perimeter bands and decay of the interior floor joist around the nails. Therefore, it is recommended that the band joist at the deck-house interface be PPT or equivalent and that flashing be installed between the band and the ledger board. He also observed the corrosion of aluminum flashing within five years when in contact with CCA treated lumber, therefore no longer preventing water from entering the structure. If aluminum is used, it is recommended that it be coated to prevent corrosion. One possible option is draping 15# felt paper over the flashing to separate the aluminum from the CCA lumber.

4.3.3 Lag Screw Shear Values

National Design Specification for Wood Construction (NDS-97) tables can not be used to calculate the allowable shear capacity of a 1/2 in. lag connection for two pieces of 2x_ material because the NDS tables are based on a penetration 8D (4 in. for 1/2" lag) into the main member (band) and a minimum of 4D (2 in.) for reduced design values. Thus, the question is: How much shear load can a 1/2 in. lag screw carry in a ledger/band application?

Because the point of a lag screw is not effective in load transfer, a 1/2"x3.5" lag screw was assumed to connect two pieces of 2x_ material with no sheathing or insulation between the two members. Assuming the ledger board was 2x_ Southern Pine (SP) and the band joist was 2x_ SPF, formulae provisions of the NDS-01 were used to calculate the allowable shear load per screw. The result was 180 lb per lag screw without any adjustment for the fact that the SP could have a moisture content greater than 19% which, theoretically, lessens the shear value. Using the 180 lb/screw allowable shear value, Table 4.8 was generated to determine the screw spacing for various joist spans.

Table 4.8. Required spacing* of 1/2"x3.5" lag screw connecting SP ledger to SPF band joist for residential deck joist spans (loaded by 40 psf live plus 10 psf dead load).

Joist Span (ft)	6	8	10	12	14	16	18
On-Center Spacing (in.)	14.4	10.8	8.6	7.2	6.2	5.4	4.8

*Values are based on the root diameter of lag screws purchased at a local building supply store.

At first glance, the lag screw spacings in Table 4.8 appear to be overly conservative. Common lag screw spacings for deck attachments are two to five times greater than the required spacings in Table 4.8. The spacing requirements in this table versus common field practice was noted by Christopher DeBlois, P. E., in *Practical Engineering* (JLC, March 1996): "What I am sure of, though, is that almost all the decks that I do inspect don't have enough bolts connecting the deck band joist to the house." So, the question is, "Why don't residential deck-to-house connections fail on a routine basis?"

4.3.3.1 Reason Number 1:

Assuming a 12'x18' residential deck, a 40 psf live load is equivalent to 58 people based on an average weight of 150 lb per person. In reality, this many people are not likely to gather at one time on most 12'x18' residential decks during the entire life span. (However, it is possible and thus the code contains the 40 psf live load requirement.)

4.3.3.2 Reason Number 2

The required spacing for the lag screws in Table 4.7 is based on an assumed uniform loading of the entire deck. However, large groups of people don't normally sit right next to the house. Instead of the uniform loading depicted

in Figure 4.7a, the occupancy loads on the deck are probably greater on the outside section (Figure 4.7b) and thus the outer supports are more heavily loaded compared to the house side.

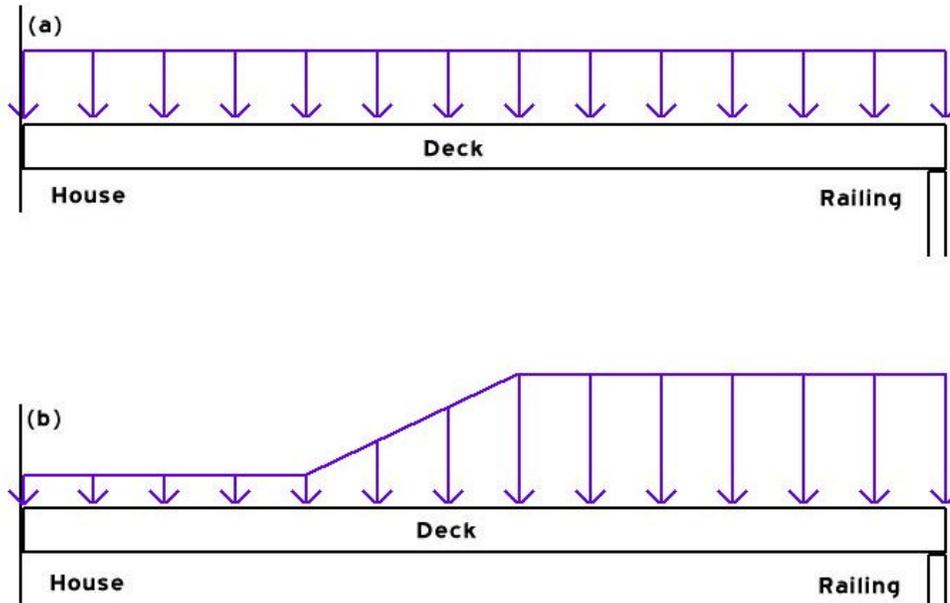


Figure 4.7. (a) Building codes require decks to be designed to carry a uniformly distributed load over the entire surface. Half of the assumed uniform load is carried by the deck-to-house connection; the other half is carried by the posts. (b) People tend to gather near the railings of the deck, and less load is likely carried by the deck-to-house connection.

4.3.3.3 Reason Number 3

Lag screw allowable shear values are based on code approved engineering standards. The safety factor on the allowable design value can be as high as 5.0 when tested in a laboratory. Thus, a perfectly installed 1/2 in. lag screw installed in a band joist/ledger application will typically carry a lot more than 180 lbs of load before the connection ruptures. However, the safety factor should not be encroached upon in design, as the purpose of a safety factor is to account for the many uncertainties of construction, service conditions, and design. For one example of construction uncertainty, it is very easy to drill a lead hole for the threads that is too large, thus compromising the strength of the connection (lead hole requirements are described in the NDS-01, Section 11.1.3). For a service example, referring to Figure 4.6, when the band joist is not

PPT or untreated heartwood of the PPT ledger is exposed to water that migrates along the length of the screw, decay around the lag screw can reduce the connection capacity. For an example of design uncertainty, when two screws are placed in a row in PPT material that is wet and connected to “dry” lumber (band joist), the PPT lumber can split when it dries and shrinks perpendicular-to-grain. (This case is addressed in the NDS-01 and the NDS requires a 60% reduction in lag screw shear values.) In most cases, lag screws are installed with good workmanship, the band joist and ledger lumber are not decayed, and the PPT lumber doesn't split due to two screws being aligned in a row, thus the safety factor on the screws prevent failures when the decks are lightly loaded in-service, probably 20 psf or less.

4.3.4 Engineered Decks

Many residential decks are not engineered per national design standards, yet an inspector may be called upon to certify that a deck (or balcony) is safe. In this section a conceptual detail is offered that will accommodate in-service inspection, utilizes more efficient connections, and has elements of structural redundancy.

Bearing reaction points that utilize only mechanical fasteners (bolts, screws, nails) are inherently inefficient compared to wood-to-wood bearing. The use of wood-to-wood bearing for vertical support is more efficient, relying on the perpendicular-to-grain compression strength of the lumber. In design, connections are at least as important as properly sized members, yet often in practice they are not given the same attention.

From an engineering and inspection point of view, a self-supporting deck is easier to inspect and to verify that the deck is safe for future service because all elements (except the footers) are exposed. Figure 4.8 shows one possible detail for this idea. The post is located next to the house and notched to receive the beams. The ends of the posts placed in the ground should not be cut, as it would expose untreated heartwood. Southern pine heartwood, as well as the heartwood of other softwood species, does not accept the penetration of the CCA chemical treatment, thus only the end surface contains the chemical. The heartwood of “naturally durable wood” is recognized by the model building codes as being equivalent to PPT, but it would certainly be more expensive than a PPT softwood such as southern pine.

For a longer post life, the use of 0.60 lb/ft³ (minimum) preservative retention is recommended by AWWA Standard C15-00 for “Sawn Building Poles and posts as structural members.” Many post-frame builders use the 0.60 lb/ft³ treated post product and they may be a local source of this material. (NFBA, 2002). The 6x6 posts that have been seen in the retail building supply centers are treated to the 0.40 lb/ft³ standard. All treated posts should bear the mark or tag of a third party inspection agency for the pressure treatment. Another post option is PPT Parallam[®] PSL, and engineered lumber product made by TrusJoist. The lowest CCA minimum retention shown at their website is 0.60 lb/ft³ (TruJoist MacMillan, 1998). The specific type of treatment should be considered by the deck designer in view of the fact that CCA is scheduled to be

phased out for some residential applications beginning in December 2003 (AWPI, 2002).

Trash, vegetation, or construction debris should not be placed in the post hole as it will compromise the lateral resistance of the embedded post section. It is suggested that the post hole be back-filled with an 80 lb bag of concrete mix, followed by well-compacted (at most 8 in. before tamping) native soil or sand and gravel mixture. The concrete above the footing pad will stabilize the bottom of the post in the unlikely event that the footing pad should rotate in-service. The size of the post footing pad and depth of post embedment for a design should be determined by the deck designer for the local climate (frost line) and soil strength, and local building code if applicable.

The deck joists are stacked on the built-up beams. This approach minimizes the reliance on mechanical connections to resist gravity loads (40 psf or 60 psf plus dead load). For lateral support, which is *extremely* important and not quantitatively addressed by the building codes (as stated earlier), galvanized thru bolts or rods are used to connect the post to the concrete foundation in two places, at the top of the post and about 12 in. above grade. This connection avoids penetrating the band joist, preventing the potential decay problems described earlier.

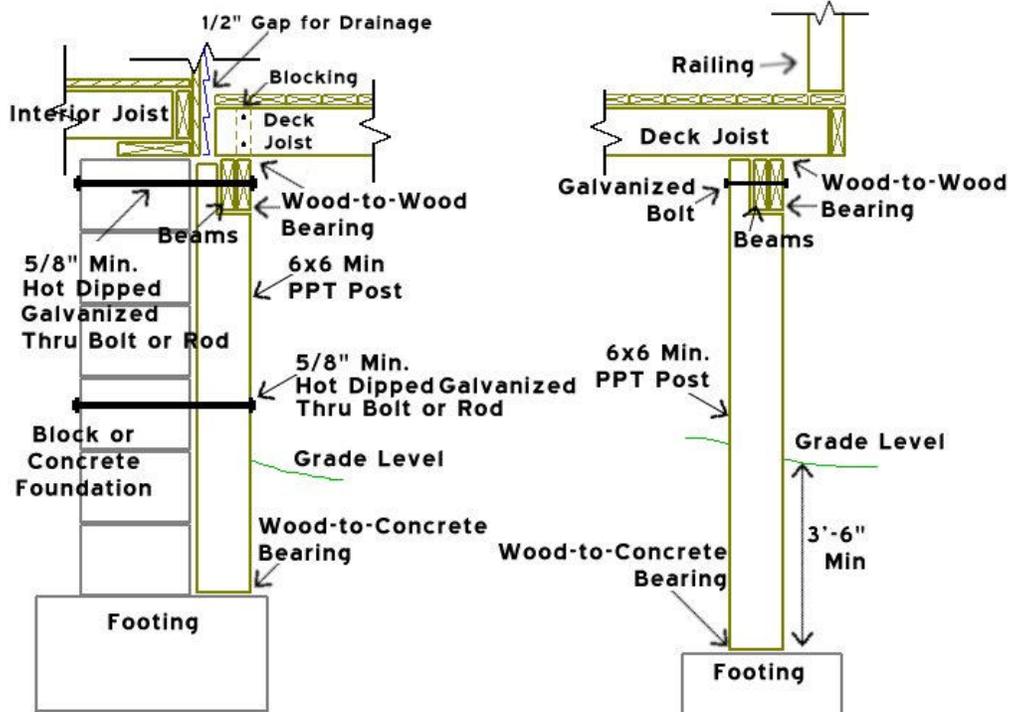


Figure 4.8. In this deck detail, wood-to-wood bearing or wood-to-concrete bearing is relied upon for vertical load transfer, rather than connections. The attachment to the house is for lateral support only. Hollow masonry must be reinforced by rebar and concrete.

The detail in Figure 4.8 demonstrates the design concept of redundancy. In this detail, the thru bolt prevents “sideway” of the deck that would occur if only the outside posts were embedded a minimum of 3’-6” in the ground. However, in the unlikely event that the thru bolts should fail due to corrosion or any reason, the embedded PPT 6x6 posts would prevent a lateral collapse of the entire deck. Thus, the detail in Figure 4.8 has a “fail safe” feature, or is redundant, in that possible failure of one element (the thru bolts) should not cause massive collapse of the entire deck. It is possible for the deck without the thru bolts to move laterally an inch or more, but this amount of movement should not result in collapse or personal injury.

4.3.5 Inspection

When an engineer or home inspection professional is engaged to verify the adequacy of a deck, their job is to establish that the deck being inspected is unequivocally safe for future use. A statement by the engineer that the deck is “probably safe” is not sufficient. A self-supporting deck can be verified for with ease at a later date, decades after the original construction.

4.3.6 Conclusion

Two approaches to deck support at the house interface have been reviewed. Ledger board attachment is very common, but it is difficult to inspect for code conformance and to verify that the elements have not been degraded in-service. Since decks are subjected to both code specified gravity type loads (40 psf plus dead) and unspecified lateral loads, a conceptual detail was proposed in Figure 4.8 that addresses both loading directions - vertical and horizontal. The detail has redundancy features whereby the possible failure of one element will not automatically produce or permit collapse of the entire structure. The detail also eliminates the need to penetrate the house siding, sheathing, and band joist, thus eliminating the decay hazard for the house elements.

Deck-to-house or balcony-to-house connection details that accommodate professional inspection of in-service decks are also good details for homeowners. The owner and contractor, after careful consideration, are left to choose between the two general deck-to-house attachment methods.

4.4 The Pick Test

4.4.1 Introduction

From a structural engineer's point of view, the design and construction is only valid for future service if the material is in its original condition, free of degradation. There are several factors that degrade the state of the lumber used, including fungal decay. The "pick test," described below, is based on toughness and has been proven to detect decay with as little as 5 to 10% weight loss (Wilcox, 1983).

4.4.2 Objective

To introduce and demonstrate the use of the "pick test" as a tool for evaluating the condition of lumber and timbers that may contain early stages of fungal decay.

4.4.3 Decay Detection using the Pick Test

Fungal decay is common in areas near fasteners, joints, checks, end grain, paint discoloration and where lumber and timbers are near or in contact with soil. The "pick test" uses an ice-pick tool to penetrate the wood surface. Other tools such as an awl or even a small screw-driver can also be used. After penetrating the wood, the tool is rotated to pry a splinter, parallel to the grain, away from the surface. The appearance of the broken splinter is used to determine if the piece is decayed. Since different species have different densities and all lumber is affected by its environment, trying the pick test in an

area where the wood is known to be sound would be a way to determine a “control” for the rest of the inspection. The test should be conducted in a late-wood zone (the darker, thinner growth rings), although the test also may work in early wood zones. The testing should begin in areas that are conducive to fungal decay, noting how much pressure is required to penetrate the surface. The depth should be about 1/4 in. A small amount of the surface wood should be pried out and compared to the results of the non-decayed wood.

Wilcox (1983) identified three distinct modes of failure for decayed and non-decayed wood. Non-decayed wood will generally fail with either a fibrous failure or a splintering break as shown in Figure 4.9 and 4.11. Decayed wood will have a brash, brittle failure with breaks directly over the tool. Very few splinters, if any, will appear and the break will be across the grain as shown in Figures 4.10 and 4.12. Figure 4.9 and 4.10 were taken in a salvaged Douglas Fir log yard. In the Figure 4.9 and 4.10 examples, the wood is weathered, and to the inexperienced eye they look the same.



Figure 4.9. The sound wood broke in a solid piece, and far from the tool. It was difficult to penetrate deeply. One end did not break at all. The wood under the splinter is intact and looks new.



Figure 4.10. The decayed wood broke easily; the break is across the grain with no splinters.

In a fibrous failure, the splinters are long and separate out of the surface far from the tool as shown in Figure 4.11 (Virgin Douglas Fir). A splintering break typically occurs directly over the tool with numerous splinters. It is possible that the wood is very dense and in such good condition that penetration is difficult. Also noticeable with sound wood is the noise associated with the break. In non-decayed wood, the sound will be as expected when wood breaks. However, with decayed wood, the breaking noise will not be as loud, or there may be almost no audible sound.



Figure 4.11. This example is a block of virgin Douglas Fir with no decay. It shows a splintered break that begins far from the penetration.



Figure 4.12. On the surface, this 50-year-old Douglas Fir purlin looked sound, however with the pick test, decay is indicated by a brittle cross grain break directly over the tool. The entire break is less than one inch long.

4.4.4 Conclusions

The pick test is a simple, subjective test that is useful to detect decay near the surfaces of wood members. With experience, the user will be able to identify fungal decay more readily and detect the subtle differences between the decayed and non-decayed areas. For inspections where only knowledge of the presence of decay is needed, such as residential decks, the pick test is useful. For the case of a residential deck or balcony, we recommend wooden members be replaced if any decay is detected. If the lumber is not pressure preservative treated (PPT), this fact should be considered when making the decision to replace the elements.

5. Full Inspection of Residential Deck

5.1 Introduction

A residential deck built in 1988 was selected for this inspection. The full history of this deck was known before the inspection took place and the homeowner was willing to have parts of the deck temporarily removed. A full inspection was conducted and the structural elements were checked for conformance with the NDS-01 and the IRC-2000, assumed to be applicable for this inspection.

5.2 Objectives

1. To thoroughly examine a residential deck, with full documentation and report.
2. To gain insight into the inspection process.
3. To determine if the inspected deck is in accordance with the 2000 International Residential Code for one-and two-family dwellings and the 2001 National Design Specification for Wood Construction.

5.3 Materials and Methods

5.3.1 Preparation

The homeowner was questioned about the history of the deck, including age, the presence of plans and any repairs that had been made.

5.3.2 Plan View and Typical Sections

The plan view of the deck was created first, as a way for the inspector to become familiar with the deck layout and identify areas that may need closer scrutiny. All railing posts, deck posts, beams, joists, stairways and major dimensions were shown on the drawings.

A typical railing section was drawn for future analyses. It included details of the railing posts, infill, and connections. The stairway detail included risers, railing, and stringers.

5.3.3 Railing

Information gathered on the deck railing was the height of the railing above the deck, materials, how the railing was attached to the deck (including hardware) and a thorough inspection of each rail post and the infill areas between the posts. Components were checked from both the deck side and from the ground side (when possible). The height was measured from the top of the deck boards to the top of the cap rail. The rail posts were checked for decay, insect holes, splits and construction errors. The connection of each rail post to the deck was also assessed. Infill pickets were checked for loose elements and their connection to the deck. Any findings were documented in tables.

For the structural analysis, a typical 16 in. section of the infill was selected and photographed as well as measured. One lag screw and one nail were removed for examination, and then replaced. For practicality, all other

connections of the same type were assumed to be in the same condition based on a comparison of the visible parts of the hardware.

5.3.4 Decking

The deck board material, size, and treatments were all recorded. The condition of the deck boards was noted, as well as the type of connection to the joists. A level was used to check that the structure was level. The nails were checked for pullout and actual attachment to the joists.

5.3.4 Joists

Grade stamps were located to document the species and grade of the joists. Their attachments to the ledger board, beam and perimeter joists were also noted. The spans, sizes and overhangs were all recorded in a table. Also recorded was the overall condition of each joist and the condition of all connections.

5.3.5 Beams/Girders

Beams were checked for species and grade. The type of attachment to the deck posts was noted. Built up beams were examined for the nailing pattern and edge distances for the nails. Spans, sizes and overall condition of each beam span were recorded in a table. The tightness of the connections, or presence of gaps, was also observed and noted.

5.3.6 Posts

The slope of the ground under the deck was noted, as well as the condition of the ground. Post species, grade and treatment were all recorded. The sizes, heights (from ground to the beam) and overall conditions the posts were logged into a table. At the foot of one typical post, the soil was removed to a 6 in. depth and the post was checked (using the pick test (Anderson et al., 2002)) for decay.

5.3.7 Stairways

The stringer type, presence of handrails, railing height, dimensions and materials of the stairway were all recorded. Dimensions included the total rise, total run, tread width, each rise and nosing. Each riser was examined for decay and other hazards. The riser details were recorded in a table. Other stairways on the deck were also examined and included in the table.

The stringers were each checked and their condition was also recorded.

5.3.8 Lateral Support/Bracing

Lateral support for the deck was provided by the attachment to the house and the embedment of the 6x6 posts. Depth of embedment was not determined.

5.3.9 Attachment at Ledger

The species, grade, size and treatment of the ledger board were all recorded. Areas around the ledger were all examined closely to determine how

the ledger was attached to the house and what materials were present between the ledger and the band joist. Photographs were taken of the material and a magnet was used to determine if the flashing material was galvanized steel or aluminum.

From the inside of the house, the material of the band joist and the penetration of the lag screws were determined. The spacing and pattern of the lag screws were noted. One lag screw was removed and the condition and measurements were noted (especially at the shear plane). The wood fibers surrounding the hole after the lag screw was removed were also inspected for decay. After photographing, the lag screw was replaced. Other lag screws were assumed to be in the same condition as the typical lag screw based on the size and condition of the heads.

5.3.10 Other

The condition of the decorative trim around the deck was also noted.

All structural elements were assigned a decay category based on inspection methods described by Eslin et al. (1979). Category 1 is existing decay with severe strength loss, requiring immediate repair and restricted use. Category 2 is existing decay without any limitations on use, but maintenance is needed. Category 3 is conducive to decay and preventative maintenance is needed in that area. Category 4 is no decay present.

Any areas that could not be seen without taking the deck apart were noted so the inspector could disclaim responsibility for concealed parts not possible to

inspect, and at the same time, communicate to the owner (client) the limitations of a non-evasive inspection.

5.4 Results

5.4.1 Preparation

The homeowner informed the inspector that the deck was constructed in 1988 with the new construction of the house by a local building contractor. Since then, the homeowner has added lag screws at a few areas that seemed weak and at splits in the rail posts. He had not checked the tightness of the bolts or lag screws since the deck was built.

5.4.2 Plan view and typical sections

The deck had three main sections at different levels. The plan view with locations of beams, joists, deck posts and rail posts can be found in Section 5.6.1. A typical rail section and stairway section are also located in this section.

5.4.3 Railing

The railing measured 36 in. above the deck boards. The infill pickets were nominal 2x2 Western Cedar posts 8 in. on-center attached to a 2x4 top rail and the perimeter joist. The rail posts were attached to the top rail by toenailing; the infill pickets were attached to the top rail with one nail each. The cap rail was 2x6 and protected all railing posts from end grain exposure. Rail posts were 2x4 Western Cedar attached to the perimeter joist by notching the posts (1-3/4 in. by

11-1/2 in.) and using nominal 1/4"x3" lag screws and 16d annular threaded nails in two different configurations, shown in Figure 5.1.

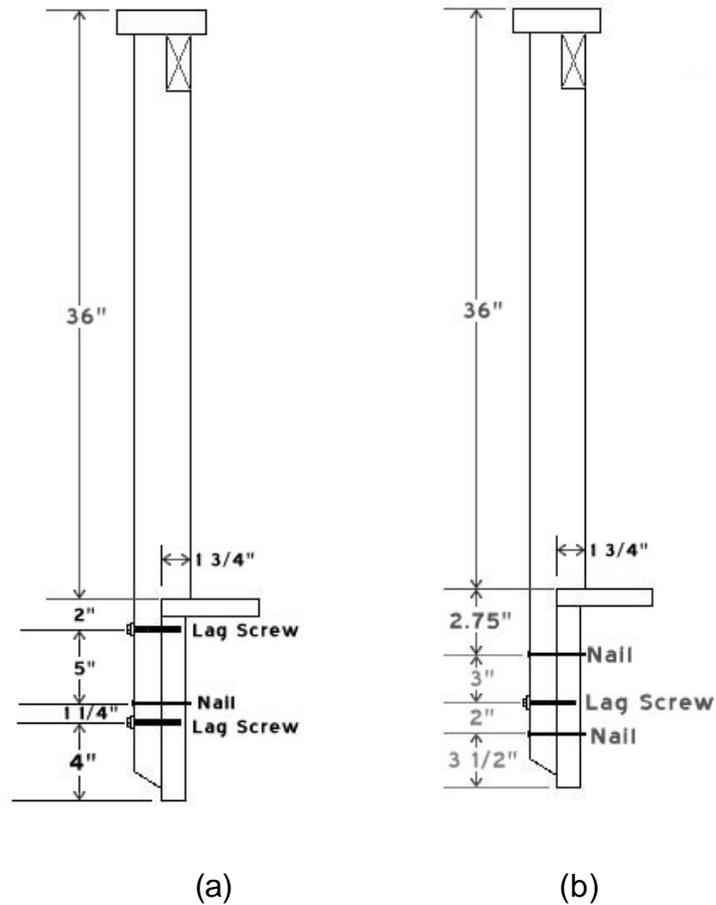


Figure 5.1. Section views of the rail post attachment to the deck: (a) Connection type A is two 1/4"x3" lag screws and one 16d annular threaded nail (b) Connection type B is one two nails and one lag screw.

One lag screw was removed and it had a 0.234 in. shank diameter and 0.175 in. root diameter. The effective length of the lag screw was 2.75 in. Discoloration was seen in the shear plane. The nail was 16d annular threaded with a shank diameter of 0.150 in. The root diameter was 0.140 in. at the unaffected areas and 0.139 in. at the discolored area. Photographs of the nail and lag screw are shown in Figures 5.2 and 5.3.



(a)



(b)

Figure 5.2. The nail removed from a rail post (a) before it was removed and (b) when it was measured.



Figure 5.3. The lag screw that was removed from the rail post. There is noticeable discoloration at the screw head and the interface between the post and the perimeter joist.

Tables 5.1 and 5.2 give the results of the inspection process on the rail posts and infill sections. The type of connection is identified as A or B with A being two lag screws and one nail, and B being two nails and one lag screw. The NDS-01 specifies allowable notch depth in bending members at the bearing as less than one-quarter of the depth of the member. The notch depth in the rail posts is one-half of the depth, and the rail posts are not in conformance with the NDS-01. Splits at the notches were a concern and were measured. Splits and kerfs are shown in Figure 5.4.



(a)



(b)

Figure 5.4. (a) The kerf on railpost #12 (b) The split in railpost #14 and the lag screw added by the homeowner.

The section of railing selected for the structural analysis unit was 16 in. in width and included two infill posts as shown in Figure 5.5. Calculations (shown in Section 9.1.2.2) were performed to determine the strength of the railing in this area. The IRC-2000 states that a railing must withstand a concentrated load of 200 lbs in any direction along the top of the rail. Other building codes required that the railing resist a distributed force of 50 lb/ft and the infill must be able to carry a 50 lb load over 1 ft².



Figure 5.5. The rail analysis unit was two typical infill posts.

Using the NDS-01, a lag screw in the picket safely resists 115 lb in withdrawal and 95 lb in shear. The allowable withdrawal strength of the nail (assuming hardened steel) is 28 lb and the allowable lateral resistance value (shear) is 99 lb.

Common design loads in the model building codes are a concentrated load applied at the cap rail in any direction and/or a uniform load on the cap rail in any direction. These two loads are not applied at the same time. To avoid difficult analysis, the concentrated load was checked at the rail posts and the uniform load was checked on the cap rail over the infill.

The 50 lb/ft (4.2 lb/in.) distributed load is distributed by the cap rail to the infill posts, placed 8 in. on center. When a force is exerted outward, the top fastener of each infill post must be able to resist 252 lb. When the force is

downward, the fastener must resist 34 lb in shear. The railing is not adequate to hold the 50 lb/ft load when it is applied horizontally and outward.

The cap rail also distributes the 200 lb concentrated force. When a force of about 50 lb was applied by the hand of an inspection team member horizontally outward on the cap rail, the railing deflected more than 1 in. The railing was stiffer when a vertical load and an inward horizontal load was applied. Based on the deflection under a 50 lb load, it was determined that the railing could not safely withstand the 200 lb concentrated load in any direction as required by the IRC-2000 building code.

Within the infill area, a 50 lb force over 1 ft² causes a maximum of 50 lb on 1 ft of one post. Calculations (Section 9.1.2.2) show that the infill pickets can adequately carry this load.

5.4.4 Decking

The decking was 5/4x6 Western Cedar attached to the joists using two 2.5 in. annular threaded nails. A small amount of iron stain was noticed around the nail heads. Some nails were missing on a few deck boards. In one area, nails were missing, but the deck boards were well secured to the joists by other nails. A level was used to show that there was no detectable deflection or sagging of the structure. Decay was found in a few deck boards, as shown in Figure 5.6, where the end grain was exposed to weather. It was limited to a small section of sapwood. Mildew was present on the deck and a few knotholes were found. On the underside of the deck boards, a white substance was seen, probably mold.



Figure 5.6. Sapwood decay was present at the end-grain of deck boards.

5.4.5 Joists

The joists were 2x8 No. 2 Southern Pine. They were attached to the ledger with four toenails and rested on a 2x2 support. The 2x2s were nailed to the ledger at 8 in. on center. Nail size was not determined as the 2x2 support was not relied upon in any engineering calculations. The 2x2's were probably not stress rated material and they were not included in the analysis of the connection. Assuming the toenails were 16d common nails, each can support 89 lbs laterally and the connection must support 320 lb. Joists were stacked over the beam and toenailed to the beam. The perimeter joist was attached to the joists with alternating TECO-11-GRIP type 28 hangers and nails. Where the joists met the beams at a diagonal, they were nailed.

Each joist was checked over and noted in Table 5.3. One-half of joist No. 30 was supported by beam No. 4 as shown in Figure 5.7.



Figure 5.7. One half of joist No.30 was supported by the beam, which was probably a construction error and not intended.

Using calculations, as shown in Section 9.1.2.3, the spans of the joists were checked in shear, bending and deflection. All joists were found to be adequate for carrying loads as required by code, including joist No. 30.

5.4.6 Beams/girders

Built-up beams (beam spans 1 through 4) were 2-2x12 No. 2 Southern Pine nailed together with two nails roughly spaced at 2 ft on-center with a single nail located near the middle of the space. The posts were notched for the beams and attached with two 1/2 in. machine bolts. The beams were notched at the posts (no more than 1/4 in.) to make up for the top of the posts not being at the correct elevation during installation. Other beams were 2x12 No. 1 Western Cedar and 2x8 No. 2 Southern Pine. Figure 5.8 shows how the carriage bolts were rusted around the edges and crushed the wood around them.



Figure 5.8. Carriage bolts, which are not recognized by the NDS rusted around the edges and crushed the wood around them.

Specifics of each beam are shown in Table 5.4. Beams were checked using methods from NDS-01 in bending shear and deflection using the same methods as the joists. Beam No. 10 failed in the bending stress check. However, beam No.10 is paired with a No. 1 2x12 Western Cedar perimeter board, nailed with three 16d threaded nails 24 in. on center. With this added strength, the beam is adequate.

5.4.7 Posts

The ground below the deck sloped sufficiently away from the house and was dry at the time of inspection. All posts were No. 2 Southern Pine, CCA pressure-preservative-treated. The amount of treatment could not be determined, but was deemed to be effective after a pick test. The pick test was performed 6 in. below the ground surface at post No. 5 and revealed no evidence of decay as shown in Figure 5.9.



Figure 5.9. Digging 6 in. below the ground surface and using the pick test showed no evidence of decay on Post No. 5.

Heights and conditions of posts are shown in Table 5.5. Each post was checked for adequacy according to the NDS-01 requirements and sample calculations are shown in Section 9.1.2.4.

5.4.8 Stairways

Solid stringers (2x12 Cedar) made up the outside of the stairways with a cut stinger in the center. The seven risers were 36.5 in. wide with an 11.25 in. tread depth, 7.5 in. rise and 1.5 in. nose. The total rise of the stairs was 60 in. and the total run was 6'-3". Each riser was made with two 2x6s with a 1/4 in. space between them. Risers were placed into a 3/4 in. notch in the solid stringers. Conditions of each riser and the stringers are shown in Table 5.6.

Each riser was checked and determined to be in good condition. Some white mold was found on the underside of the risers, near the stringers. One

riser had a 1.75 in. diameter knothole that could cause a fall accident for a person wearing “high heels”. Also, artificial lighting was not present around the stairway as required by the IRC-2000.

Two other sets of two steps were also checked and found to be code conforming.

5.4.9 Lateral Support/Bracing

Lateral support for the deck was provided by the attachment of the ledger board to the house frame.

5.4.10 Attachment at Ledger

The ledger board was a PPT 2x10 No. 2 Southern Pine. It was attached to the house framing with 1/2"x4" nominal lag screws placed 24 in. on center. At the ends and butt joints, two lag screws were present. In all other places the lag screws were placed alternating top and bottom. Aluminum flashing and 1/2 in. insulation were placed between the ledger and the band joist, as shown in Figure 5.10. From the inside of the house, the lag screws were found to penetrate fully into the band joist with the pointed end coming completely through the joist. The band joist was untreated Spruce-Pine-Fir.



Figure 5.10. Material between the ledger and the band joist was observed at a vent. From right to left the materials are the band joist, 1/2" insulation, ledger and 2x2 support for joists.

One lag screw was removed and measured. The shank diameter was 0.481 in. and the root diameter was 0.372 in. The lag screw and the surrounding wood were in excellent condition. The ledger board could not be removed and, as a result, the condition of the aluminum flashing is unknown.

Figure 5.11 shows the lag screw as viewed from inside the house and once it was removed from the ledger.



(a)



(b)

Figure 5.11. (a) The ledger screw (nominal 1/2"x4") was long enough for the pointed end to stick out of the band joist. (b) The shiny area on the ledger screw is where it penetrated the insulation.

As constructed, each lag screw must support 480 lb in shear or lateral loading. The penetration of the lag screw into the band joist was the full 1.5 in., which exceeds the 4D minimum ($4 \times 0.372 \text{ in.} = 1.49 \text{ in.}$) required for the use of the NDS-01 tables. The lateral strength of the lag screw was found using the yield equations in TR-12 (AF&PA, 1999). The analysis included the 1/2 in. gap (for insulation) between the ledger and the band joist. The design value for a 1/2 in. lag screw in this case was 120 lbs.

5.4.11 Other

The decorative cedar trim around the deck was found to be pulling out in some areas. One area, in place to protect a post from end-grain exposure, had decay present.

5.5 Conclusion

5.5.1 Conformance with NDS-01

Structural elements were checked by the methods described in the NDS-01. The structural checks included bending stress, shear stress, deflection and axial compression stress in posts. All members were adequate, except beam No.10, which was discussed in Section 5.4.6. The lag screws in the ledger are not adequate to support the gravity design loads. The lag screws and nails in the railing are also insufficient.

Carriage bolts, used in connecting beams to posts, are not recognized by the NDS. Section 11.1.2.3 of the NDS-01 requires washers on both sides of the connection. The notches on the railing posts are greater than the allowable sizes as described in NDS-01, Section 5.4.3.

5.5.2 Code conformance per the IRC-2000

The 200 lb concentrated load requirement of the IRC-2000 could not be proven for the railing construction. Also, the 50 lb/ft railing load could not be backed up by calculations for the construction. The 50 lb/ft load is not required by the IRC-2000, but it was evaluated for demonstration purposes. The infill

pickets were placed 8 in. on center, leaving 6-1/2 in. open in between them, which is greater than the 4 in. maximum as required by the IRC-2000. At the stairway, the railing was 31 in. above the riser at some points, which is less than the minimum 36 in. The openings behind the stairs were greater than 4 in. and no blocking was provided. Also, at the stairway, an artificial light source was not present, thus the stairway is not code conforming.

5.5.3 Other concerns

Insect holes were found on some deck elements, but infestations were not a problem.

Small knotholes in the deck boards could cause a person to trip, and the hole on stairway riser No. 5 (from the bottom) was large enough to be a safety hazard.

Splits in the rail posts are a problem that could “grow” with time. The ability of those posts to withstand the loads dictated by building codes is less than predicted by calculations that assume all elements are in good condition.

5.6 Appendix to Full Deck Report

5.6.1 Deck Plans and Drawing

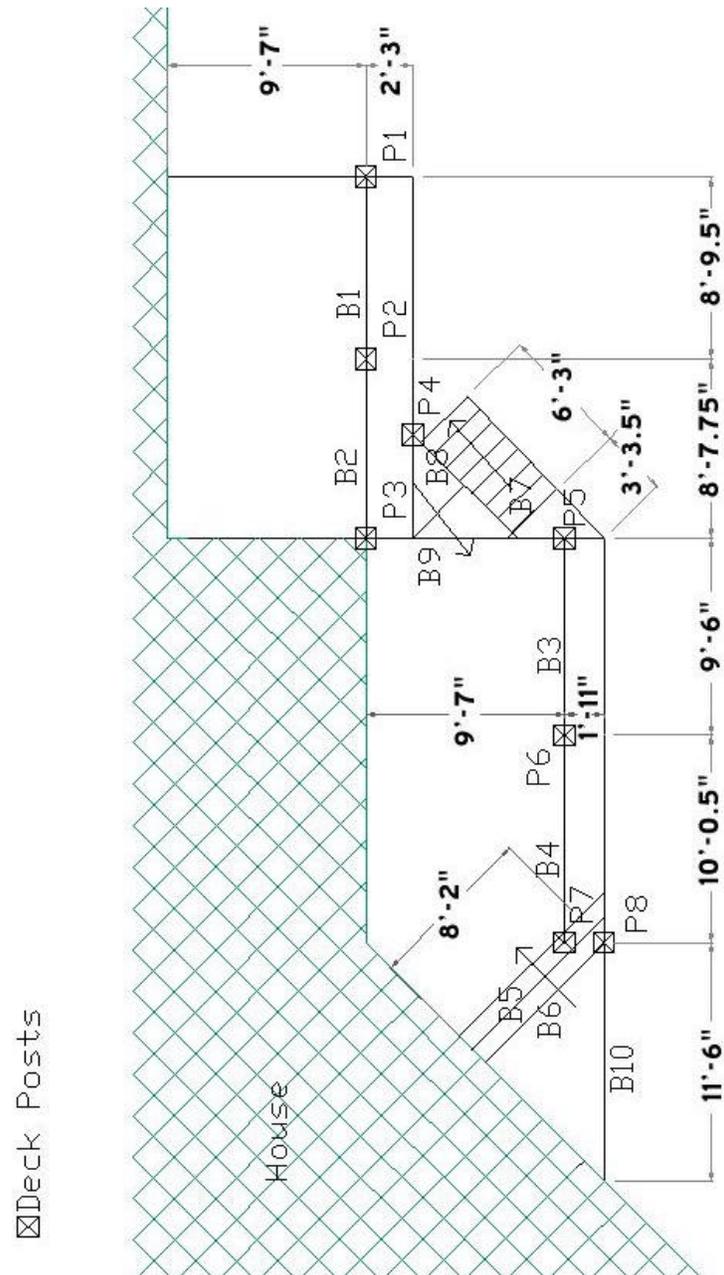


Figure 5.12. (a) Plan view of deck including beam and post numbering

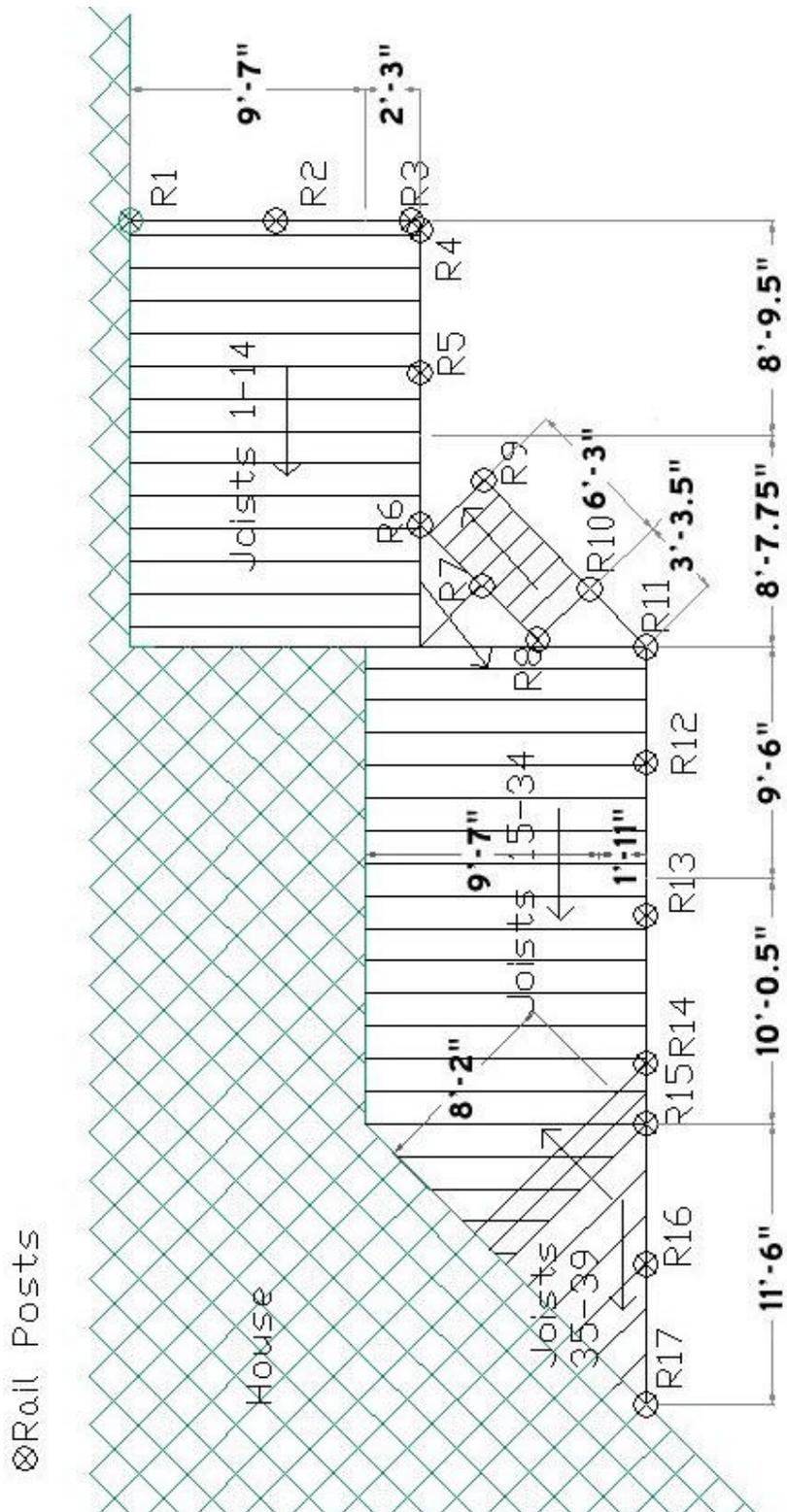


Figure 5.12. (b) Plan view of deck including joist and rail post numbering.

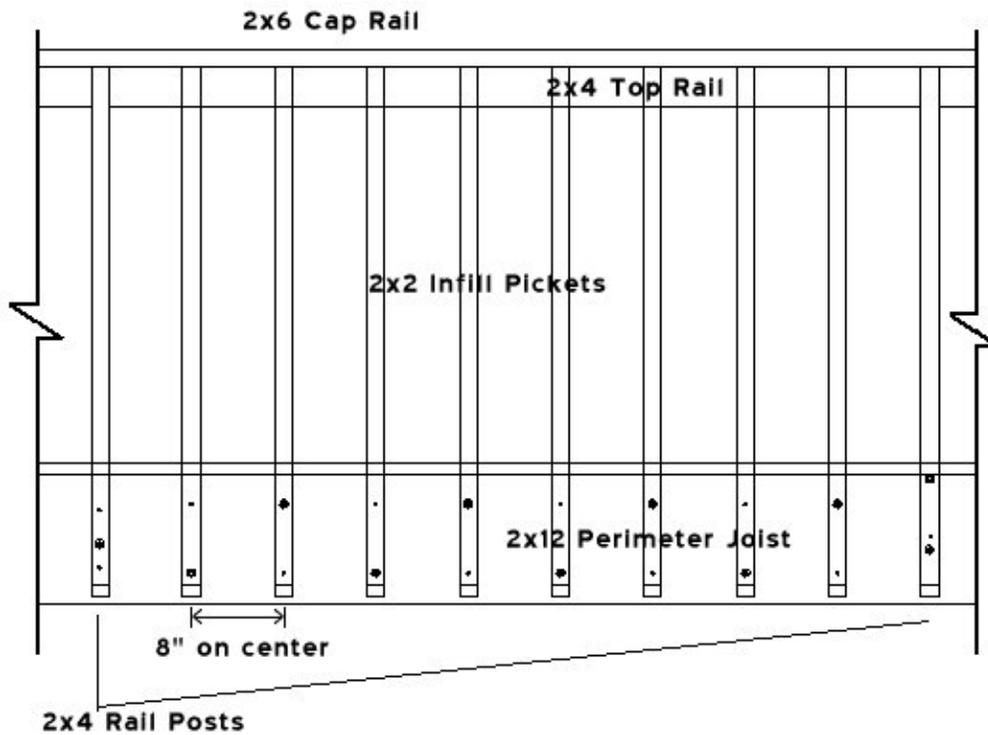


Figure 5.13. Typical section of railing.

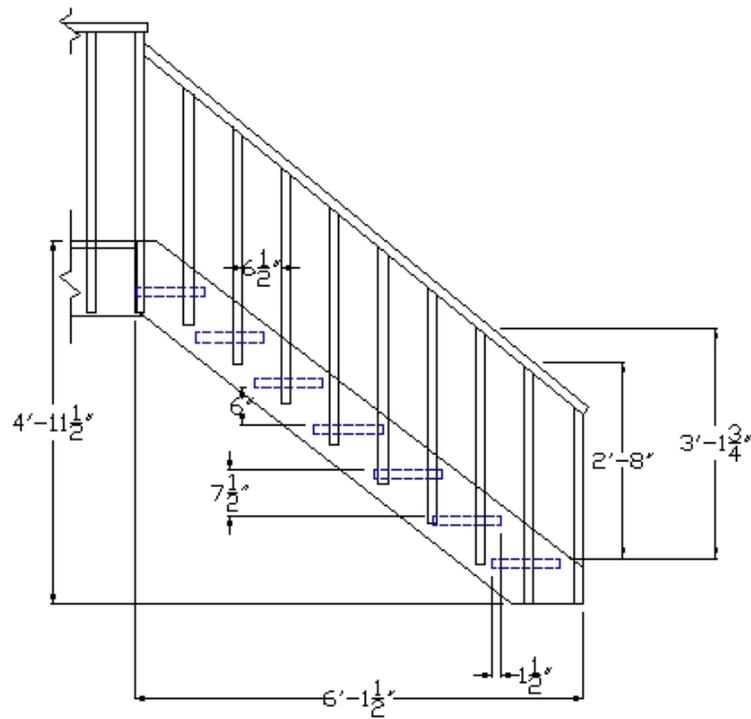


Figure 5.14. Side view of stairway.

5.6.2 Inspection Tables

Table 5.1. Size and condition of railing posts and the deck-post connection.

Railing Post Number	Size	Post Condition	Decay Category	Post attachment to deck	
				Condition of Post at Connection	Condition of Fastener*
1	2x4		4	5" split, continues to top	Type B
2	2x4		4		Type A
3	2x4	Insect holes	4		Type B
4	2x4		4	8"split	Type A
5	2x4		4	Vertical 3/4" kerf at notch, 3/4" check, continues to top	Type A
6	2x4		4		Type A
7	2x4		4	2" split, continues to top	Type B
8	2x4		4		3 screws
9	2x4	Contact with ground	2	Horizontal 1/4" kerf, check to top	Type A with screw at split
10	2x4		4	6.25" split at notch, continues to top	Type A
11	2x4		4	2" split at notch	Type B
12	2x4		4	Vertical 1.5" kerf on one side	Type A
13	2x4		4	28" check at notch	Type B
14	2x4		4	Split closed with screw	Type A with screw at split
15	2x4		4		Type A, no washer
16	2x4		4		Type B, no washer
17	2x4	One Side Hidden	4		Type A, no washer

*Connection type A is two 1/4"x3" lag screws and one 16d annular threaded nail. Connection type B is one two nails and one lag screw

Table 5.2. Condition of railing infill

Infill Between Post Numbers	Condition of infill	Infill Attachment to posts	Cap Rail	Top/Bottom Rail
1-2	Some mildew	N/A		Knots
2-3				
4-5	Knot holes, mildew			
5-6	Insect holes			
6-7				
7-8	Insect holes			
9-10	Some loose			
10-11		Not inspected from outside because of height		
11-12		Not inspected from outside because of height		
12-13		Not inspected from outside		Bee hole
13-14		Not inspected from outside because of height		
14-15	Very Flexible	Not inspected from outside because of height		
15-16		Not inspected from outside because of height		

Table 5.3. Size, span and condition of joists and joist fasteners.

Joist Number	Size	Span	Overhang	Joist Condition	Decay Category	Attachment to Ledger	Attachment to Beam/Girder	Meets NDS-01 Design Criteria
1	2x8	9'-7"	27"		4			OK
2	2x8	9'-7"	27"		4		Toenails are slightly rusted	OK
3	2x8	9'-7"	27"		4			OK
4	2x8	9'-7"	27"		4			OK
5	2x8	9'-7"	27"		4			OK
6	2x8	9'-7"	27"		4			OK
7	2x8	9'-7"	27"		4			OK
8	2x8	9'-7"	27"		4			OK
9	2x8	9'-7"	27"		4			OK
10	2x8	9'-7"	27"		4			OK
11	2x8	9'-7"	27"		4	Spacer at bottom		OK
12	2x8	9'-7"	27"		4			OK
13	2x8	9'-7"	27"		4			OK
14	2x8	9'-7"	27"	Attached to house, flashing is covering band joist	4			OK
15	2x12	6'	23"	No. 1 Cedar, Also Beam #9	4		Notch 4" deep to fit around beam span #3, 1/4" kerf	OK

*Criteria checked according to the methods described in NDS-01 are: bending stress, shear stress and deflection

Table 5.3. Size, span and condition of joists and joist fasteners (continued)

Joist Number	Size	Span	Overhang	Joist Condition	Decay Category	Attachment to Ledger	Attachment to Beam/Girder	Meets NDS-01 Design Criteria
16	2x8	9'-7"	23"		4			OK
17	2x8	9'-7"	23"		4			OK
18	2x8	9'-7"	23"		4			OK
19	2x8	9'-7"	23"		4			OK
20	2x8	9'-7"	23"		4			OK
21	2x8	9'-7"	23"		4			OK
22	2x8	9'-7"	23"		4			OK
23	2x8	9'-7"	23"		4			OK
24	2x8	9'-7"	23"		4			OK
25	2x8	9'-7"	23"		4			OK
26	2x8	9'-7"	23"		4			OK
27	2x8	9'-7"	23"		4			OK
28	2x8	9'-7"	23"		4			OK
29	2x8	9'-7"	23"		4	Check near ledger, not decay		OK
30	2x8	9'-7"	23"	Mildew	4		1/2 of joist is on beam	OK

*Criteria checked according to the methods described in NDS-01 are: bending stress, shear stress and deflection

Table 5.3. Size, span and condition of joists and joist fasteners (continued)

Joist Number	Size	Span	Overhang	Joist Condition	Decay Category	Attachment to Ledger	Attachment to Beam/Girder	Meets NDS-01 Design Criteria
31	2x8	9'-1"	0	White Mold	4		Toenailed to beam #6	OK
32	2x8	6'-5 1/4"	0		4			OK
33	2x8	3'-10"	0		4			OK
34	2x8	1'-2 1/2"	0		4			OK
35	2x8	6'-10 1/4"	0	Hidden	4			OK
36	2x8	5'-6 1/2"	0	Hidden	4			OK
37	2x8	4'-3"	0	Hidden	4			OK
38	2x8	2'-11 1/4"	0	Hidden	4			OK
39	2x8	1'-7 1/2"	0	Hidden	4			OK

*Criteria checked according to the methods described in NDS-01 are: bending stress, shear stress and deflection

Table 5.4. Size, span, and condition of beams and beam fasteners.

Beam Span Number	Size	Span	Overhang	Species	Beam Condition	Decay Category	Attachment to Post	Attachment to Post	Meets NDS-01 Design Criteria
1	2-2x12	8'-9 1/2"	0	No. 2 S.P.		4	1	2	OK
2	2-2x12	8'-7 3/4"	0	No. 2 S.P.		4	2	3	OK
3	2-2x12	9'-6"	0	No. 2 S.P.		4	5	6	OK
4	2-2x12	10'-1/2"	0	No. 2 S.P.		4	6	7	OK
5	2x12	8'-2"	0	No. 1 Cedar		4	Ledger	8	OK
6	2x8	8'-2"	0	No. 2 S.P.		4	Ledger	8	OK
7	2x12	38"	0	No. 1 Cedar	Large Crack on front face. Not decay	4	Beam 9	Stringer 3	OK
8	2x12	6'-11"	0	No. 1 Cedar		4	4	Beam 9	OK
9	2x12	9'-7"	25"	No. 1 Cedar		4	Ledger	5	OK
10	2x8	11'-6"	0	No. 2 S.P.		4	Ledger	8	OK

*Criteria checked according to the methods described in NDS-01 are: bending stress, shear stress and deflection

Table 5.5. Size and condition of deck posts.

Post Number	Size	Height	Decay Category	Post Condition	Meets NDS-01 Design Criteria*
1	6x6	55 1/2"	3	1/4" Horizontal Kerf, Checks at notch - 6" and 2 1/2"	OK
2	6x6	56"	3	1" Horizontal kerf	OK
3	6x6	52 1/2"	3	One Side Hidden, attached to house frame with lag screw	OK
4	4x4	55"	3	Notch 1.5"x1.5"x11.25" for Beam 8	OK
5	6x6	55"	3		OK
6	6x6	63"	3		OK
7	6x6	78"	3	Horizontal kerf	OK
8	6x6	87 1/2"	3		OK

*Criteria checked according to the methods described in NDS-01 is compression stress parallel-to-grain

Table 5.6. The condition of stairway risers, listed from bottom to top.

Riser Number	Condition of Riser	Decay Category	Attachment to Stringer
1		3	Water trapping for all risers (1-7)
2		3	White mold on bottom of all risers (1-7)
3		3	
4		3	
5	1 3/4" Knot Hole	3	
6		3	
7		3	
8		4	Bottom is hidden
9		4	Bottom is hidden
10		4	
11		4	

Table 5.7. The condition of stairway stringers.

Stringer Number	Condition	Decay Category	Attachment to beam
1		3	3 nails that go through beams 9 and 7
2 (center)	Few small splits	3	2 nails, 1/4" spacer
3 (outside)	Few small splits	3	
Under Stairs 8 & 9		4	

6. Inspections to Define the Type and Extent of Deficiencies of In-Service Decks and Balconies

6.1 Introduction

The following inspections are not presented with full reports, but are summarized to highlight deficiencies that exist in typical in-service decks and balconies. These additional inspections were conducted to discover the range and extent of problems that may be present and encountered by the professional inspector. Each deck and balcony was visually inspected and engineering calculations were performed to check the structural elements under several criteria, including bending stress, shear stress, compressive stress parallel-to-grain, and deflection. The conformance of the decks and balconies to Building codes was also checked.

6.2 Objectives

1. To show the variety of deck and balcony designs available
2. To present examples of deck and balcony deficiencies
3. To learn more about the inspection process and issues that may arise

6.3 Deck B

Deck B was a 13' x 24' residential deck attached to a brick home. The date it was built was unknown. Since the residents were renters, no parts of the deck were removed. Grade stamps were not visible, so the elements were conservatively assumed to be No. 3 Southern Pine, one grade below the commonly manufactured PPT No. 2 Southern Pine.

6.3.1 Plan View and Sections

First, the plan view of Deck B and the typical railing section were drawn (Figures 6. 1 and 6. 2). No stairway lead from the ground to the deck.

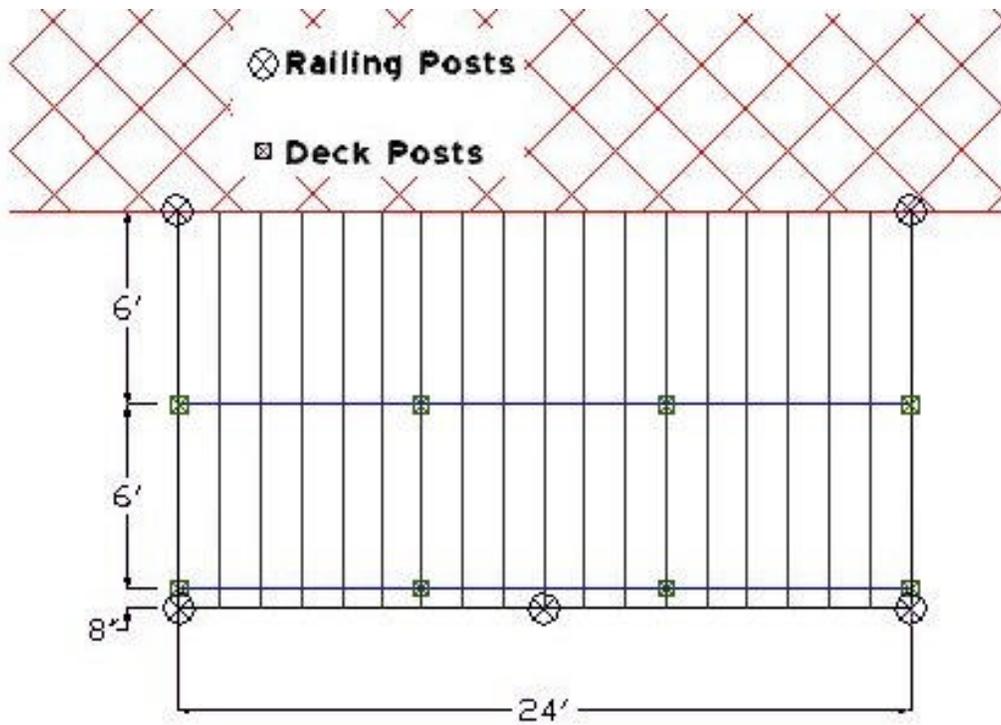


Figure 6.1. Plan view of Deck B showing the locations of joists, beams, rail posts and deck posts.

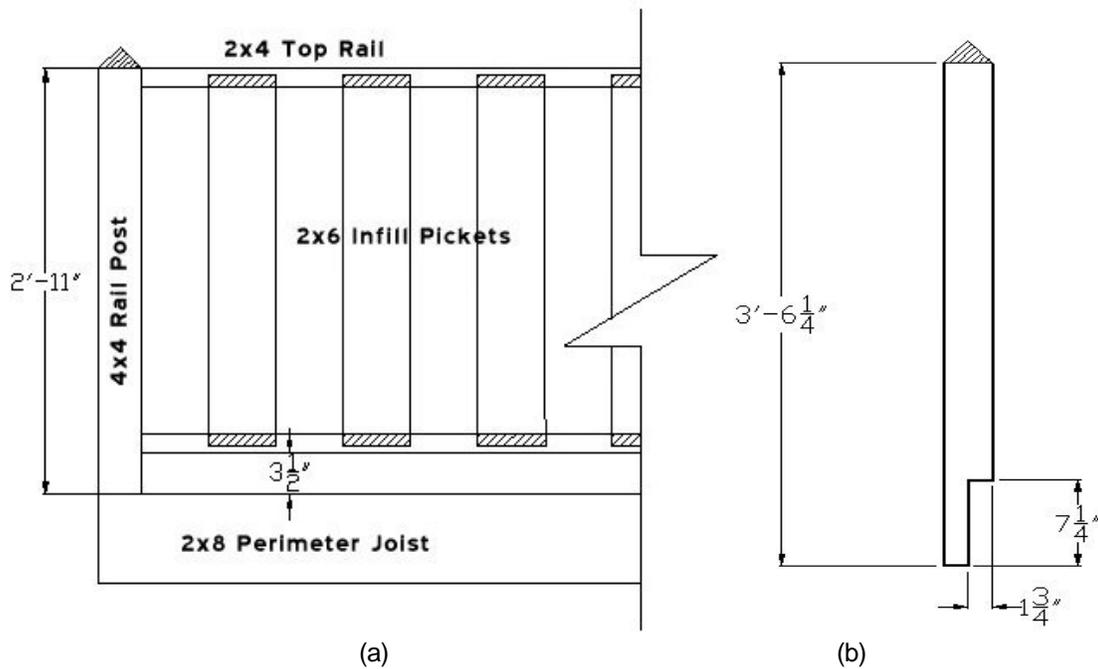


Figure 6.2. (a) A typical railing section. The 2x6 infill pickets were staggered on either side of a 2x4 cap rail. (b) A typical 4x4 rail post.

6.3.2 Railing

The rail posts were 4x4's, 35 in. high (the requirement was 36 in.) and were notched to rest on the deck. The end grain of the rail posts and the infill pickets was exposed. The notch on the rail posts was 7.25 in. long and was 1.75 in. deep. The railing leaned out of plumb at all spans. In the longest section, the railing was 4 in. out of plumb and, when lightly pushed on, the railing moved another inch. Another section was curved along its span. The leaning and curvature are shown in Figure 6.3.



Figure 6.3. (a) The longest section of deck railing leans outward 4 inches. (b). The side section of railing curves inward.

The rail posts were connected to the deck by notching and lag screws. The notches were 7.25 in. deep. Three-quarters of each corner deck post and one-half of the mid-span post were removed. All of the rail posts had checks at the notches and the mid-span post had a split up the center of the notched section.

6.3.3 Decking

The deck boards were 2x6's with no evidence of treatment based on color. Please note that in an actual inspection, the indicator Chrome Azurol S (Mordant Blue 29) could be used to detect the presence of copper, indicating the presence of CCA. Weathered or stained CCA Southern Pine may not show a greenish

color. The nails were popping out and several of the boards were severely warped. In some places, the gap between the boards was 1/2 in. The deck boards near the house were loose.

6.3.4 Joists

The joists were all 2x8's, continuously spanned over two beams with an 8 in. overhang and two rows of staggered solid blocking. They were attached to the ledger board with hangers. The joist spacing was 16 in. on center. The joists were checked according to the NDS-01 design requirements and summarized in Table 6.1. They were adequate in bending, shear and deflection.

Table 6.1. The actual and allowable design values for the joists in Deck B as determined by using the methods in the NDS-01.

	Allowable Design Value based on assumed No. 3 Southern Pine	Design Stress or Deflection
Bending Stress (psi)	805	274
Shear Stress (psi)	170	28
Live Load Deflection (in.)	0.2*	0.012

*Deflection based on L/360

6.3.5 Beams

The beams were 4x4's, spanning 8 ft. They were stacked over the posts and attached with 3"x7" plates with 6 to 10 nails in each plate. The beams were checked according to the NDS-01 design requirements (shown in Table 6.2) and were not adequate. The design failed in bending, shear, and deflection.

Table 6.2. The actual and allowable design values for the beams in Deck B as determined by using the methods in the NDS-01.

	Allowable Design Value based on assumed No. 3 Southern Pine	Design Stress or Deflection
Bending Stress (psi)	850	3,224
Shear Stress (psi)	170	176
Live Load Deflection (in.)	0.27*	0.67

*Deflection based on L/360

6.3.6 Posts

The ground under the deck was flat and covered with grass. The posts were 8'-3.5" high and attached to the footings with non-adjustable post anchors. According to the NDS-01 requirements for compression stress parallel-to-grain, as summarized in Table 6.3, the posts were adequate for the design loads.

Table 6.3. The actual and allowable compression stress parallel-to-grain design values for the deck posts in Deck B as determined by using the methods in the NDS-01.

	Allowable Design Value based on assumed No. 3 Southern Pine	Actual Design Stress
Compression Stress Parallel-to-grain (psi)	405	196

6.3.7 Attachment to house

The ledger board was a 2x10 attached to the house with bolts or lag screws 24 in. on center. The type of connection could not be confirmed without removing the fasteners. The head of the fastener was 1/2 in. in diameter, which, according to the NDS-01 typical dimensions in Appendix L, is a 5/16 in. lag screw or bolt. The length was unknown.

Assuming a 5/16"x3" Lag screw with geometry as described in Appendix L of the NDS-01 and a connection directly to the SPF band joist inside the house (which may not be true), the design load for this connection is 160 lb. With the wet service factor applied, this load is 112 lb. In Deck B, each lag screw would be required to hold 300 lb.

Without removing the ledger board, it looked as though the brick on the house was not removed and the attachment to the house was made through the brick. At the entrance to the deck, the ledger was attached directly to a poured concrete slab. In both cases, flashing could not be seen.

6.3.8 Conclusion

Deck B, as described above, does not meet design criteria as specified in the NDS-01. The beams used were too small to carry the design loads and the

notches on the rail posts were too large. The assumed lag screws are not adequate to carry the design load. The IRC-2000 requirements on railings were not checked, as the railing is in need of a complete replacement. The railing was only 35 in. above the deck boards, 1 in. shorter than required by the IRC-2000.

The final recommendation on this deck is a complete replacement of the structural elements with properly sized beams, a building code-conforming railing and a proper connection to the house framing (as described in Section 4.3).

6.4 Deck C

Deck C was a 8' by 9'-10.5" deck that was recently replaced after the original deck was damaged by a fire. Both grade stamps and PPT quality tags were still visible on the structure. All elements were No. 2 Southern Pine treated with CCA-C to a 0.40 lb/ft³ retention level. The railing pickets were the only elements where a grade stamp could not be located. This was a non-invasive visual inspection; the inspector was not authorized by the homeowner to temporarily remove any parts of the deck structure as would be needed to obtain all information required for a professional inspection.

6.4.1 Plan View and Sections

The plan view of the deck is shown in Figure 6.4. A typical section of the railing and an infill picket are shown in Figure 6.5. In Figure 6.6, a view of the deck posts, which ran through the deck to act as rail posts, is shown.

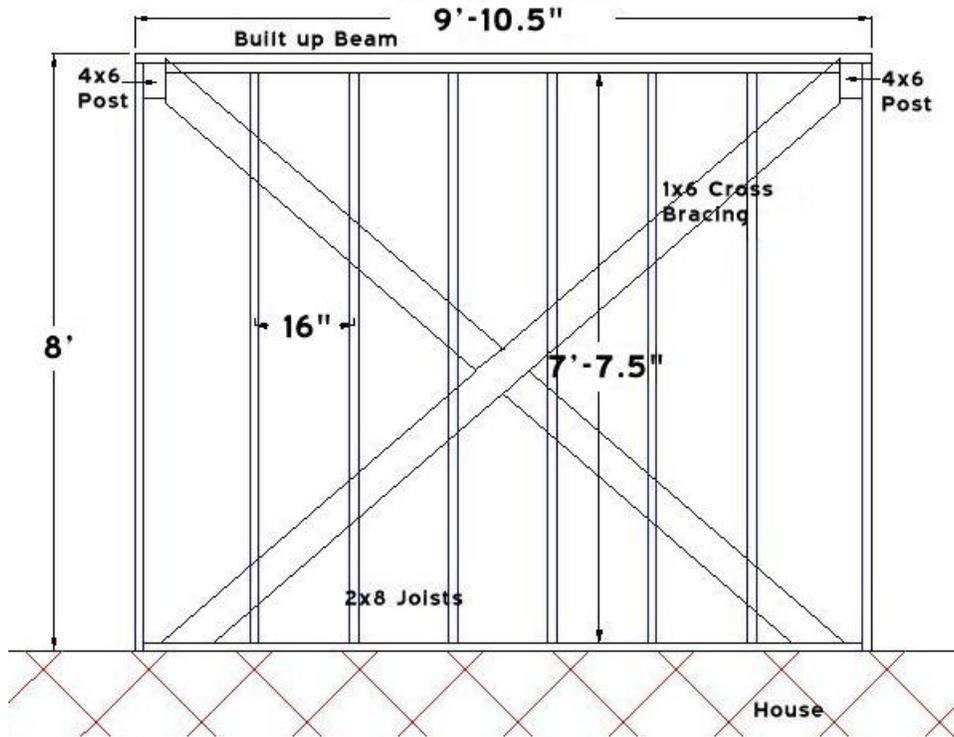


Figure 6.4. The plan view of deck C showing the joists, posts, beams, ledger and cross-bracing.

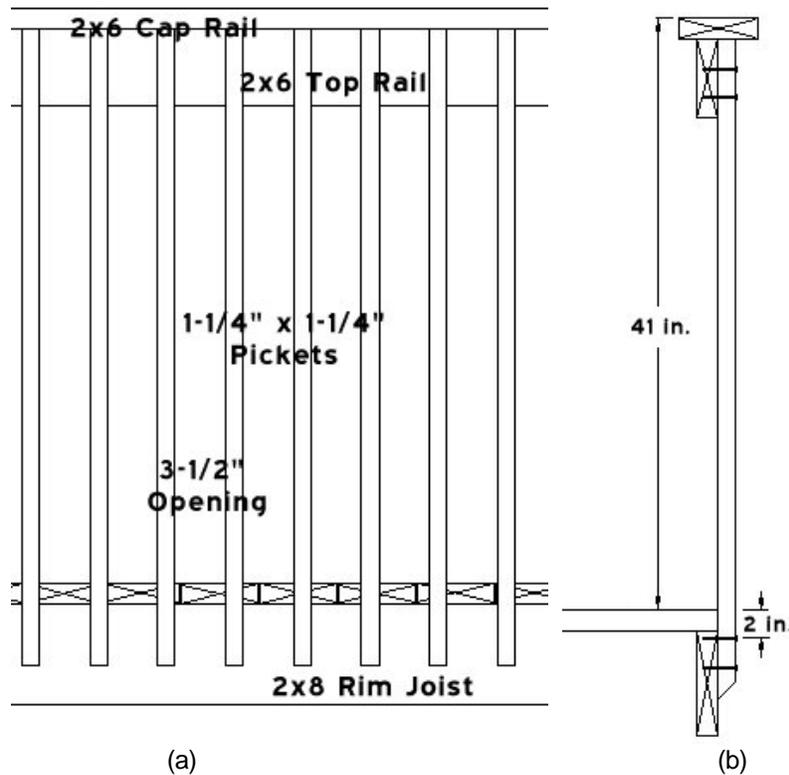


Figure 6.5. (a) A typical section of the railing. (b) A typical railing picket.

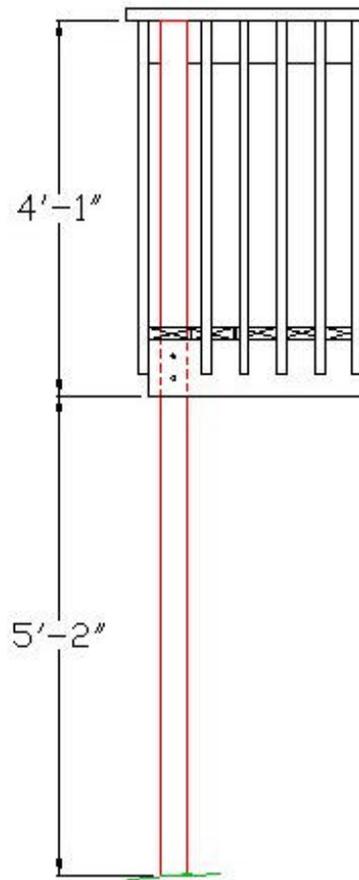


Figure 6.6. A deck post, running from the ground to the top of the railing.

6.4.2 Railings

The railings were 41 in. above the deck surface. The cap rail was a 2x6 nailed to a 2x6 top rail. Each picket was attached to the top rail with two nails and to the rim joist with two nails. The pickets were 1-1/4 in. by 1-1/4 in. spaced 4-7/8 in. on-center. The open space between each picket was 3-1/2 in.

The deck posts continued through the deck to be used as the rail posts. A triangular piece was used in addition to the cap rail to protect the end-grain of the posts. The posts were not notched. At the house, a 2x6 was nailed to the siding

to provide lateral support for the railing. The cap rail was nailed to this 2x6, but did not cover the end grain. The 2x6 was toenailed to the decking, but a gap prevented the 2x6 from touching the deck boards.

The pickets were checked and found adequate for holding the model building code requirement of 50 lb/ft² load anywhere in the infill. This load could cause a maximum of 2.3 lb/in. over a 12 in. section on the picket. These calculations were based on the assumption of adequate support of the pickets at the cap and top rails and the attachment to the deck. The results of the analysis are shown in Table 6.4.

Table 6.4. The actual and allowable design values for the 50 lb/ft² design load on the railings of Deck C as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress
Bending Stress (psi)	1594	770
Shear Stress (psi)	212	22

The railing pickets would not support the 50 lb/ft distributed load at the cap rail when it was directed outward. The bending and shear stresses were too large for the pickets. The top nail in the connection to the deck would be subjected to 450 lb withdrawal load. Any available nail in this situation would not be adequate (NDS-01, Table 11.2.C). The results of the analysis of the 50 lb/ft load on the pickets are shown in Table 6.5. These calculations are based on the assumption that there is no lateral support at the cap rail.

The railing was checked for the 200 lb concentrated load and 50 lb/ft uniform load, which are required in several model building codes. These loads were applied at the cap rail. The 200 lb load was applied at the rail posts and the uniform load was applied to the cap rail over the infill.

Table 6.5. The actual and allowable design values for the pickets in Deck C as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending Stress (psi)	1594	2642
Shear Stress (psi)	212	413
Deflection (in.)	3.58*	1.7

*The allowable deflection on a guardrail is $h/12$ as stated in the Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (ICBO, 2002).

When the 50 lb/ft distributed load is vertical, the nails were found to be able to support the required 10 lb in shear. Using Table 11N in the NDS-01, the smallest nail available will carry 61 lb in this situation. With a wet service factor (C_M) of 0.7 and a load duration factor (C_D) of 1.25, the smallest lateral design value is 53 lb.

The 200 lb concentrated load was checked at the posts, both horizontally and vertically and the posts were found to be adequate. Since the rail post acts as the deck post, both cases of the concentrated load had to be considered with the design loads (40 psf live and 10 psf dead). The results of the analysis are shown in Table 6.6.

Table 6.6. The actual and allowable design values for the posts in Deck C as determined by using the methods in the NDS-01.

		Allowable Design Value	Design Stress or Stress Index (NDS-01 eq. 3.9-3)
Vertical	Compression Parallel-to-grain (psi)	1296	62
Horizontal	Compression Parallel-to-grain (psi)	1296	51
	Bending Stress (psi)	1328	1266
	Shear Stress (psi)	212	16
	Combined Stress Index (CSI)	1.0	0.99

6.4.3 Decking

The deck boards were 2x6 No. 2 Southern Pine. Each board was attached to the joists with two annularly threaded nails. The nails were over-driven into the deck boards and the wood around the nail holes was crushed. In some cases, the wood was soft around these holes. Most of the boards had some cupping near the edges of the deck.

6.4.4 Joists

The joists were 2x8 No. 2 Southern Pine that spanned 93 in. and were placed 16 in. on-center. Hangers were used at both the ledger and the beam. The hangers were USP connectors No. JL28 (REF #LU28) with three nails on the beam/ledger and three nails in the joist. The type of nails could not be determined in a visual inspection. In a professional inspection, a nail would be removed and checked with the hanger manufacturer's literature to find the holding power of the hanger. The joists were checked according to the NDS-01 design requirements and summarized in Table 6.7. They were adequate in bending, shear and deflection.

Table 6.7. The actual and allowable design values for the joists in Deck C as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending stress (psi)	1369	457
Shear stress (psi)	170	35
Live Load Deflection (in.)	0.26*	0.05

*Deflection based on L/360

The rim joists were attached to the posts with one lag screw and toenailed to the ledger. The toenailed connection is adequate to support the reaction load at the ledger. The head of the lag screw was the only visible part. The head was measured and found to be 1/2 in. across the flats. The lag screw was assumed to be a 5/16"x3" lag screw using Appendix L of the NDS-01 because that is the only diameter lag screw with a 1/2 in. head. This lag screw is not adequate for supporting the reaction of the design s at the post.

6.4.5 Beams

The beam was a built-up assmebly made up of two No. 2 Southern Pine 2x8's. The nailing pattern was two nails spaced every 11 to 16 in. The edge spacing for the nails varied from 1 to 2 in. The outer ply of the beam was attached to the posts with two lag screws. Like the lag screws on the rim joists, only the 1/2 in. heads of the lag screws were visible and they were assumed to be 5/16 in. lag screws using Appendix L of the NDS-01. The length was unknown. The beams were checked according to the NDS-01 design requirements, as seen in Table 6.8, and were adequate.

Table 6.8. The actual and allowable design values for the beams in Deck C as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending Stress (psi)	1190	1113
Shear Stress (psi)	170	68
Live Load Deflection (in.)	0.33*	0.21

*Based on L/360

Assuming the lag screws were 5/16"x5", they could each carry 158 lb in lateral loading. The reaction to the design loads at this connection was 990 lbs. The two lag screws used at this connection were not adequate.

6.4.6 Posts

The ground under the deck sloped away from the house and was covered with mulch and grass. The height from the ground to the bottom of the rim joist was 62 in. A detail of the entire post structure is shown in Figure 6.6. The posts were buried in the ground to an unknown depth. The condition of the posts below the ground was unknown.

The posts were checked as described in Section 6.4.2 and were found to be adequate.

6.4.7 Attachment to house

The ledger board was PPT 2x8 No. 2 Southern Pine. The band joist could be seen through an opening by a window. The only material between the ledger and the band joist was aluminum flashing. Without removing the ledger, the condition of the flashing could not be evaluated. The flashing was not seen from the top of the deck, so the coverage is unknown. The basement was not accessible and the fasteners could not be removed, so the type and penetration

of the lag screws or bolts was not determined. The heads of the fasteners were the only visible part and were placed 24 in. on-center. The head was measured and found to be 1/2 in. Using Appendix L of the NDS-01, it was assumed that this connection was 5/16"x4" lag screws. With an assumed Spruce-Pine-Fir band joist, this connection was able to withstand 119 lb in lateral loading. For the design load, the connection must carry 400 lb.

6.4.8 Bracing

The deck was braced against sideways movement on the underside of the joists. The braces were 1x6 and nailed to each joist. A 2x6 was placed between the joists where the center of the cross was located (Figure 6.4). The braces were toenailed to each post and to the ledger.

6.4.9 Conclusions

This inspection was non-invasive; therefore the type and strength of the connections could not be verified. The railing pickets were not able to resist the 50.lb/ft distributed load at the cap rail as required by building codes and their connection to the deck was not adequate. The pickets were not adequate in resisting the design bending stress and shear stress, and the nails were not adequate in withdrawal. It is recommended that they be replaced with 2x4 PPT No.2 Southern Pine (or better) pickets and attached to the deck with two 1/2 in. thru bolts with washers.

The posts were checked for compression parallel-to-grain, bending stress, shear stress and combined stress index (CSI). They were adequate in carrying both the railing loads and the deck loads.

The joists were adequate in bending stress, shear stress and deflection when carrying the design load. The connection of the rim joists to the posts was not adequate under gravity building code loads. The lag screw at this connection should be replaced with two 1/2 in. thru bolts with washers.

The built-up beam was adequate in bending stress, shear stress and deflection. However, the shear connection to the post was inadequate. The load that must be transferred to the post at this connection is 990 lb, which cannot be achieved with fasteners alone in this situation. It is recommended that the connection be repaired to provide wood-to-wood bearing. Another option for repair would be the addition of a scab under the connection with 4 1/2"x5" lag screws to carry the remainder of the design load, as shown in Figure 6.7.

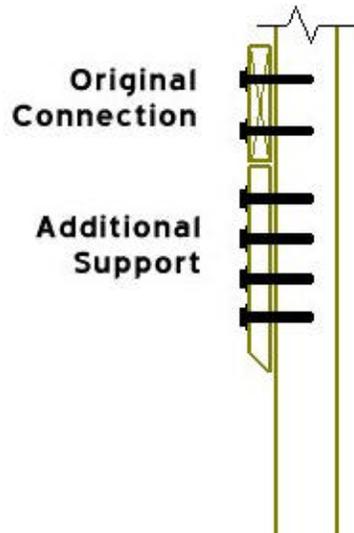


Figure 6.7. The addition of a scab with four 1/2"x5" lag screws would support the design load.

The spacing for the lag screws (24 in. on center) at the ledger is inadequate. The lag screws must be placed at least 4-3/4 in. on center to carry the design load, however, a connection as per the description in Section 4.3 is recommended.

Aluminum flashing has been found to corrode when in contact with PPT lumber, which could allow moisture to enter the band joist. The condition of the band joist should be checked periodically for any decay.

A quality tag on the lumber showed that the treatment retention used was 0.40 lb/ft³. AWPA Standard C15-00 recommends 0.60 lb/ft³ retention for sawn structural posts. The over-driven nails on the cap rail and the deck boards were a concern because they may reach below the penetration of the treatment chemicals, creating water-trapping areas in the untreated material.

This design is lacking in redundancy and relies only on fasteners. If one connection fails, the entire structure could fail. The addition of posts at the deck-

to-house connection and wood-to-wood bearing at the beam-post connection would improve the safety of the structure.

6.5 Deck D

Deck D was 2 to 3 years old when it was inspected. It was built onto an existing brick ranch-style home. Steel posts and beams were used to support the deck. Grade stamps were visible on all structural elements except for the railing pickets. The railing, joists and decking elements were tested for preservative treatment with Chrome Azural S and found to be positive for treatment. Therefore, all elements were PPT No. 2 Southern Pine.

The inspection was based on the visual assessment of the structure only. The inspector was not authorized by the owner to temporarily remove any elements or connections on the deck. In an actual inspection, several fasteners and possibly some structural elements would have to be removed to obtain sufficient information to produce a detailed report.

6.5.1 Plan View and Sections

Figure 6.8 shows the plan view of the deck. The steel posts and beams are shown in the drawing. The steel beams supported the joists in the larger section of the deck. A typical railing section, post and picket are shown in Figure 6.9 and the stairway is shown in Figure 6.10.

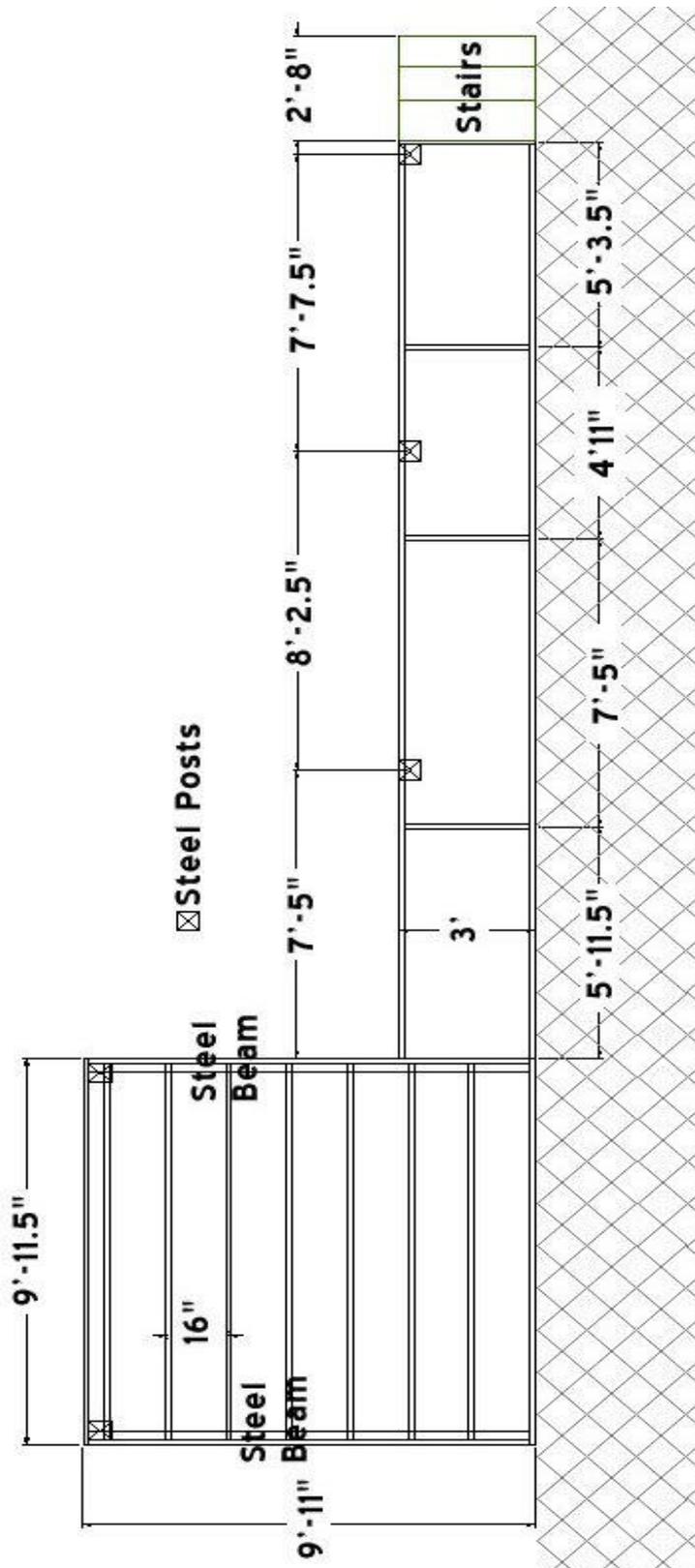


Figure 6.8. The plan view of Deck D, showing the steel posts, steel beams, and joists.

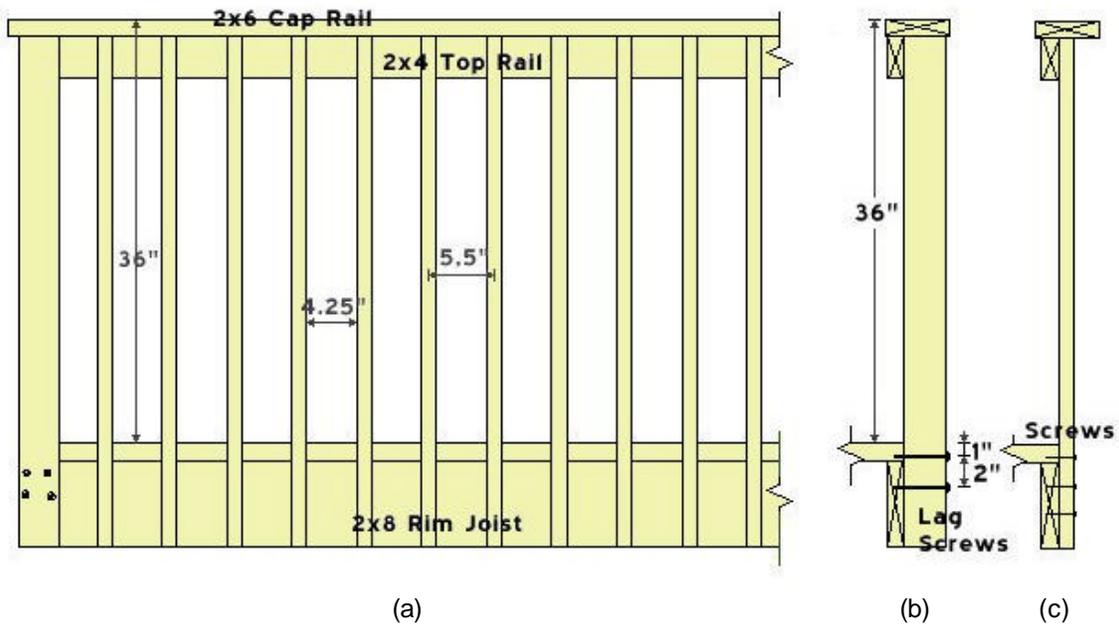


Figure 6.9. (a) A typical railing section on Deck D (b) a typical rail post and (c) a typical infill post

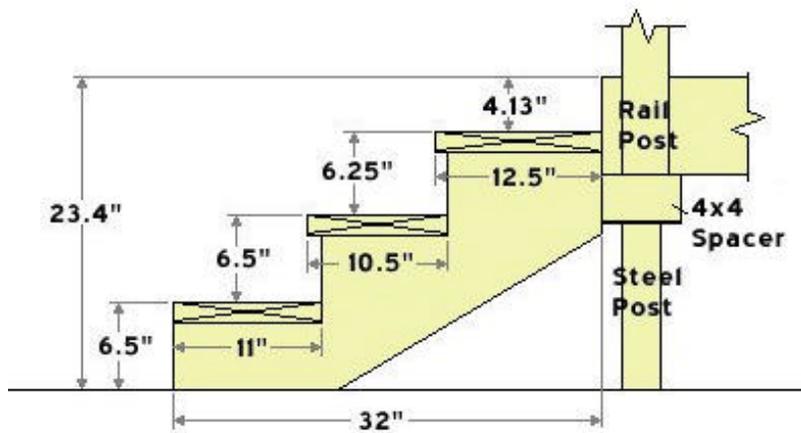


Figure 6.10. A side view of the stairs on Deck D.

6.5.2 Railings

The railing pickets were 1-1/4 in. by 1-1/4 in. and the posts were 4x4s. They reached to 36 in. above the deck surface. The 2x6 cap rail was screwed to the 2x4 top rail. Each picket was attached to the top rail with one screw and to the rim joist with three screws. The rail posts were attached to the rim joists with

four lag screws. The only visible part of the lag screw was the head, which was measured and found to be 1/2 in. across the flats. In Appendix L of the NDS-01, this measurement corresponded to a 5/16 in. lag screw. For this evaluation, the lag screws are assumed to be 5/16"x5". In an actual inspection, a lag screw would be removed, checked for corrosion and measured. The rail posts were located at each corner of the deck with one at the midspan of the 23 ft walkway. The post at the stairway had only three lag screws attaching it to the rim joist.

Model building codes require design loads of 200 lb concentrated at the cap rail, 50 lb/ft uniformly distributed at the cap rail and 50lb over 1 ft² in the infill. The concentrated load was checked at the rail posts and the uniform load was checked at the cap rail over the infill pickets.

The infill pickets were checked for the 50 lb/ft² building code load requirement at any point over the infill and the pickets were found to be adequate. Table 6.9 shows the results of these calculations. These calculations are based on the assumption that the connection strengths at the top and cap rails and the attachment to the deck were adequate.

Table 6.9. The actual and allowable design values for the 50 lb/ft² design load on the infill pickets of Deck D as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending Stress (psi)	1594	756
Shear Stress (psi)	212	26

The infill pickets were also checked with the 50 lb/ft distributed load requirement projected outward and were not adequate. The pickets failed in bending stress and shear stress. These calculations were based on the assumption of no lateral support at the cap rail and the results are shown in Table 6.10.

Table 6.10. The actual and allowable design values for the 50 lb/ft distributed load on the infill pickets of Deck D as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending Stress (psi)	1594	2720
Shear Stress (psi)	212	850
Deflection (in.)	3.21*	1.38

*The allowable deflection on a guardrail is $h/12$ as stated in the Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (ICBO, 2002).

The pickets were attached to the rim joist with screws. The type of screw was not determined. For an actual inspection, one screw would have been removed and its length, root diameter and condition would have been recorded. For this report, the screws were assumed to be wood screws that penetrated fully into the rim joist (1.5 in.). When the 50 lb/ft distributed load was applied outward from the deck, the three screws form an indeterminate connection. The top two screws were assumed to be working together and would be subjected to 284 lb (141 lb each) withdrawal load. In these conditions, the smallest withdrawal load a wood screw can carry is 156 lb (NDS-01, Table 11.2B). When the 50 lb/ft distributed load was applied downward, the screws would have to resist 7 lb each in shear. The smallest load that a wood screw in these conditions can carry is 68 lb based on NDS-01, Table 11L.

The 200 lb load was checked both horizontally and vertically at the rail posts. When it was applied horizontally outward, the post was checked for adequate bending strength, shear strength and deflection. The post was not adequate in shear strength. The results, based on the assumption of no support at the top and cap rails, are shown in Table 6.11.

Table 6.11. The actual and allowable design values for the 200 lb load applied horizontally on the rail posts of Deck D as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending Stress (psi)	1594	1036
Shear Stress (psi)	212	453
Deflection (in.)	3.08*	0.18

*The allowable deflection on a guardrail is $h/12$ as stated in the Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (ICBO, 2002).

The 4x4 rail posts were connected to the rim joist with four lag screws. The lag screws were not removed for this inspection, but would be for a professional inspection for measurement and an assessment of their condition. Only the heads of the lag screws were visible and they did not penetrate the thickness of the 2x8 rim joist. The heads were measured across the flats and found to be 7/16 in. From Appendix L of the NDS-01, the lag screws were determined to be 1/4 in. lag screws. The length was not determined.

When the 200 lb concentrated load was applied downward, each lag screw needed to support 50 lb in shear. 1/4"x5" lag screws in these conditions could support 142 lb each. When the 200 lb load was applied outward, the top two lag screws would need to resist 3900 lb withdrawal load. Each lag screw in this condition could hold a maximum of 306 lb in withdrawal.

The top lag screws holding the 4x4 rail posts to the deck and the top screw holding the pickets to the deck were driven into the end grain of the deck boards. This detail reduces the withdrawal design value of the lag screws by 25% (NDS-01, Section 11.2.1.2) and the lateral design values of both fasteners by 33% (NDS-01, Section 11.5.2.2). The NDS-01 states that nails are driven into end grain for withdrawal loads are not permitted (Section 11.2.3.2).

6.5.3 Decking

The deck boards on Deck D were 2x6 No. 2 PPT Southern Pine. The gaps between the boards were up to 1/4 in. wide. The boards were attached to the joists with screws. Several boards were split near the ends.

6.5.4 Joists

The joists were 2x8 No. 2 PPT Southern Pine. In the large section of the deck, the joists were spaced 16 in. on-center and were supported by the steel beams. The span of the joists in this section was 116.5 in.

The joists on the main section of the deck were checked for bending strength, shear strength, and deflection under the design loads. They were adequate and the results are shown in Table 6.12.

Table 6.12. The actual and allowable design values for the joists in Deck D as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending stress (psi)	1369	717
Shear stress (psi)	170	45
Live Load Deflection (in.)	0.32*	0.13

*Deflection based on L/360.

The walkway section of the deck was supported by the joist at the house and a rim joist. The rim joists were supported by the steel beams. The rim joist was checked in bending strength, shear strength and deflection and found to be adequate.

The joist that ran alongside the house was attached with Tapcon screws for masonry with two screws every 24 in. In an actual inspection, one screw could be removed and measured. Since a screw could not be removed for this inspection, the strength and adequacy of this connection is unknown. Literature for the Tapcon screws stated that the minimum shear strength of the screw in lightweight hollow brick was 731 lb, but use of the European Yield Mode equations (NDS-01, Section 11.3.1) showed that the failure would occur at 180.lbs. This value is adequate for the holding the gravity loads required by code.

6.5.5 Steel Beams

The beams were steel W-shape beams. They were supported by steel posts at one end and a hole was cut into the masonry foundation at the house.

6.5.6 Steel Posts

The posts were round steel posts that were not buried in the ground. The footings could not be seen, but the renter informed the inspector that the posts would move out of plumb when he hit them with his truck.

The post below by the stairway was too short to support the rim joist. A small section of a 4x4 was inserted at this point as a spacer.

The ground below the deck was covered with grass.

6.5.7 Stairs

The rise of the stairs was 23 in. Railings were not present at the stairway. The width of the stairway was 36 in. and the total run was 32 in. The tread depths and riser heights were not uniform. The largest opening behind the risers was 5 in., larger than allowed by the building codes. Artificial lighting was not provided at the stairway.

6.5.8 Attachment to house

Deck D was not dependant on the connection to the house for the larger section of the structure. A single joist was attached to the brick masonry wall using two Tap-con Screws placed 24 in. on-center. The strength of this connection was discussed above in Section 6.5.4.

6.5.9 Conclusions

The inspection of Deck D was non-invasive. Mechanical fasteners were not removed and therefore their actual design strengths were not found. The elements were pressure-treated.

The railing pickets on Deck D were adequate for supporting the 50 lb/ft² load anywhere in the infill but could not safely support the 50 lb/ft distributed load in bending and shear when it was projected outward. The three screws used to connect the infill to the rim joist were adequate for resisting this design load if they were not attached to end grain. This connection should be moved to the rim joist and not the end grain of the deck boards.

The rail posts and their connection to the rim joists were checked for strength with the 200 lb concentrated load applied. The posts were not adequate for this load in shear and must be replaced with larger posts or position the fasteners at the connection at least 5 in. apart rather than the current 2 in. apart. The top two fasteners should be replaced with 5/16 in. thru bolts with washers and moved out of the end grain of the deck boards.

The joists on Deck D were adequate for the design loads and their connection to the house (Tapcon screws) was also adequate.

The total rise of the stairway was 23 in. and does not require any railings. However, the risers are not equally spaced. According the IRC-2000, "the greatest riser height within any flight of stairs shall not exceed the smallest by more than 3/8 in." (R314.2). The difference within these steps is 2-3/8 in. The tread depths are also unequal. Like the risers, the greatest difference allowed by

the IRC-2000 is 3/8 in. The greatest difference within these tread depths was 2 in. Behind the stairs, blocking was not provided and the opening was greater than 4 in. The stairway should be rebuilt with equally spaced risers and uniform tread depths. Blocking should be included to reduce the opening behind the steps to less than 4 in.

The attachment of the deck to the house and the lateral support of the deck were provided with Tapcon masonry screws. These screws were adequate in supporting the design loads. The steel beams were supported by the house foundation, but the condition of this area could not be determined.

The steel beams and posts in this inspection were assumed to be adequate. The quality of the footings for the steel posts was not determined. The renter claimed that the posts moved out of place when he hits them with his truck. The posts should be properly anchored to concrete footings that are below the frost line.

Wood-to-steel bearing is used in several places in Deck D. The joists run parallel to the house, using the steel beams for support rather than a ledger board. The steel beams were supported by steel-to-concrete bearing rather than mechanical connections. With these design features, the failure of one element on Deck D is not likely to cause the entire deck to fail.

6.6 Deck E

Deck E was a two-story structure with continuous posts that also served at railing posts. The lower deck was inspected. The date of construction was unknown. The inspection was non-invasive because the residents were renters

and permission from the owner was not given to remove any hardware from the structure. For all needed information to be obtained, some fasteners and lumber would have to be temporarily removed and checked for condition and size.

The structural elements on the deck were checked for the presence of copper using Chrome Azurol S. The tests were positive, indicating the lumber was preservative pressure treated, most likely CCA. Grade stamps were not visible on any elements. All lumber was stained on the topsides with a reddish tint. The typical lumber used on decks in this area is No. 2 Southern Pine or No. 1 Western Cedar. For all calculations in this inspection report, the joists and decking were assumed to be No. 3 Southern Pine. The built up beams, railings, and posts were assumed to be No. 2 Western Cedar.

6.6.1 Plan View and typical Sections

The plan view of Deck E is shown in Figure 6.11. A full view photograph of the two-story structure is shown in Figure 6.12. The typical measurements of the post are shown in Figure 6.13. Typical sections of the railing and the railing post at the side of the building are shown in Figure 6.14.

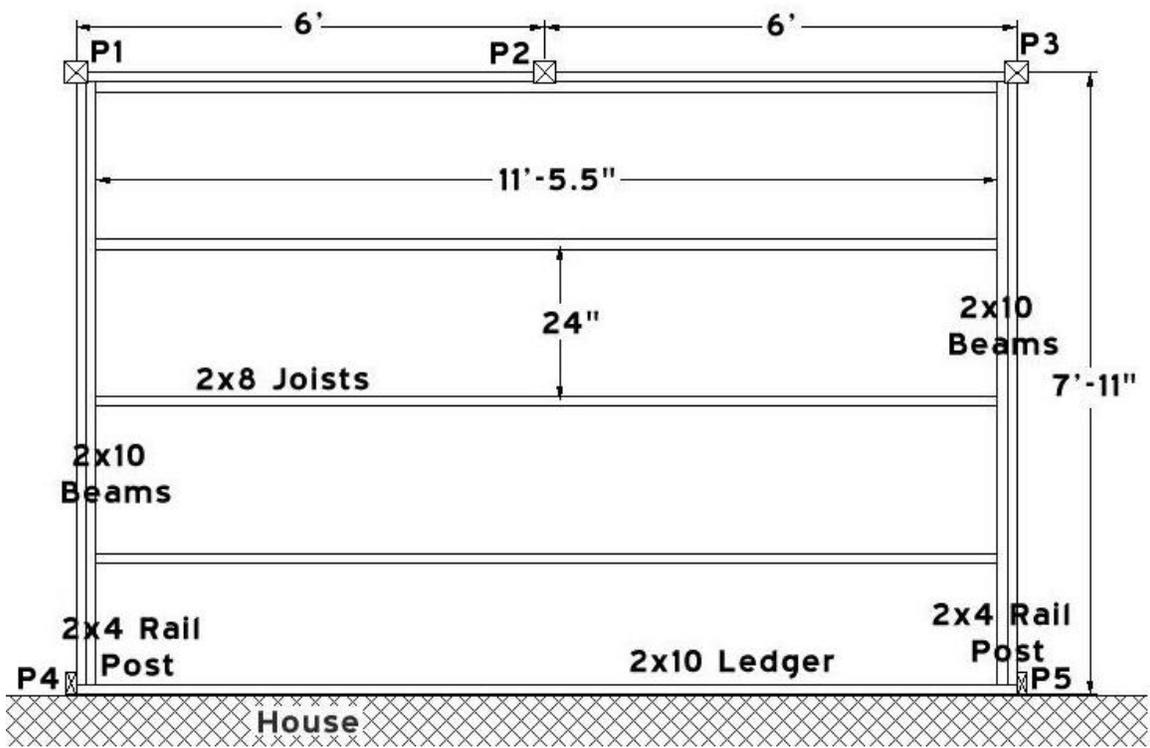


Figure 6.11. The plan view of Deck E, showing joists, posts and railing posts. Posts P1, P2 and P3 are 4x4 posts detailed in Figure 6.13. Posts P4 and P5 are 2x4 railing posts shown in Figure 6.14.



Figure 6.12. Deck E was two stories. The 4x4 deck posts were continuous up to the bottom of the top deck.

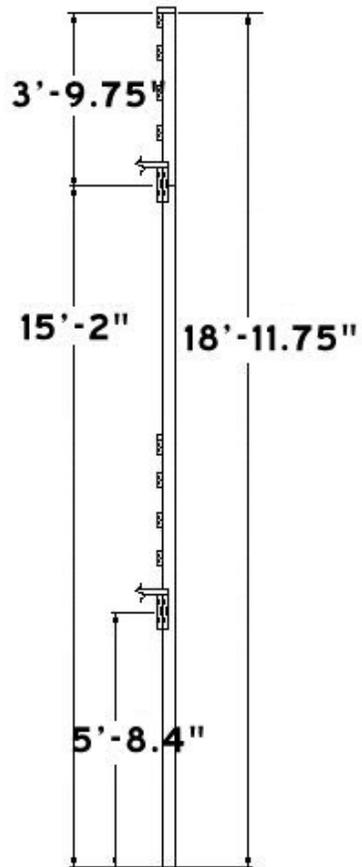


Figure 6.13. A side view of posts P1, P2 and P3 on deck E showing notches and where the railings were attached.

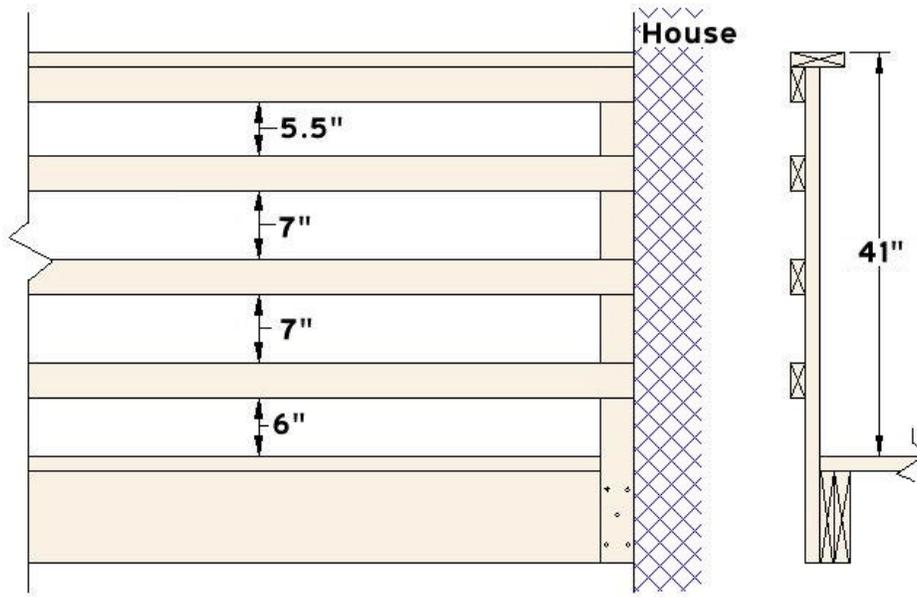


Figure 6.14. (a) The typical railing on Deck E. (b) A side view of rail posts P4 and P5 on the deck located next to the exterior house wall.

6.6.2 Railings

The railings of Deck E were 2x4 horizontal rails with a 2x6 cap rail. The rails were nailed to the posts with two or three nails. The building code design loads were 50 lb/ft distributed load applied in any direction at the cap rail, 50 lb over one square foot anywhere on the infill and a 200 lb concentrated load applied at the cap rail in any direction.

The infill was checked for supporting the 50 lb over one square foot and found to be adequate. This load would be supported by a single rail. The results were calculated using tabulated allowable design values for Western Cedar in Table 4A of the NDS-01. The nails used to attach the rails to the posts were assumed to be threaded nails. When the 50 lb/ft² is applied outward, the nails must be able to resist 47 lb in withdrawal. The smallest diameter threaded nail (0.120 in., Table 11C of the NDS-01) is able to resist 14 lb/in. penetration when

loaded in withdrawal. With all adjustment factors and assuming a 1.5 in. penetration, the nail is able to resist 26 lbs in withdrawal. With at least two nails, the connection at the rails to the post was adequate.

Table 6.13. The actual and allowable design values for the 50 lb/ft² design load on the railings of Deck E as determined by using the methods in the NDS-01.

	Allowable Design Value based on Assumed No.2 Western Cedar	Design Stress
Bending Stress (psi)	1440	847
Shear Stress (psi)	188	13

The 50 lb/ft distributed design load was checked when applied to the cap rail horizontally, and the cap rail alone could support the load. When the load was applied vertically, the 2x6 cap rail could not support it. The cap rail was nailed to the 2x4 top rail, which, by itself, could not support the design load applied in either direction. The railing is not adequate under the 50 lb/ft distributed design load.

The 200 lb concentrated design load was checked at the posts. The single rail posts next to the building (P4 and P5) were not adequate for supporting the design load in shear strength, bending moment resistance or deflection. The results of the analysis on the 2x4 posts are shown in Table 6.14. The connection of the post to the deck was made up of five nails. The top two nails were not adequate to resist the required load of 1938 lb in withdrawal.

Table 6.14. The actual and allowable design values for the 2x4 rail posts in Deck E as determined by using the methods in the NDS-01.

	Allowable Design Value based on Assumed No. 3 Southern Pine	Design Stress or Deflection
Bending Stress (psi)	1313	6827
Shear Stress (psi)	188	554
Deflection (in.)	3.73*	6.72

*The allowable deflection on a guardrail is $h/12$ as stated in the Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (ICBO, 2002).

The 4x4 posts (P1, P2 and P3) were continuous and used as the deck posts as well as the rail posts. The posts extended up to support the top deck. The posts were notched to hold the built up beams and decking. When the 200.lb concentrated load required by the model building codes is applied outward on the post, the post becomes a bending member. Table 6.15 shows the allowable notch sizes for a bending member and the actual notch sizes, which exceed the allowable. The notch for the post, shown in Figure 6.15, was in the middle third of the member, which is not allowed by the NDS-01 in a bending member (Section 4.4.3).

Table 6.15. The actual and allowable notch sizes for the 4x4 post used in Deck E based on the NDS-01, Section 4.4.3

	Allowable Depth	Actual Depth	Allowable Length	Actual Length
Top Notch	7/8"	1-3/4"	NA	4-5/8"
Notch for beam	7/12"	1-3/4"	1-1/6"	10-3/4"



Figure 6.15. The notch for the 2-2x10 beams on posts P1, P2 and P3 in Deck E. The length, depth, and location of this notch are not allowed by the NDS-01.

Table 6.16 shows the results from checking the gravity loads (40.psf live and 10.psf dead) and the horizontal railing design loads (200.lbs concentrated outward) applied to the posts. The posts were not adequate in compression parallel-to-grain or bending moment resistance.

Table 6.16. The actual and allowable design values for the posts in Deck E when the 200 lb load required by building codes was applied outward and the post supported the required gravity loads.

	Allowable Design Value	Design Stress or Stress Index (NDS-01 Eq. 3.9-3)
Compression Parallel-to-grain (psi)	163	195
Bending Stress (psi)	1285	3233
Shear Stress (psi)	188	24
Combined Stress Index (CSI)	1.0	N/A*

*When design compression stress is too high, the combined stress index becomes negative

6.6.3 Decking

The deck boards on Deck E were 2x6 Southern Pine. They were stained red and attached to each joist with two nails. Several burn marks and watermarks were found on the deck boards. The renters placed a tarp below the joists on the top deck to prevent water from dripping onto the lower deck. The deck boards were all in good condition with the exception of the surface discolorations.

6.6.4 Joists

The joists on Deck E were 2x8 PPT Southern Pine placed 24 in. on center. The joists were supported by a 2x2 ledger that was nailed to a built up beam. The nails for the ledger were placed every 4 in. The joists were also attached to the beams with two nails. The joists were checked with the design gravity loads in bending stress, shear stress and deflection. They were not adequate in bending stress or deflection (Table 6.17) based on assumed No. 3 grade.

Table 6.17. The actual and allowable design values for the joists in Deck E as determined by using the methods in the NDS-01.

	Allowable Design Value based on No. 3 Southern Pine	Design Stress or Deflection
Bending stress (psi)	805	1499
Shear stress (psi)	170	79
Live Load Deflection (in.)	0.38*	0.47

*Deflection based on L/360

The nails on the 2x2 ledger would need to transmit 96 lb in shear to the beams. Assuming the nails penetrated 3 in. into the beam, the largest nail available (0.244 in.) could hold 91 lb. The nailing of the ledger was not adequate.

The joist next to the house was attached to the house with lag screws or bolts with washers placed 24 in. on center. Since the fastener could not be removed, the only visible part, the head, was measured across the flats. The head measurement was 9/16 in., which corresponds to a 3/8 in. lag screw (NDS-01, Appendix L). Assuming that the ledger is attached to a Spruce-Pine-Fir band joist, each lag screw would be able to carry 106 lbs. The required capacity of each lag screw under the gravity loads would be 100 lbs.

6.6.5 Beams

The beams on Deck E were two-ply built-up 2x10 Western Cedar. The nailing of the two members was 24 in. on center with 2 in. edge distances. The number of nails varied from one to three. The beams sat in notches created in the posts and were toenailed to the ledger board with three nails. The toenails would be required to support 900 lbs under the gravity loads. In these

conditions, the toenailed connection (assuming a length of at least 3 in. and a 20d Common nail - 0.192 in. diameter) is able to support 63 lb. The penetration of the nail is below the required 6D (1.15 in.). The connection of the beam to the ledger is inadequate. Also, the ledger connection to the house is only adequate for supporting the joist, as described in Section 6.6.4.

The beams were checked for their ability to support the gravity loads and were adequate. The results are shown in Table 6.18.

Table 6.18. The actual and allowable design values for the beams in Deck E as determined by using the methods in the NDS-01.

	Allowable Design Value based on assumed No. 2 Western Cedar	Design Stress or Deflection
Bending Stress (psi)	700	659
Shear Stress (psi)	150	64
Live Load Deflection (in.)	0.26	0.11

* Deflection based on L/360

6.6.6 Posts

The posts were 4x4 material and extended through the deck to support the second level deck. The posts were not adequate; the analysis was described in Section 6.6.2.

The ground under the deck sloped along the house and was covered with mulch and weeds. The cross sectional areas of posts P1, P2 and P3 were decreased at ground level, possibly from erosion by surface drainage and runoff. They were checked with the “pick test” and decay was not detected. Figure 6.16 shows the condition of the bottom of one of these posts.



Figure 6.16. The erosion of a post in Deck E, possibly caused by surface water runoff.

6.6.7 Attachment to building

The joists ran parallel to the building in Deck E. The joist and beam attachment to the building were described in sections 6.6.4 and 6.6.5, respectively.

6.6.8 Conclusions

The railing of Deck E did not meet the requirements of the current model building code. The openings were up to 7 in. wide and were built in a way that encouraged climbing. The joists were inadequate in bending moment resistance and deflection and their connection to the beams. The beams were adequate, but the connection to the house was not. The posts were undersized and inadequate in compression parallel-to-grain and could not support the railing

design loads. The notches in the posts were unacceptable according to NDS-01, Section 4.4.3.

It is recommended that this deck be replaced with properly sized structural elements, connections and railing system.

6.7 Deck F

Deck F was, by definition, a deck. It was surrounded on three sides by the building and had a post in one corner. For this reason, all calculations were completed using the required gravity loads for decks (40 psf live and 10 psf dead). Deck F was located on the third floor of a building. Under Deck F, one other deck was supported by a post in the same position as the post in Deck F for a total of three 96 in. posts leading from the ground to the roof.

Grade stamps were visible on all elements of the deck except the railing pickets. All elements were No. 2 Southern Pine. A test for the presence of copper, using Chrome Azural S, was performed. The results of the test were positive, indicating that the lumber used on the deck was preservative pressure treated.

The inspection of Deck F was non-invasive. In a professional inspection, permission from the homeowner would be obtained to temporarily remove a few structural elements and connections to verify their condition and take measurements.

6.7.1 Plan View and Typical Sections

The plan view of Deck F is shown in Figure 6.17. A section of the railing and a typical picket are shown in Figure 6.18. The post is shown in Figure 6.19.

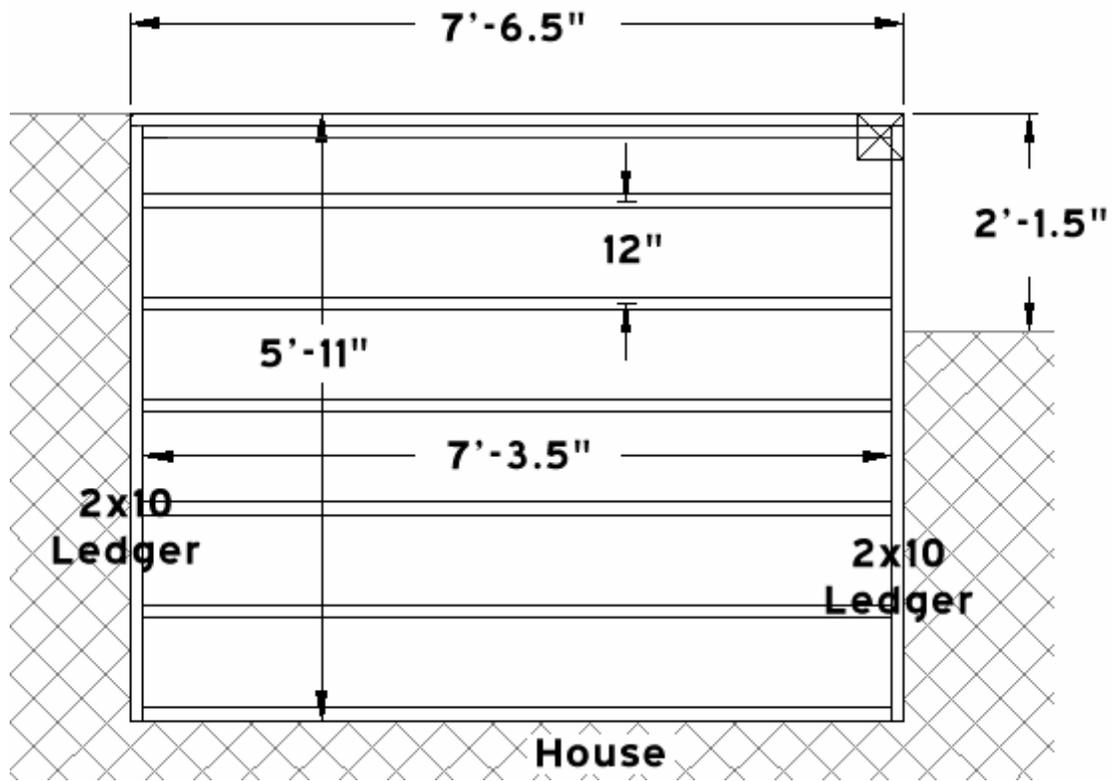


Figure 6.17. The plan view of Deck F, showing the joists, ledgers and the single post.

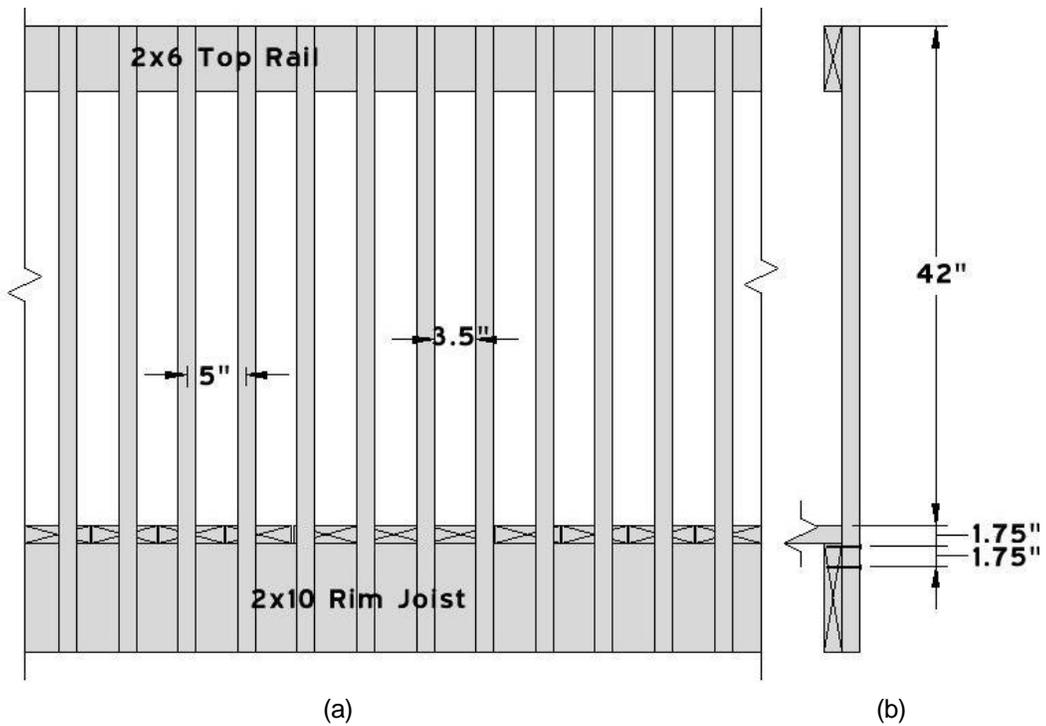


Figure 6.18. (a) A typical section of the railing on Deck F and (b) A typical picket from the railing on Deck F.

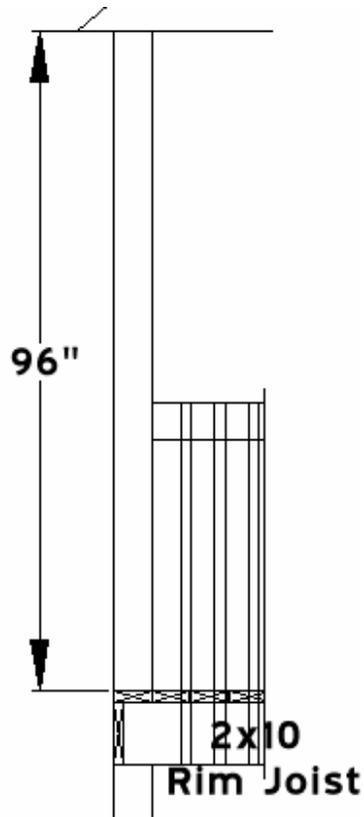


Figure 6.19. The post was not continuous through Deck F. It was toenailed to the deck boards. Another deck was under Deck F. Three separate posts created a column from the ground to the roof of the building.

6.7.2 Railings

The railing pickets of Deck F were 2x2 Southern Pine. They were spaced 5 in. on-center, leaving a 3.5 in. open space between pickets. The pickets were anchored to the rim joist with two threaded nails of unknown length as shown in Figure 6.18(b). Building codes require the pickets to resist a 50 lb load distributed over a 1 ft² area. This loading was checked under the assumption of proper support at the top rail and at the attachment to the deck. Assuming the pickets were No. 2 grade, the results were that the pickets were adequate (Table 6.19.).

Table 6.19. The actual and allowable design values for the 50 lb/ft² design load on the railings of Deck F as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress
Bending Stress (psi)	1594	472
Shear Stress (psi)	212	16

The building codes also require that the railings be able to resist a 50 lb/ft distributed load at any direction at the top rail. When this load is applied outward, assuming no support at the top rail, the top fastener of the picket to the rim joist would have to resist 541 lb in withdrawal. When the 50 lb/ft is applied downward, each of the two fasteners would have to resist 10 lb in shear. The fasteners at this connection were not removed, as they would be in a professional inspection, so their diameter and length were not determined. The nails did not penetrate through the thickness of the rim joist. In Southern Pine, the largest threaded nail available has a design value of 131 lb in withdrawal (87 lb/in. with the Load Duration Factor equal to 1.25 and assuming 1.5 in. penetration into the main member; NDS-01, Table 11.2C). The smallest threaded nail has a design value of 76 lb in shear. The nails used are not adequate in withdrawal but are adequate in shear.

The pickets were checked in bending stress, shear stress and deflection under the 50 lb/ft distributed load and found to be inadequate in bending and shear strength. The results of the checking are shown in Table 6.20. These results were based on the assumption of no support at the top rail.

Table 6.20. The actual and allowable design values for the pickets in Deck F as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending Stress (psi)	1594	1620
Shear Stress (psi)	212	347
Deflection (in.)	3.58*	0.90

*The allowable deflection on a guardrail is $h/12$ as stated in the Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (ICBO, 2002).

The top rail of the deck was attached to the brick siding of the building with L-brackets and six Tap-con screws, as shown in Figure 6.20(a) and to the post with L-brackets and four screws (Figure 6.20(b)). The thickness of the brackets was measured to be between 1/8 in. and 3/16 in. Using the actual dimensions in the NDS-01 for steel side plates (NDS-01, Table 11M), the brackets were determined to be 10 gage (0.134 in. thick). When the 50 lb/ft distributed load is applied outward, the screws would have to resist 177 lb total in shear. This is approximately 30 lb each for the Tapcon screws and 44 lb each for the screws at the post. As discussed in Section 6.5.4, the sample Tapcon screws were able to hold 180 lb each in shear when used to connect Southern Pine to Brick. With steel side members, this value would be greater. The smallest diameter wood screws with a steel side plate in Southern Pine (assuming ASTM A653, Grade 33 as used in NDS-01, Table 11M) would be able to resist 100 lb in shear. For this inspection, both the Tapcon screws and the screws into the post are adequate. In a professional inspection, the fasteners would be temporarily removed to find their actual dimensions and condition.



(a)



(b)

Figure 6.20. The top rail of Deck F was attached (a) to the building with six Tapcon screws and (b) to the post with four screws.

The screws in the wood would be loaded in withdrawal if the 50 lb/ft distributed load was applied outward. When this load is applied, the first set of screws would have to resist the entire load; if they fail, the second set would have to resist the load. Each screw would have to carry 94 lb in withdrawal. In these conditions (assuming 1.5 in. penetration into the Southern Pine and wet service conditions), the smallest allowable load carried by a wood screw (NDS-

01, Table 11.2B) would be 125 lb. The screws are adequate for carrying the 50 lb/ft design load applied outward.

The building codes also call for the railings to be able to resist a 200 lb concentrated load in any direction. This load was checked outward at the post, which also served to hold a 4 ft² section of the roof. Based on a 30 psf snow load and a 20 psf dead load, the axial loading on this post would be 200 lb. The post was checked in compression parallel-to-grain, bending stress, shear stress and combined stress index (CSI) and found to be adequate. The two posts under the deck were also checked for their adequacy in carrying all design loads required, including the loads from the decks and roof. The results for the top post are shown in Table 6.21.

The 200 lb load was also checked when applied in any direction at the cap rail immediately next to the L-bracket connections. Since the first screw would have to carry the entire load, the design load of 200 lb in shear or withdrawal was checked on the first screw of each fastener only (100 lb/fastener). The screws embedded in the brick would not be loaded in withdrawal, only shear. These screws can carry 180 lb or more in shear. As discussed above the fasteners in Southern Pine would be able to carry 125 lb in withdrawal and 100 lb in shear. The connection of the cap rail to the house is adequate.

Table 6.21. The actual and allowable design values for the top post in Deck F when the 200 lb load required by building codes is applied outward and the roof load is applied as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Stress Index (NDS-01 Eq. 3.9-3)
Compression Parallel-to-grain (psi)	578	7
Bending Stress (psi)	1063	171
Shear Stress (psi)	206	3
Combined Stress Index (CSI)	1.0	0.16

6.7.3 Decking

The deck boards on Deck F were 2x6 PPT No. 2 Southern Pine. The deck boards were attached to the joists with nails and several were overdriven. All boards were in good condition showing no signs of decay.

6.7.4 Joists

The joists were 2x10 No. 2 PPT Southern Pine. They were attached to the ledgers with hangers. The hanger type could not be seen because of height, but all nail holes were filled. The joists were placed 12 in. on-center. They were checked for bending strength, shear strength and live load deflection when carrying the building code design loads of 40 psf live and 10 psf dead and found to be adequate. The results from these checks are shown in Table 6.22.

Table 6.22. The actual and allowable design values for the joists in Deck C as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending stress (psi)	1208	186
Shear stress (psi)	170	20
Live Load Deflection (in.)	0.24*	0.02

*Based on L/360

6.7.5 Post

The adequacy of the post on Deck F was discussed in Section 6.7.2. The post was 6x6 PPT No. 2 Southern Pine and was 96 in. long. The top post was toe-nailed to the deck boards and the connection at the top could not be determined. The second floor post was toenailed to the deck boards and the joists sat on top of the post and were toenailed to the post. Blocking was not present. The ground floor post was sitting on the masonry pad with a barrier protecting it from the concrete. The anchorage of the post was not determined. All posts were in good condition.

6.7.6 Attachment to building

The ledgers were attached to the building with 1/2 in. through bolts with washers placed 24 in. on-center. The side of the deck with the post had two bolts and the side without the post had three bolts. On the side with two bolts, each bolt would need to carry 457 lb in shear and on the side with three bolts, each would be loaded by 560 lb of shear. The material behind the ledger could not be seen. Assuming that the ledger is attached directly to a 2x_ SPF band

joist, each bolt would be able to carry 132 lbs in shear. To adequately support the design loads, the bolts would have to be placed 9 in. on-center.

Without removing a ledger board, the material behind the ledger and its condition could not be verified.

6.7.7 Conclusions

The railing was checked for adequacy in supporting the building code design loads of 50 lb/ft in any direction at the top rail and 50 lb/ft² on the infill. The pickets were not adequate in bending or shear strength for supporting the 50 lb/ft. The shear stress in the pickets can be reduced by increasing the amount of space between the two fasteners used to attach the picket to the rim joist to at least 3 in. Using a larger picket, which would eliminate the shear strength problem, would increase the bending moment capacity. The fasteners used could not resist the withdrawal loads from the 50 lb/ft applied outward. The nails should be replaced with two 1/2 in. through bolts with washers. The pickets were adequate in supporting the 50 lb/ft² design load.

The joists used in Deck F were checked in shear strength, bending strength and deflection. They were found to be adequate in supporting the gravity loads as required by the building codes.

The post was checked for supporting the roof loads and the 200 lb concentrated load applied 42 in. from the top of the decking (the same height as the top of the railing). All posts were adequate in supporting the railing load and the gravity loads.

The fasteners used at attachment of the ledger board to the building were not adequately spaced. The spacing of the 1/2 in. bolts should be reduced to 9 in. on-center. The attachment at the ledger board could not be verified without removing the ledger. In a full inspection, the connection should be verified to ensure the presence of corrosion resistant flashing and proper attachment to the band joist.

With the modification listed, Deck F will meet all design criteria required by the current model building codes. Additional safety features observed for this structure are wood-to-wood bearing at the posts and redundancy on the side of the deck with the post. The redundancy is in that if a single bolt at the deck-to-house connection fails, the post will still be in place and it is unlikely that the entire structure will fail.

6.8 Balcony G

Balcony G was built in the 1970's as part of the original construction of the building. This inspection was non-invasive because the inspector was only authorized for a visual inspection. In practice, the professional inspector needs permission from the client (owner) to remove and replace balcony parts to obtain the required information from the site. It was surrounded on three sides by the building and had no other supports. The balcony was 11'-10" by 6'. Every surface and connections on the balcony were covered with several layers of gray paint. The paint was very thick in some places and peeling off in others.

6.8.1 Plan View and Typical Sections

The plan view of Balcony G is shown in Figure 6.21 and the railing section is shown in Figure 6.22.

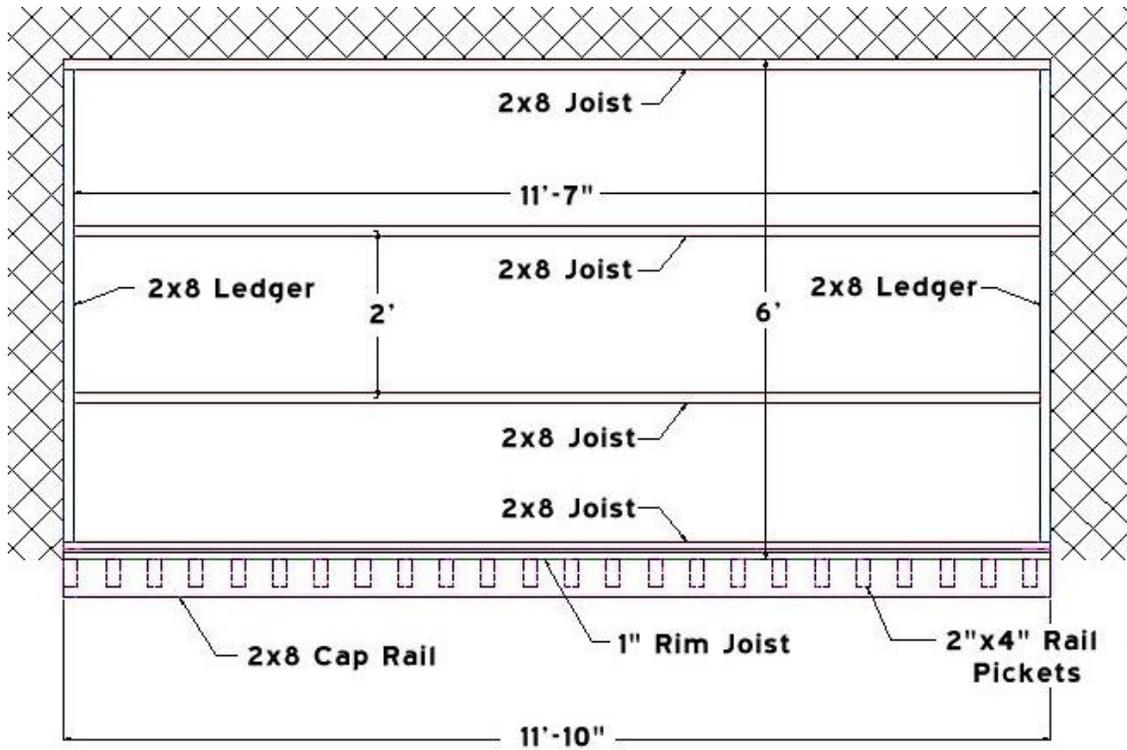


Figure 6.21. The plan view of Balcony G, showing the two ledgers and joists.

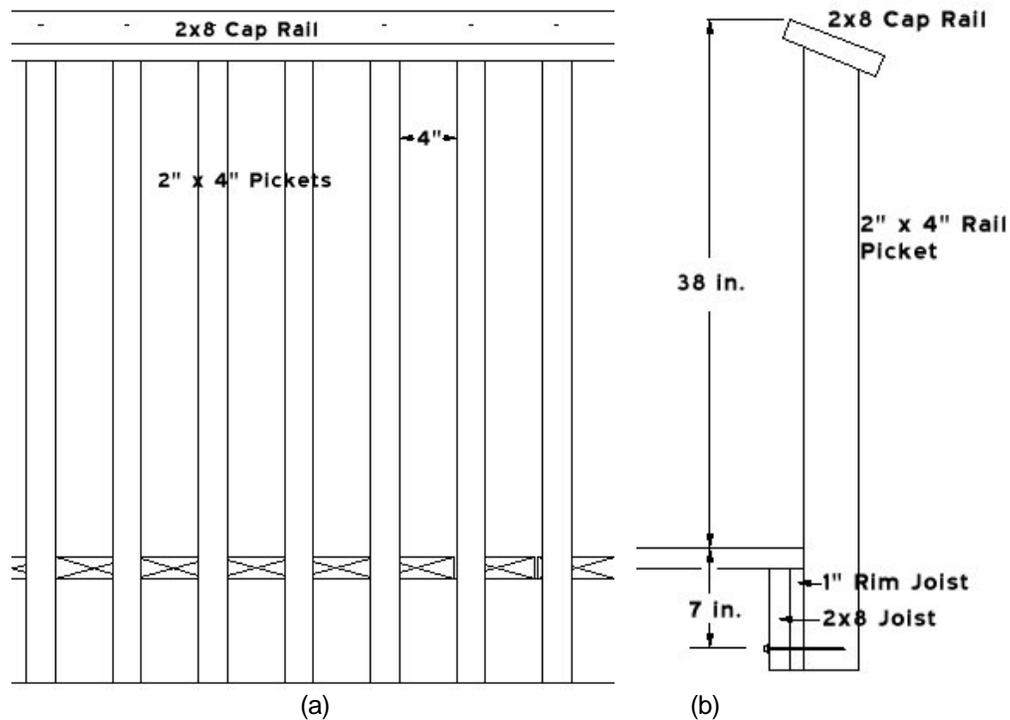


Figure 6.22. (a) The railing of Balcony G. The pickets were 2x4's (actual size) placed 6 in. on-center. (b) A typical picket from the railing.

6.8.2 Railing

The railing was 38 in. high with a 2x8 cap rail. The pickets were 2 in. by 4 in. (actual) and extended from the bottom of the rim joist to the cap rail. Each picket was attached to the balcony with one lag screw and the cap rail was attached to each picket with one lag screw. Since the lag screws could not be removed, the only exposed part, the head, was measured. The heads were 1/2 in. across the flats. Using Appendix L of the NDS-01, the lag screws were probably 5/16 in. diameter lag screws because the only screw with a 1/2 in. head width across the flats is a 5/16 in. diameter lag screw. The cap rail was angled so that water would run off away from the building. It was not determined how the cap rail was attached to the building because that area was covered with decorative trim. Two of the pickets were not in contact with the cap rail; the gap was 1/8 in. on the high side to 1/4 in. on the low side.

The penetration of the lag screws that hold the pickets to the deck could not be determined without removing them. This connection would have to resist 200 lb laterally or 1085 lb in withdrawal based on the 200 lb concentrated load in any direction as required by the IRC-2000 and 91 lb in withdrawal or 25 lb in shear based on the 50 lb/ft distributed load (directed horizontally outward or vertical downward) required by some building codes. Without knowing the penetration, the actual resistance is unknown. The rail pickets were analyzed under the 50 lb/ft distributed load. Assuming no resistance at the cap rail, the pickets are able to carry the 50 lb/ft distributed load.

Table 6.23 gives the results of checking the 50 lb/ft distributed load on the pickets and Table 6.24 shows the results of checking the 50 lb/ft² loads required by building codes on the railing pickets and infill.

Table 6.23. The actual and allowable design values for the railing pickets in Balcony G as determined by using the methods in the NDS-01.

	Allowable Design Value based on assumed No. 3 Southern Pine	Design Stress or Deflection
Bending Stress (psi)	1594	141
Shear Stress (psi)	212	17
Deflection (in)	3.75	0.03

*The allowable deflection on a guardrail is $h/12$ as stated in the Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (ICBO, 2002).

Table 6.24. The actual and allowable design values for the railing infill in Balcony G as determined by using the methods in the NDS-01.

	Allowable Design Value based on assumed No. 3 Southern Pine	Design Stress
Bending Stress (psi)	1062	50
Shear Stress (psi)	210	5

6.8.3 Decking

The deck boards were 2x6's covered with gray paint. The boards were attached to the joists with one nail at each interface, but were not loose. The decking was level. Discoloration was seen on the underside of the decking and the renter described it as marks from dripping water.

6.8.4 Joists

The joists were 2x8's attached to the ledger boards with hangers. The type of hanger was unknown because of the paint coverage, but some rust did show through the paint. All nail holes on the hangers were filled, however, the size of the nails was not determined. The joist closest to the building was attached to the building with nails at approximately 10 in. on-center. Hangers were not used to support this joist. The results of checking the joists according to methods in the NDS-01 are shown in Table 6.25. The joists were not adequate in bending stress or deflection.

Table 6.25. The actual and allowable design values for the joists in Balcony G as determined by using the methods in the NDS-01.

	Allowable Design Value based on assumed No. 3 Southern Pine	Design Stress or Deflection
Bending Stress (psi)	805	2144
Shear Stress (psi)	170	112
Live Load Deflection (in.)	0.39*	0.73

* Based on L/360

6.8.5 Attachment to building

The two ledger boards were 2x8's attached to the building with fasteners as shown in Figure 6.23.

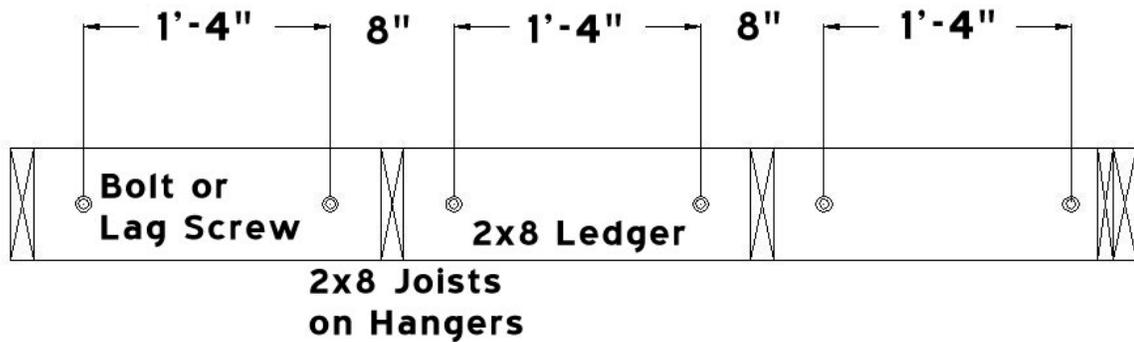


Figure 6.23. The fastener pattern of the ledger board of Balcony G.

Each fastener had a 5/8 in. head, which, assuming a lag screw, is a 7/16 in. lag screw (according to Appendix L of the NDS-01). For an actual inspection, the size and type of connector should be determined. The material between the ledger and the framing of the building could not be determined without removing the ledger boards. Table 6.26 shows the allowable design loads and the actual loads on the fasteners. The allowable design loads were based on the assumption that the ledger and the rim joist of the building were in contact. This assumption was not verified and the actual connection details are unknown.

Table 6.26. The actual and allowable design values for the fasteners in Balcony B as determined by using the methods in the NDS-01.

	Allowable Design Value (lbs)*	Design Value (lbs)
7/16"x3" Lag Screw	Does not meet 4D penetration	621
7/16"x4" Lag Screw	118	621
3/8" Bolt	95	621

* Allowable design value based on No. 3 Southern Pine Ledger, Spruce-Pine-Fir Framing and no material between the ledger and framing.

In one corner, the ledger bowed out away from the building 3/4 in. This deformation is shown in Figure 6.24. This deformation produced a gap between the ledger and the element it was attached to, therefore weakening the connection by reducing the design value of the bolt or lag screw.



Figure 6.24. The ledger board bowed out $\frac{3}{4}$ " away from the building in one corner.

6.8.6 Conclusion

The actual material for the railing was unknown. Based on the assumption of No. 3 Southern Pine, the railing was able to resist the 50.lb/ft² load over the infill. The strength of the lag screws that held the railing onto the deck was unknown. It is recommended that each lag screw be replaced with two $\frac{1}{2}$ " thru bolts with washers, which would be able to adequately resist the loads required by building codes.

The joist material was unknown because of the paint coverage and was assumed to be No. 3 Southern Pine. With this assumption, the joists are not adequate in bending or deflection. The joists should be replaced with 2x12 No. 2 Southern Pine (or better) elements.

The method of attaching the balcony to the building was unknown. The connection was only observed from the outside so the unknown details are: fastener size and type, presence of flashing, what part of the building framing receives the lag screws or bolts, and the material between the ledger and framing. Regardless of the length and diameter of the fasteners, the spacing is not adequate. The ledger must be removed and replaced as per the detail in Section 4.3.

A sign posted on the balcony stated, "Balcony structure provides for no more than 10 persons of average weight & sundry furniture for a maximum of no more than 2500 lb (total)." The 2500 lb is a 35 psf live load that, with a 10 psf dead load, could not be supported with the assumed 2x8 No. 3 Southern Pine joists or by the assumed 7/16 in. lag screws. Therefore, based on a visual inspection, the balcony cannot support the posted load limit of 2500 lb.

The quality of the materials used on Balcony G could not be determined because of the paint coverage. Because of the varying thickness of the paint, the pick test could not be used. Paint could hide deterioration of structural elements and connections.

6.9 Balcony H

Balcony H was a three-year-old balcony that was part of the original construction of the building. Grade and treatment stamps were visible on the joists, deck boards and ledgers, but were not entirely clear. One typical stamp is shown in Figure 6.25. The word "Durapine" is visible. According to a distributor's

website, all Durapine products are No. 1 or Better Southern Pine and are treated with CCA to a retention of at least 0.40 lb/ft³.



Figure 6.25. The stamp visible on the lumber used on Balcony H.

The inspection of Balcony H was non-invasive; therefore none of the structural elements or connections were removed. In a professional inspection, permission from the client would be needed to temporarily remove structural elements and fasteners to inspect and obtain the required information.

6.9.1 Plan View and Typical Sections

The plan view of Balcony H is shown in Figure 6.26. An electric fireplace protruded out from the building and the balcony was framed in around it. The balcony was surrounded on three sides by the building and the occupants were protected on the open side by a metal railing. A typical section of this railing is shown in Figure 6.27.

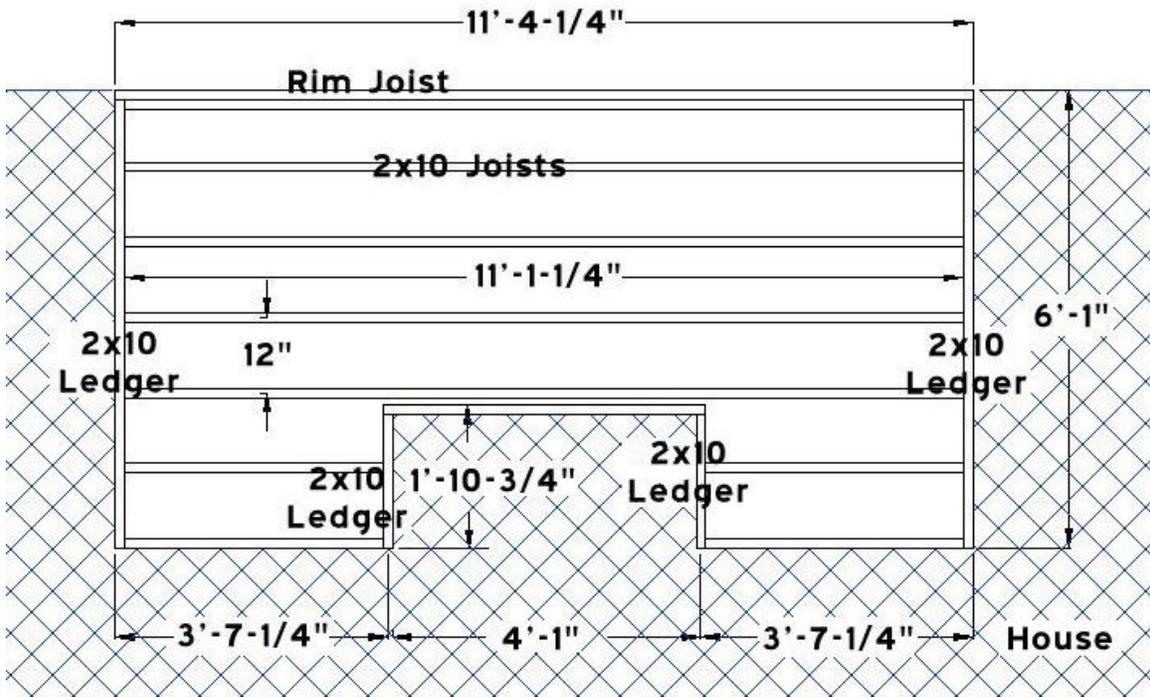


Figure 6.26. The plan view of Balcony H showing the joists, ledgers and the rim joist.

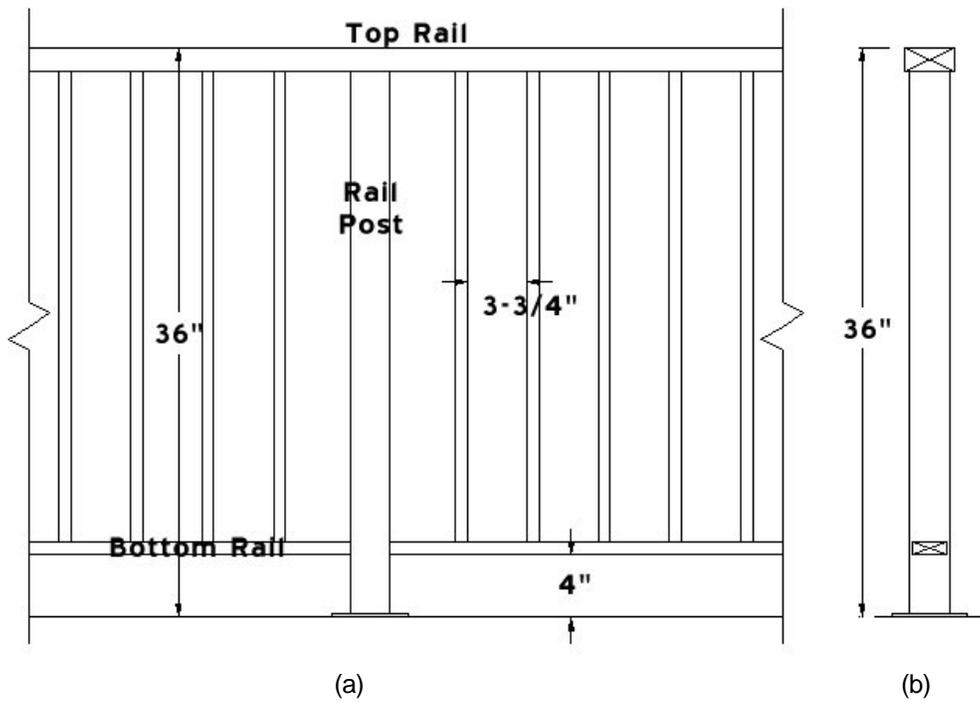


Figure 6.27. (a) A typical railing section of the metal railing on Balcony H. (b) A view of the railing post in the center of the railing.

6.9.2 Railing

The railing on Balcony H was metal. It was 36 in. above the deck surface and the largest opening was 4 in. The railing was screwed into the siding of the building at the two ends and supported by a 2.5 in. square post in the center. The post was anchored into the deck with four lag screws. The lag screws were not removed for this visual inspection, but in a professional inspection, with permission from the homeowner, one lag screw would be temporarily removed to note its condition and measured to find its design strength. In this case, the only visible part of the lag screw was the head, which was measured across the flats and found to be 1/2 in. Appendix L of the NDS-01 lists the only lag screws with heads of this size to be 5/16 in. lag screws. The length was unknown.

Specifications for the testing of metal railings are listed in ASTM standards E984-88, E935-00 and E985-00. The inspection of the metal railings is beyond the scope of this thesis.

6.9.3 Decking

The deck boards of Balcony H were 2x6 Durapine (Southern Pine) boards. Several of the nails used to attach the boards to the joists were over-driven and some nails were popping out near the edge of the balcony.

6.9.4 Joists

The joists were 2x10 Durapine (Southern Pine) attached to the ledger boards with hangers. The type of hangers could not be seen due to their height.

All nail holes were filled, but the nail type could not be determined without removing a sample of nails. The spans of the joists were 41 in. and 133 in. Both spans were checked under the gravity loads required by the building code (60 psf live and 10 psf dead) and found to be adequate. The results from checking the 133 in. joist span in bending stress, shear stress and deflection are shown in Table 6.27.

Table 6.27. The actual and allowable design values for the joists in Balcony H as determined by using the methods in the NDS-01.

	Allowable Design Value	Design Stress or Deflection
Bending stress (psi)	1271	437
Shear stress (psi)	170	36
Live Load Deflection (in.)	0.31*	0.06

*Deflection based on L/360.

6.9.5 Attachment to building

The ledgers for Balcony H were 2x10 Durapine (Southern Pine) boards attached to the building with 1/2 in. bolts with washers. The placement of the bolts is shown in Figure 6.28. The bolts are spaced approximately 12 in. on-center with two bolts in the center of the span and at the ledger ends.



Figure 6.28. One of the ledger boards on Balcony C showing the spacing of the 1/2 in. thru bolts used to hold the ledger onto the building.

Using the yield mode equations in the NDS-01 and assuming the ledger is attached to a 2x_ Spruce-Pine-Fir (SPF) band joist with nothing in between them, each bolt can carry 132 lb in shear. In the configuration shown in Figure 6.28, each bolt must carry 330 lb. Where there are two bolts, each must carry 165 lb.

On the two smaller ledger boards, two 1/2 in. bolts were present. These bolts were adequate for supporting their tributary area of the balcony.

6.9.6 Conclusion

Code conformance of the metal railing with respect to loads on Balcony H was not verified in this inspection. In a professional inspection, the load capacity of the railing and it's connection to the balcony and the building would be checked and reported. The height and opening sizes were both in conformance with the IRC-2000 requirements.

The Durapine lumber used was, according to literature, treated to 0.40 lb/ft³ retention of CCA. Standard C15-02 of the American Wood Preserver's Association recommends minimum retentions of 0.25 lb/ft³ for decking and joists

used above ground. The retention of the lumber used to construct Balcony H was above the recommended CCA retentions.

The joists on Balcony H were adequate in bending strength, shear strength, and deflection under the building code gravity loads of 60 psf live and 10 psf dead.

The spacing of the 1/2 in. bolts on the longer ledger boards was not adequate for supporting the gravity loads on the balcony. Each 1/2 in. through bolt can support 132 lb in shear assuming “wet use” and a single SPF band joist. To support the required design loads, the bolts must be placed 4.75 in. on-center. The condition of the bolts was not determined and the actual material behind the ledger board was not verified.

Balcony H was supported entirely by mechanical connections. The addition of wood-to-wood bearing could not easily be introduced into the existing structure without compromising other building code requirements, such as headroom.

6.10 Conclusion

This chapter contains inspection data and analyses for several residential decks and balconies. The inspections yielded several different deck designs from simple to complex. Design details found were continuous posts that served to support both the deck and the railing, steel beams and posts, metal railings used with wooden flooring, cross bracing, joists parallel to the main structure, masonry screws, and painted decks.

Design deficiencies found included undersized structural elements, inadequate connections, unsafe railings, uneven stairways and large openings. Table 6.28 is a summary of all decks and balconies inspected for this thesis. An “X” in any field indicates a deficiency for the various inspection categories – structural, condition assessment, code conformance, presence of redundant element, and overall condition. As seen in the table, structural deficiencies with the ledger attachment and connections were common, building code conformance at the railings was poor and few structures had structural redundancy. Overall, every deck or balcony inspected had a deficiency in some aspect of the design or construction.

Table 6.28. A summary of deficiencies found in all decks and balconies inspected. An “X” indicates a deficiency in that category for the deck or balcony. A “NC” means the category was not evaluated for that deck or balcony due to constraints of the study. A “NA” means that category is not applicable to that deck or balcony.

	Structural			General Condition			Code Conformance					Redundant	Overall
	Framing	Ledger Attachment	Other Connections	Framing	Railing	Fasteners	Railing Post	Railing Infill	Railing Height	Open Space	Stairway		
Deck A		X	X		X		X	X		X	X		X
Deck B	X	X	X		X	NC	X	X	X		NA		X
Deck C		X	X			NC		X			NA	X	X
Deck D		NA	X			NC	X	X	X	X	X		X
Deck E	X	X	X	X		NC	X	X	X	X	NA	X	X
Deck F		X	X			NC	NA	X			NA		X
Balcony G	X	X	X	NC	NC	NC	NA				NA	X	X
Balcony H		X				NC	NC	NC			NA	X	X

Throughout these inspections, the process was streamlined and complete worksheets were created to aid in taking notes on the decks and balconies. Deficiencies were found in every deck inspected and some were critical for structural safety. These sample inspections proved the need for regular, comprehensive inspections of residential wood decks and balconies by experienced professionals. Also, it was evident that many deficiencies existed at the time of original construction, suggesting that decks and balconies may not be subject to building code inspections, at least in the geographic area sampled in this study.

7. Summary and Conclusions

7.1 Summary

The review of literature, conversations with building inspectors, and other professionals brought focus to the process of creating a methodology for deck and balcony structural analysis and the preparation of an inspection manual. Included in this thesis are an analysis of deck details provided to homeowners applying for building permits, an investigation on strength of lag screws, proposals for adequate deck-to-house attachment construction, and a description on how to perform the “pick test” for detecting wood decay.

Eight decks and balconies were inspected and analyzed from the point of view of a professional engaged in deck and balcony inspections. The overall condition, structural adequacy, and building code conformance of each deck or balcony was summarized to find typical design examples and deficiencies. Experience gained from these inspections was used to create *A Manual for the Inspection of Residential Wood Decks and Balconies* and worksheets that would aid the professional inspector in the collection of information at the site for later analysis. The draft manual is presented in Appendix B, and after extensive outside reviews, it will be submitted to a publisher for distribution as a stand-alone reference for inspectors and other interested parties.

7.2 Conclusions

A methodology for inspection of residential wood decks and balconies was created through research, conversations with building officials, wood connection researchers, and inspection experience obtained in the Blacksburg area. The methods presented are for dimension lumber (2x_ material) construction that is accessible to a building inspector or consultant.

The *Manual for the Inspection of Residential Wood Decks and Balconies* was written and contains information on structural issues, occupant safety, wood decomposition issues, engineering analysis methods, and other information and concepts unique to the safe performance of wood decks and balconies. The manual also contains a full sample report to serve as a guide. Two sets of worksheets were created for the inspector to reproduce and take to the site to facilitate data collection.

Awareness of the problems with decks and balconies has been increased by publications in several trade journals (Anderson et al., 2002; Cushman 2002; Anderson et al., 2003) and a presentation at a Virginia Tech continuing education course. These efforts by the Virginia Tech Team will continue after this thesis is completed.

7.3 Recommendations

Further research is needed to resolve issues unique to engineered wood products, such as open web floor trusses and I-joists. The ledger board connection to the house framing should be the primary focus of research for the

engineered joist products. New technology being introduced in deck building, such as the Maine Deck Bracket, plastic lumber, EB-TY, and others, is creating complexity and uncertainty in the inspection process. Research on their quality and limitations is needed for their evaluation in the field.

8. References

- American Forest & Paper Association (AF&PA). 1997. *National Design Specification for Wood Construction*. Washington, D.C.: American Forest & Paper Association, Inc.
- American Forest & Paper Association (AF&PA). 2001. *National Design Specification for Wood Construction*. Washington, D.C.: American Forest & Paper Association, Inc.
- AF&PA. 2001. *National Design Specification for Wood Construction Supplement: Design Values for Wood Construction*. Washington, D.C.: American Forest & Paper Association, Inc.
- AF&PA. 1999. *General Dowel Equations for Calculating Lateral Connection Values*. Technical Report 12. Washington, D.C.: American Forest & Paper Association. <http://www.awc.org/Publications/TR/index.html>
- American Institute of Steel Construction (AISC). 1994. *Manual of Steel Construction. Load & Resistance Factor Design, Volume 1: Structural Members, Specifications & Codes*. Second Edition. U.S.A: American Institute of Steel Construction.
- American Lumber Standard Committee. 1999. *American Softwood Lumber Standard, Voluntary Product Standard*. DOC PS 20-99. Washington D.C.: U.S. Government Printing Office.
- American Society of Civil Engineers (ASCE). 1995. *Minimum Design Loads for Buildings and Other Structures*. ASCE 7-95. New York, NY: ASCE.
- American Society of Mechanical Engineers (ASME). 1997. *Square and Hex Bolts and Screws (inch series)*. B18.2.1 -1996. New York: ASME.
- ASME. 1997. *Wood Screws (inch series)*. B18.6.1 -1997. New York: ASME.
- American Society for Testing and Materials. 2002. *Standard Specification for Driven Fasteners: Nails, Spikes, and Staples*. F1667-02. ASTM: West Conshohocken, PA.
- American Society for Testing and Materials (ASTM). 1992. *Standard Test Methods for Use and Calibration of Hand-Held Moisture Meters*. D4444-92e1. Philadelphia, PA: American Society for Testing and Materials.

ASTM. 1993. *Standard Test Methods for Performance of Permanent Metal Railing Systems and Rails for Buildings*. E935-93(1998). Philadelphia, PA: American Society for Testing and Materials.

ASTM. 2000. *Standard Specification for Permanent Metal Railing Systems and Rails for Buildings*. E985-00e1. Philadelphia, PA: American Society for Testing and Materials

American Wood-Preserver's Association (AWPA). 2002. Standard C15-02: Wood for commercial-residential construction preservative treatment by pressure process. <http://www.awc.org/HelpOutreach/fac/C15-02.pdf>. AWPA. PO Box 5690, Granbury, TX 76049.

American Wood Preserves Institute (AWPI). 2002. Manufacturers to transition to new generation of wood preservatives. <http://www.preservedwood.com/news/020212ccatrans.html>. AWPI. 2750 Prosperity Avenue, Suite 550, Fairfax, VA 22031.

Anderson, C.A., J.R. Loferski, and F.E. Woeste. 2002. Wood Bits: Detecting early wood decay. *Building Standards* 71(4): 6-7.

Anderson, C.A., F.E. Woeste, and J.R. Loferski. 2003. Practical engineering: Deck attachment and inspection issues. *Journal of Light Construction*. In Press.

Anderson, L.O., T.B. Heebink, and A.E. Oviatt. 1972. Construction guides for exposed wood decks. *Agriculture Handbook*. 432. USDA.

Baker, A.J. 1980. Corrosion of metal in wood products. *Durability of Building materials and components*. ASTM STP 691. Philadelphia, PA: American Society for Testing and Materials. p. 981-993.

Baker, A.J. 1988. Corrosion of metals in preservative-treated wood. In: *Proceedings of Wood Protection Techniques and the Use of Treated Wood in Construction*. October 28-30, 1987. Memphis, TN. Madison, WI: Forest Products Society. Proc. No. 47358. pp. 99-101

Baker, A.J. 1992. Corrosion of nails in CCA- and ACA-treated wood in two environments. *Forest Products Journal* 42(9):39-41.

Bohnhoff, D.R. 2002. Hidden danger. *Wood Design Focus* 12(1):19-25

Building Officials and Code Administrators (BOCA). 1978. The BOCA basic property maintenance code: The model property maintenance regulations for the protection of public health, safety and welfare. Chicago, IL: Interstate Printers and Publishes

- BOCA. 1996. The BOCA national building code. Chicago, IL: BOCA
- Clausen, C.A. 1997. Immunological detection of wood decay fungi: An overview of techniques developed from 1986-present. *International Biodeterioration and Biodegradation* 39(2-3):133-143
- Clausen, C.A., R.J. Ross, J.W. Forsman, and J.D. Balachowski. 2001. Condition assesment of roof trusses of Quincy Mine Blacksmith Shop in Keweenaw National Historic Park. Research Note. FPL-RN-0281. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.
- Cory, S. 2001. *Decks: Planning, Designing, Building*. Upper Saddle River, NJ: Creative Homeowner.
- Council of American Building Officials (CABO). 1992. CABO one and two family dwelling code. Falls Church, VA: CABO
- CABO. 1995. CABO one and two family dwelling code. Falls Church, VA: CABO
- Cushman, T. 2002. Notebook: Deck disasters spotlight faulty connections. *Journal of Light Construction* 20(12):17,26.
- Dannenberg, R.W. 1988. Deck Construction from A to Z. In: *Proceedings of Wood Protection Techniques and the Use of Treated Wood in Construction*. October 28-30, 1987. Memphis, TN. Madison, WI: Forest Products Society. pp 12-16
- DeBonis, A.L. 1999. Inspecting mill-type buildings for decay. *Wood Design Focus* 10(1): 6-10.
- DeGroot, R.C., Gjovik, L.R. and A.J. Baker. 1981. Preservative-Treated Wood for Foundations. FPL-0245. USDA, Forest Service, Forest Products Laboratory.
- Emerson, R., Pollock, D., McLean, D., Pellerin, R. and R. Ross. 2002. Ultrasonic inspection of large bridge timbers. *Forest Products Journal*. In Press.
- Eslyn, W.E. and J.W. Clark. 1979. Wood bridges – decay inspection and control. *Agricultural Handbook*. 557. Washington, D.C.: US Dept of Agriculture
- Fairfax County. 1998. Typical Deck Details. <http://www.fairfaxcounty.gov/dpwes/publications.htm>. The Department of Public Works and Environmental Services. 12055 Government Center Parkway, Director's office, Suite 659, Fairfax, VA 22035.
- Falk, R.H. and A.J. Baker. 1993. Fasteners for exposed wood structures. *Wood Design Focus* 4(3):14-17.

Forest Products Laboratory. 1985. Inspection of Wood Beams and Trusses. NAVFAC MO-111.1. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Forest Products Laboratory. 1999. Wood Handbook: Wood as an Engineering Material. Gen. Tech. Rep. FPL-GTR-113. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Foley, B. 2002. Personal Communication. Chief Structural Engineer, Fairfax County, VA.

Freas, A. 1982. *Evaluation, Maintenance and Upgrading Wood Structures: A Guide and Commentary*. New York, NY: American Society of Civil Engineers

Gaby, L.I. and J.E. Duff. 1978. Moisture content changes in wood deck and rail components. Research Paper No. SE-190. Athens, GA: U.S. Department of Agriculture, Forest Service, Southeastern Forest Experiment Station.

Goodell, B., J. Jellison, and J.P. Hosli. 1988. Seriological detection of wood decay fungi. *Forest Products Journal* 38(3):59-62

Grossthanner, O.A., D.B. Brakeman, and N.D. Wood. 1991. Metal-plate connected wood truss inspection: part 1. *Wood Design Focus* 2(4):17-19

IBCO. 2002. Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems. Whittier, CA: IBCO Evaluation Service, Inc.

International Code Council. 1998. International One- and Two- Family Dwelling Code. Falls Church, VA: International Code Council.

International Code Council. 2000. International Building Code. Falls Church, VA: International Code Council, Inc.

International Code Council. 2000. International Residential Code for one- and two-family dwellings. Falls Church, VA: International Code Council, Inc.

Lanius, R.M., R. Tichy and W.M. Bulleit. 1981. Strength of old wood joists. In: *Journal of the Structural Division: Proceedings, American Society of Civil Engineers*. December 1981. New York, NY: American Society of Civil Engineers. 107(ST12): 2349-2363.

Lemaster, R.L., F.C. Beall, and V.R. Lewis. 1997. Detection of termites with acoustic emission. *Forest Products Journal* 47(2):75-79

Marshall, P. 2002. *Complete Decks, Plan and Build Your Dream Deck*. Des Moines, IA: Better Homes and Gardens Books.

McDonald, K.A., R.H. Falk, R.S. Williams and J.E. Winandy. 1996. Wood decks: materials, construction and finishing. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest products Laboratory.

McLain, T.E. 1992. Strength of lag screw connections. *Journal of Structural Engineering* 118(10): 2855-2871.

National Frame Builders Association (NFBA). 2002. Current list of building professionals. <http://www.postframe.org/locate.htm>. NFBA, 4840 W. 15th Street, Suite 1000, Lawrence, KS, 66049.

Pneuman, F.C. 1991. Inspection of a wood-framed warehouse-type structure. *Wood Design Focus* 2(4):9-12

Robertson, R. 2002. Personal communication. Building Official, Chesterfield County, VA.

Ross, R.J., R.C. DeGroot, W.J. Nelson and P.K. Lebow. 1997. The relationship between stress wave transmission characteristics and the compressive strength of biologically degraded wood. *Forest Products Journal* 47(5): 89-93

Ross, R.J., R.F. Pellerin, N. Volny, W.W. Salsig, and R.H. Falk. 1999. Inspection of timber bridges using stress wave timing nondestructive evaluation tools: a guide for use and interpretation. Gen. Tech. Rept. FPL-GTR-114. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Ross, R.J., and R.F. Pellerin. 1991. Nondestructive testing for assessing wood members in structures: A review. Gen. Tech. Rept. FPL-70. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Shook, S.R. and I.L. Eastin. 2001. A characterization of the U.S. residential deck material market. *Forest Products Journal* 51(4):28-36

Simpson Strong-Tie. Wood construction connectors. Catalog C-97. Simpson Strong-Tie Company, Inc.

Smith, A. 2002. Repairing cantilevered balconies. *Journal of Light Construction* 20(12): 67-71.

Southern Building Code Congress International (SBCCI). 1997. Standard Building Code. Birmingham, AL: SBCCI

SBCCI. 1976. Standard Housing Code. Birmingham, AL: SBCCI

Squirrell, J.P. and R.W. Clarke. 1987. An investigation into the condition and conservation of the hull of the Mary Rose. Part I: Assessment of the hull timbers. *Studies in Conservation* 32:153-162.

Stern, E.G. 1952. Immediate vs. delayed holding power of nails. Report No. 8. Virginia Polytechnic Institute Wood Research laboratory. Blacksburg, VA. 12p.

TrusJoist MacMillan. 1998. Parallam® PSL Preservative Treatment. <http://www.tjm.com/PDFFiles/3425.pdf>. TrusJoist MacMillan. 2600 East Amity Road, Boise, ID, 83716.

Verrall, A.F. and T.L. Amburgey. 1980. *Prevention and Control of Decay in Homes*. U.S. Department of Agriculture, Forest Service and the Department of Housing and Urban Development.

Wagner Electronic Products. 1993. Wood friendly moisture monitoring & management. Wagener Electronics, Inc.

Wilcox, W.W. 1978. Review of literature on the effects of early stages of decay on wood strength. *Wood and Fiber* 9(4):252-257.

Wilcox, W.W. 1983. Sensitivity of the 'pick test' for field detection of early wood decay. *Forest Products Journal* 33(2):29-30

Williams, R.S. and M. Knaebe. 2002. Iron Stain on Wood. Finish Line. USDA, Forest Service, Forest Products Laboratory.

Wolmanized Wood. 1998. How to Build the Best Deck. Pamphlet WOL-101-R9. Hickson Corporation.

Yang, D.Q. 2001. A method for the prevention of wood decay from nail penetration. *Forest Products Journal* 51(4):67-70.