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Editorial

Deck safety continues to be an important national problem. Engineered design has been hampered by knowledge gaps on structural deck loads – especially lateral loads. We believe this information is vital for registered design professionals to create safe and efficient engineered designs for decks and balconies.

In the first three articles, we explore lateral loads on a 12 ft x 12 ft deck for wind, seismic and occupancy. One motivation was to find out whether deck lateral loads were a significant concern for regions with high wind and earthquake risks. The results were surprising and should be of interest to all design professionals.

Wind and seismic loads can be calculated using the provisions of *ASCE 7-10 Minimum Design Loads for Building and Other Structures*. ASCE 7-10 procedures are complicated and require engineering judgment. In the first two articles, we demonstrate the ASCE 7-10 methodology through example calculations for a 12 ft x 12 ft deck. Of course, the results of the analyses would vary for decks with different sizes and aspect ratios. Within the specific constraints of our example deck, we found that while wind loads generally control over seismic, the wind loads would not pose much of a design challenge except for hurricane and special wind regions.

The building codes and ASCE 7-10 are silent on the subject of lateral loads due to occupant movement, with the exception of grandstands, bleachers, and stadium seating. The third article describes laboratory experiments on full-size decks with two types of occupant loadings: cyclic side-sway and impulse (run and jump stop). We found that lateral loading from occupants can exceed the worst-case design loads from either wind or seismic. The key point being that *occupant loading can occur on any deck, anywhere and have a structural impact as great or greater than that from wind or seismic design loads*.

Armed with a better understanding of lateral loads, we sought to improve our understanding of load transfer from decks to the house floor framing and diaphragm. Two 12 ft x 12 ft decks were laterally loaded to determine their ultimate strengths and stiffnesses. To measure load paths, the decks were connected to a portion of a light-frame wood diaphragm to simulate realistic support conditions. Decks were tested with and without tension hold-down connectors, and each lag screw in the deck ledger was instrumented to monitor loads. The study yielded counterintuitive results that will help guide new design solutions and products to resist lateral loads.

I hope you enjoy this issue of *Wood Design Focus*.

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Wind Load Determination for Residential Decks

Garrett H. Lyman, and Donald A. Bender, P.E., Ph.D.

Introduction

The safety of exterior elevated decks and porches is an important national issue due to numerous documented structural collapses that have resulted in serious injuries and, in some cases, deaths (Shutt 2011; Legacy Services 2010). Many deck and porch failures occur at loads well under design loads and occur without any warning. The primary causes for failures are from (1) deficient connections between the deck ledger and the house rim board, and (2) deficient guardrail systems. Frequently, related decay and corrosion of fasteners also contributed to deck failures (Carradine et al. 2007; Carradine et al. 2008). The 2009 *International Residential Code (IRC)* Section R502.2.1 (ICC 2009b) and the 2009 *International Building Code* (IBC) Section 1604.8.3 (ICC 2009a) state:

> "Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and *lateral* loads as applicable." (italics added for emphasis)

Vertical loads on decks, such as occupancy and snow, are straightforward to calculate using the provisions of the 2009 IBC and ASCE/SEI 7-10 *Minimum Design Loads for Building and Other Structures* (ASCE 2010). Determination of lateral loads on decks from seismic, wind, and occupancy is more challenging. Calculation of wind loads using ASCE 7-10 can be complicated and requires engineering judgment.

This paper is part of a larger project to characterize lateral loads on residential decks caused by seismic, wind, and occupancy. In this paper, we focus on wind load determination. Specific objectives of this paper are to highlight the differences between ASCE 7-05 and ASCE 7-10 for wind analysis, illustrate a method and example calculation for determining the wind loads on residential decks, and provide a parameter sensitivity study to demonstrate the relative magnitudes of wind loads in various regions in the US for the example deck.

Overview of Load Determination Using ASCE 7 Load Standard

ASCE 7 is a standard for calculating minimum loads for the design of buildings and other structures as required by building codes. Appropriate load combinations for allowable stress design (ASD) and load and resistance factor design (LRFD) are presented in ASCE 7. ASCE 7 is the primary reference used by designers for load calculation in the US to determine dead, live, flood, snow, ice, rain, wind and seismic loads. This document is cited by the model building code and is revised every five years. The most recent edition is ASCE 7-10.

Changes to ASCE 7-10 (from 2005 edition)

The wind provisions for ASCE 7-10 have been updated and completely reorganized. The wind provisions have been expanded from one chapter to six. The 2010 version provides three new wind speed maps that represent wind events with mean recurrence intervals (MRI) of 300, 700 and 1700 years (ASCE 7-05 had MRI of 50 years). The rationale was to incorporate the risk categories into the wind speeds and to have MRIs that were consistent with strength design format. So, for LRFD design, the load factor for wind changed from 1.6 to 1.0. Similarly for ASD design, the load factor for wind changed from 1.0 to 0.6. After appropriate factoring of wind loads for ASD or LRFD, the resulting loads from ASCE 7-10 are similar as those calculated using ASCE 7-05 for most cases.

One of the analytical procedures permitted by ASCE 7 is the directional procedure for building appurtenances and other structures (such as solid freestanding wall and solid freestanding signs, chimneys, tanks, open signs, lattice framework, and trusses towers). We used this procedure to calculate wind loads on decks. The directional proce-

Table 1. Steps to Determine Wind Load on Residential Decks Using ASCE 7
Step 1: Determine risk category of structure (Table 1.5-1)
Step 2: Determine the basic wind speed, V, for the applicable risk category (Figure 26.5-1A)
Step 3: Determine wind load parameters: Wind directionality factor, K_d (Table 26.6-1) Exposure category B, C, or D (Section 26.7) Topographic factor, K_{zt} (Figure 26.8-1) Gust effect factor, G (Section 26.9)
Step 4: Determine velocity pressure exposure coefficient, K_z (Table 29.3-1)
Step 5: Calculate velocity pressure, q_z (Eq. 29.3-1)
Step 6: Determine force coefficient, C_f Open signs, lattice frameworks (Figure 29.5-2) Chimneys, tanks, rooftop equipment (Figure 29.5-1)
Step 7: Calculate wind load, F (Eq. 29.5-1)

dure in ASCE7-10 is identical to the analytical procedure in ASCE 7-05 except for the determination of the basic wind speed V.

Method for Determining Wind Load

The directional procedure or analytical procedure is one method permitted in ASCE 7 to determine wind loads and applies to residential decks. Most residential decks are in compliance with the conditions in ASCE 7-10 Section 29.1.2. Table 1 summarizes the steps to determine wind loads.

Example

All references to tables and figures in this example refer to ASCE 7-10.

Assumptions

- Worst case wind speed, V (southern tip of Florida)
- Exposure Category C
- Deck height of 10 ft
- Topographic Factor of 1.0
- Allowable stress design (ASD) format

Step 1: Determine risk category of structure

Risk Category II (Table 1.5-1)

Note: Residential decks do not fit the structures in categories I, III, and IV and therefore fall under risk category II

Step 2: Determine the basic wind speed for the applicable risk category

V = 180 mph (Fig 26.5-1A)

This was the worst-case wind speed on the southern tip of Florida. This wind speed has a mean recurrence interval of 700 years.

Step 3: Determine wind load parameters: wind directionality, exposure category, topographic and gust effect factors

Wind directionality factor for open signs and lattice framework

K_d = 0.85 (Table 26.6-1)

Assumed exposure category

Exposure Category C (Section 26.7)

Assumed topographic factor (this factor could be greater than one for sites with isolated hills, ridges or escarpments as determined in Section 26.8.1

K_{zt} = 1.0 (Fig 26.8-1)

The fundamental frequency for this deck is assumed to be greater than 1Hz and is therefore considered rigid according to Section 26.2. ($f_n = 1/T_n$ = 8.93 Hz) In this example the gust factor was determined using ASCE7-10 Eqn. 26.9-6, but if a deck is rigid then the gust factor is permitted to be taken as 0.85.

Gust Effect Factor, G, for rigid structure (Section 26.9)

$$h = 10 \text{ ft}$$
$$z = 0.6h = (0.6)(10 \text{ ft}) = 6 \text{ ft}$$
$$z_{\min} = 15 \text{ ft (Table 26.9-1)}$$
$$c = 0.2 \text{ (Table 26.9-1)}$$
$$I_z = c\left(\frac{33}{z_{\min}}\right)^{\frac{1}{6}} = 0.2\left(\frac{33}{15}\right)^{\frac{1}{6}} = 0.23$$

$$B = 12 \text{ ft}$$
$$l = 500\text{ft (Table 26.9-1)}$$
$$\varepsilon = 1/5 \text{ (Table 26.9-1)}$$
$$I_Z = l\left(\frac{z_{\min}}{33}\right)^2 = 500\left(\frac{15}{33}\right)^2 = 427.1$$
$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{12+10}{427.1}\right)^{0.63}}} = 0.96$$
$$g_0 = 3.4$$
$$g_V = 3.4$$
$$G = 0.925\left(\frac{1+1.7g_0 I_Z Q}{1+1.7g_V I_Z}\right)$$
$$= 0.925\left(\frac{1+1.7(3.4)(0.23)(0.96)}{1+1.7(3.4)(0.23)}\right) = 0.9$$

Step 4: Determine velocity pressure exposure coefficient

Velocity pressure exposure coefficient was determined using Table 29.3-1 for a height above ground of 10 ft and Exposure C

K_z = 0.85 (Table 29.3-1)

Step 5: Calculate velocity pressure

$q_z = 0.00256\, K_z K_{zt} K_d V^2$

$= 0.00256(0.85)(1)(0.85)(180^2) = 59.93$ psf (Eqn 29.3-1)

Step 6: Determine force coefficients

The ratio of solid area to gross area is calculated in the appendix at the end of this paper, with the result being

$\varepsilon = 0.45$

The force coefficient for lattice frameworks is given by

$C_{f,\,deck} = 1.6$ (Fig 29.5-2)

The force coefficient for deck posts is given by

$$C_{f,deck\ post} = 2.0\left(\text{for } \frac{h}{D} = \frac{10\,ft}{\frac{8.5\,in}{12\left(\frac{in}{ft}\right)}} = 34.29\right)\text{(Fig 29.5-1)}$$

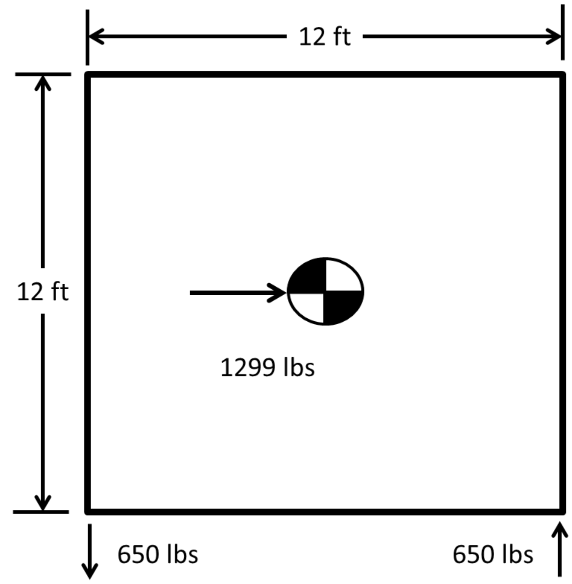

Figure 1. Hold-down Forces Due to Maximum ASD-factored Wind Load

Step 7: Calculate wind loads

Calculate area of deck framework (see Appendix for details)

$A_f = 23.35$ ft^2

Wind load on deck

$F_{deck} = q_z G C_f A_f = 59.93(0.9)(1.6)(23.35) = 2{,}018$ lb (Eqn 29.5-1)

ASD factored deck load

$F_{ASD,d} = 0.6F_{deck} = 0.6(2052\text{lb}) = 1{,}211$ lb

Wind load on deck posts

$F_{post} = q_z G C_f A_f = 59.93(0.9)(2)(2.69) = 291$ lb

ASD factored deck post load

$F_{ASD,p} = 0.6F_{post} = 0.6(291 \text{ lb}) = 175$ lb

Total factored load on deck

$F_D = F_{ad} + ½F_{ap} = 1211 \text{ lb} + (0.5)(175 \text{ lb}) = 1{,}299$ lb

To analyze the force at the reactions, the wind load F_D can be placed at the center of mass which is typically near the center of the deck (Figure 1). The effect of the posts resisting lateral loads was conservatively neglected. The reaction forces were assumed to occur at the hold-down tension devices that were assumed to be attached at the corners of the deck. To gain an understanding of decks with different length-to-width ratios, hold-down forces with different ratios using a deck area of 144 ft^2 are summarized in Table 2.

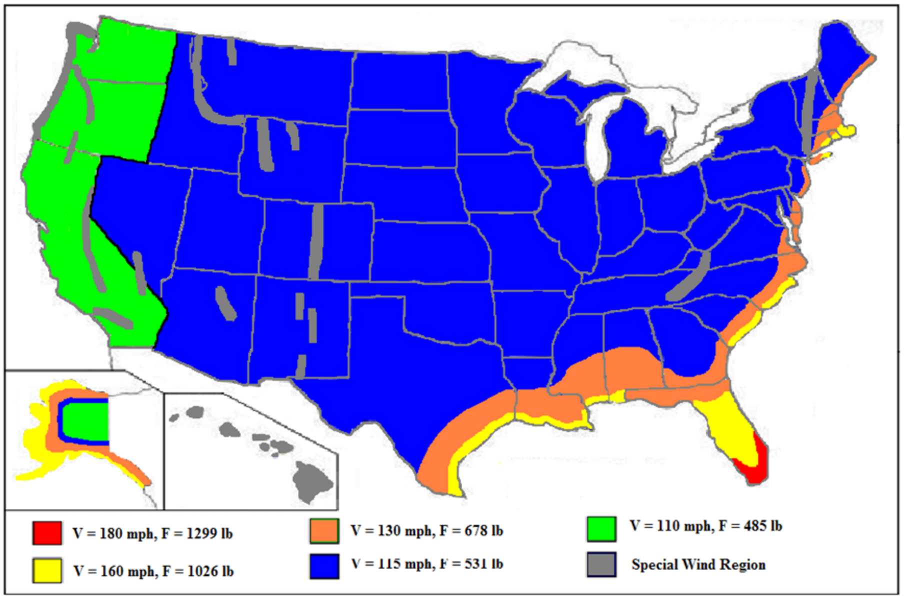

Figure 2. Approximate ASD-factored Wind Loads for Example Deck Using ASCE 7-10

Table 2. Hold-down Forces Due to Maximum ASD Wind Load for Different Deck Ratios

Deck ratio	Hold-down forces (lb)
1.5:1	975
1:1	650
1:1.5	433

Parameter Sensitivity

The example above was performed with the worst-case wind speed. For regions outside of hurricane zones, residential decks will be designed using a much smaller wind speeds. To gain an understanding of wind loads across the US, an investigation of wind loads for different wind speeds, V, using ASCE 7-05 and ASCE 7-10 was performed (Figure 2 and Table 3). The wind forces were calculated using the assumptions from the above example.

There are some differences between the wind loads using ASCE 7-10 compared to ASCE 7-05. At lower wind speeds, the difference between the two versions is minimal but at higher wind speeds, there is up to a 15% difference due to the new ultimate wind speed maps being revised and round-off from the ASD load factor.

It is important to understand wind loads and how these loads are calculated. According to the permitted lateral load connection in the 2009 IRC, Figure R502.2.2.3, at least two hold-down tension devices that have an allowable stress load capacity of not less than 1,500 lbs must be used. The 1,500 lb minimum design capacity was based on judgment. From our wind analyses and deck size with the stated assumptions, and using the directional procedure in ASCE 7-10, hold-down requirements lower than 1,500 lb can be justified if the wind load is the governing load. From our analyses, a maximum ASD-factored wind load of 1,299 lb would be reasonable, resulting in hold-down requirements of approximately 650 lb. This load can be resisted through a variety of hardware solutions.

Most regions in the US use a design wind speed of 115 mph, which results in an ASD-factored wind load of 531 lb for the deck example presented herein. For a deck ratio of one to one, the resulting hold-down forces would be approximately 266 lb.

Table 3: ASD-factored wind loads for different winds speeds assuming Exposure Category

Wind Speed, V (mph)		Velocity Pressure, q_z (psf)		ASD-factored Deck Wind Load, F (lb)	
ASCE7-05	**ASCE7-10**	**ASCE7-05**	**ASCE7-10**	**ASCE7-05**	**ASCE7-10**
85	110	13.36	22.38	483	485
90	115	14.98	24.46	541	531
110	130	22.38	31.26	808	677
130	160	31.26	47.35	1127	1025
150	180	41.62	59.93	1500	1299

Summary and Conclusions

Wind loading can be an important consideration for lateral design of decks. Wind loads were calculated using the directional procedure in ASCE 7. An example was presented to show how to calculate wind loads on residential decks, along with a summary of calculation steps involved. To gain a better understanding of the typical wind loads across the US, the wind loads for different wind speeds were determined using the assumptions from the example presented herein. From the assumptions in the example, the largest ASD wind load was 1,299 lb using ASCE 7-10 methodology and data. The resulting hold-down force for a 12 ft by 12 ft deck would be approximately 650 lb. This load is smaller than the 1,500 lb hold-down requirement in the 2009 IRC, Section 502.2.2.3. From this analysis, the 1,500 lb minimum design capacity is conservatively high for wind lateral loads. An allowable design capacity of 650 lb would be sufficient to resist the wind lateral loads based on the assumptions and calculations given in this paper. Unless you are in a hurricane or special wind region, the hold-down forces will be significantly smaller. Based on the above assumptions, the hold down forces would be approximately 266 lb. By accurately characterizing the lateral loads on decks, design professionals can pursue a range of rational design solutions to resist the loads.

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Notation

A – effective wind area, ft^2

A_f – area of deck normal to the wind direction or projected on a plane normal to the wind direction, ft^2

B – horizontal dimension of building measured normal to wind direction, ft

c – turbulence intensity factor from ASCE 7-10 Table 26.9-1

C_f – force coefficient to be used in determination of wind loads for other structures

F_{ad} – allowable stress design load on deck, psi

F_{ap} – allowable stress design load on post, psi

F_D – load on deck including half of post load, lbs

F_d – load on the deck, lbs

F_p – load on the post, lbs

G – gust-effect factor

g_Q – peak factor for background response

g_v – peck factor for wind response

h – height of deck, ft

I_z – intensity of turbulence

K_d – wind directionality factor

K_z – velocity pressure coefficient evaluated at height $z = h$

K_{zt} – topographic factor

L_z – integral length scale of turbulence, ft

l – integral length scale factor, ft, from ASCE 7-10 Table 26.9-1

Q – background response factor

q_z – velocity pressure evaluated at height z above the ground

V – basic wind speed, mph

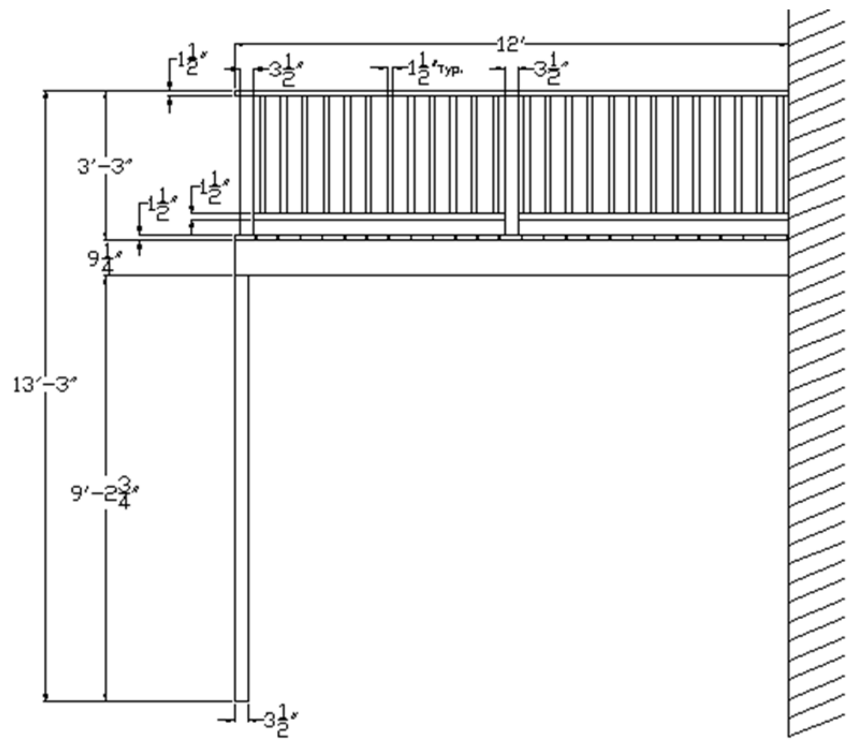
z – height above ground level, ft

z_{min} – exposure constant from ASCE 7-10 Table 26.9-1

ε – ratio of solid area to gross area for deck

ε – integral length scale power law exponent from ASCE 7-10 Table 26.9-1

Appendix – Deck Area Calculations



Deck Area:

$$Joist: \frac{9.25''}{12} x12' = 9.25\,ft^2$$

$$Decking: \frac{1.5''}{12} x12' = 1.5\,ft^2$$

$$Railing: \left(\frac{1.5''}{12}\right)12' + \left(\frac{1.5''}{12}\right)11.29' + 2\left(\frac{3.5''}{12}\right)3' + 25\left(\frac{1.5''}{12}\right)2.54' = 12.6\,ft^2$$

$$Total\ Area = 9.25\,ft^2 + 1.5\,ft^2 + 12.6\,ft^2 = 23.35\,ft^2$$

$$Gross\ Area = \left(\frac{48.25'}{12}\right)12' = 48.25\,ft^2$$

$$\varepsilon = \frac{23.35}{48.25} = 0.48$$

Post Area:

$$Post: \left(\frac{3.5''}{12}\right)9.23' = 2.69\,ft^2$$

Seismic Load Determination for Residential Decks

Garrett H. Lyman, and Donald A. Bender, Ph.D., P.E., J. Daniel Dolan, Ph.D., P.E.

Introduction

Both the 2009 *International Residential Code (IRC)* Section R502.2.1 (ICC 2009b) and the 2009 *International Building Code* (IBC) Section 1604.8.3 (ICC 2009a) require decks to be designed for vertical and lateral loads. This paper is part of a larger project to characterize lateral loads on residential decks caused by seismic, wind, and occupancy. In this paper, we focus on seismic load determination using the standard ASCE/SEI 7-10 *Minimum Design Loads for Building and Other Structures* (ASCE 2010). Specific objectives of this paper are to illustrate a method and example calculation for determining the seismic loads on residential decks and provide a parameter sensitivity study to gain understanding as to the relative magnitudes of seismic loads in various regions in the US.

Method for Determining Equivalent Lateral Seismic Force

One of the analytical procedures permitted in ASCE 7 for calculating seismic loads is the *equivalent lateral force* procedure. This is a simplified procedure that can be used for seismic analysis on residential decks because residential structures are built with light-frame construction. The equivalent lateral force procedure in ASCE 7 determines the seismic base shear in a given direction and the lateral seismic design forces are then distributed to each floor according to an inverted triangular distribution. The seismic base shear force is a function of the seismic response coefficient and the effective seismic weight. The seismic response coefficient is a function of the spectral response acceleration parameter in the short period range, the structural system, and the occupancy importance factor.

Table 1 summarizes the steps to determine seismic loads. Steps 1-10 represent the usual procedure for calculating seismic loads on each story of a light-frame building. Steps 11 and 12 describe how we determined the seismic loads for the deck attached at the second story. We assume the deck is attached to the second story of a house using lag screws and tension hold-down hardware as per IRC Section R502.2.2, resulting in a stiff connection. Hence the seismic load path is from the ground, to the house, to the deck. Referring back to Table 1, in Step 11 we calculate the acceleration on the second story floor by dividing the seismic load from Step 10 by the mass of the second story. Finally, in Step 12, we determine the seismic load on the deck by multiplying the acceleration from Step 11 by the deck mass.

Example

Below is an example following the equivalent lateral force procedure for a residential deck with the following assumptions:

- Deck is located in high-risk seismic zone
- Site class D
- 2-story house with a floor plan area of 1,400 square feet
- Deck height of 10 ft
- Deck dimensions of 12 ft by 12 ft
- Allowable stress design (ASD) format

Determine building weight

Typical Roof/ceiling dead load:

Roof truss top and bottom chord dead loads	15	psf
½ wall weight (partition load)	5	psf
Total	20	psf

Roof weight: $(1400\ ft^2)(20\ lb/ft^2) = 28$ kips

Table 1. Steps to Determine Seismic Load on Residential Decks Using ASCE 7-10 (all tables and equations cited in Table 1 are from ASCE 7-10)
Step 1: Determine importance factor, I_e (Table 1.5-2)
Step 2: Determine the mapped spectral response acceleration parameters, S_S and S_1 (Figure 22-2) or use USGS website
Step 3: Determine site coefficients, F_a and F_v (Table 11.4-1 and 11.4-2)
Step 4: Calculate the MCE_r Spectral Response Acceleration, S_{MS} and S_{M1} (Eq. 11.4-1 and 11.4-2)
Step 5: Calculate the design spectral response acceleration parameters, S_{DS} and S_{D1} (Eq. 11.4-3 and 11.4-4)
Step 6: Determine response modification factor, R (Table 12.2-1)
Step 7: Calculate the approximate fundamental period, T_a (Section 12.8.2.1)
Step 8: Calculate the seismic response coefficient, C_s (section 12.8.1.1)
Step 9: Calculate seismic base shear, V (Eq. 12.8-1)
Step 10: Distribute the lateral seismic forces to the floors and roof
Step 11: Find acceleration on second floor to distribute to the deck
Step 12: Determine the seismic load on the deck

Typical second floor dead load:

Walls (partition load)	10 psf
Plywood, ½" thick	1.6 psf
Gypsum, ½" thick	2.0 psf
Joists	6.0 psf
Lights/misc.	1.0 psf
Total	20.6 psf (Assume 20 psf)

Second floor weight: $(1400\ ft^2)(20\ lb/ft^2) = 28$ kips

Dead load summary:

Roof	28 kips
Second floor	28 kips
Total	56 kips

Residential building: Risk Category II

Step 1: Determine importance factor

$I_e = 1$

Step 2: Determine the mapped spectral response acceleration parameters

$S_S = 1.5$

$S_1 = 1.25$

Note: According to ASCE 7-10 Section 12.8.1.3, for regular structures five stories or less above the base and with a period of 0.5 seconds or less, C_s is permitted to be calculated using a value of 1.5 for S_s.

Step 3: Determine site coefficients

Soil Site Class D

F_a @ $S_S \geq 1.25 = 1$

F_v @ $S_1 \geq 0.5 = 1.5$

Step 4: Calculate the spectral response acceleration

$S_{MS} = F_a S_S = (1)(1.5) = 1.5$

$S_{M1} = F_v S_1 = (1.5)(1.25) = 1.88$

Step 5: Calculate the design spectral response acceleration parameters

$S_{DS} = 2/3 S_{MS} = (2/3)(1.5) = 1$

$S_{D1} = 2/3 S_{M1} = (2/3)(1.88) = 1.25$

Step 6: Determine the response modification factor

Response modification factor: $R = 6.5$

Note: The response modification factor was used for a wood frame house utilizing a bearing wall system of light-framed walls sheathed with wood structural panels rated for shear resistance.

Step 7: Calculate the approximate fundamental period

Building height, h_n = 25 ft

C_t = 0.02

x = 0.75

$T_a = C_t h_n^{x} = (0.02)(25^{.75}) = 0.22$

Note: The values for the approximate period parameters were chosen using the structure type of *all other structural systems*

Step 8: Calculate the seismic response coefficient

$$C_S = \frac{S_{DS}}{\frac{R}{I_e}} = \frac{1}{\frac{6.5}{1}} = 0.15$$

C_s shall not exceed:

$$\frac{S_{D1}}{T_a\left(\frac{R}{I_e}\right)} = \frac{1.25}{0.22\left(\frac{6.5}{1}\right)} = 0.86 \quad \text{for } \mathrm{T} \le \mathrm{T_L}$$

C_s = 0.15 is less than 0.86, therefore OK

Note: T_L was determined to be 12 seconds from ASCE 7 Fig. 22-12 which is greater than T_a

C_s shall not be less than:

$C_s = 0.044 S_{DS} I_e = (0.044)(1)(1) = .04 \ge 0.01$

C_s = 0.15 is greater than 0.044 therefore OK

Since S_1 is greater than 0.6g, C_s shall not be less than:

$$= \frac{0.5 S_1}{\frac{R}{I_e}} = \frac{(0.5)(1.25)}{\frac{6.5}{1}} = 0.1$$

C_s = 0.15 is greater than 0.1, therefore OK

Step 9: Calculate seismic base shear

$V = C_s W = (0.15)(56 \text{ kips}) = 8.62$ kips

Step 10: Distribute the lateral seismic forces to the floors and roof

$F_x = C_{vx} V$

$$C_{VX} = \left(w_x h_x^k\right) \Big/ \left(\sum_{i=1}^{n} w_i h_i^k\right)$$

For structures having a period of 0.5 seconds or less, k = 1.

$$C_{V2} = \frac{(28kips)(10ft)^2}{(28kips)(10ft)^2 + (28kips)(20ft)^2} = 0.333$$

$$C_{VR} = \frac{(28kips)(20ft)^2}{(28kips)(10ft)^2 + (28kips)(20ft)^2} = 0.667$$

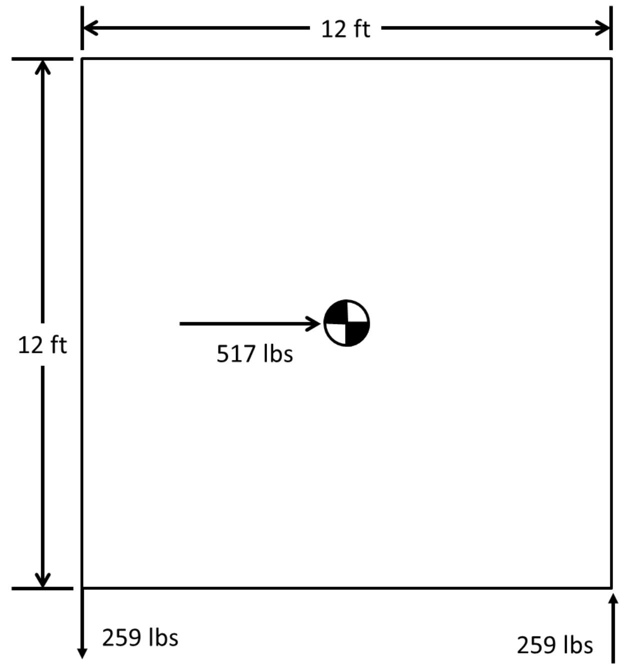


Figure 1. Hold-down Forces Due to Maximum ASD Seismic Load

$F_2 = C_{v2} V = 2.87$ kips

Step 11: Solve for acceleration on second floor to distribute to the deck

$$m_2 = \frac{W_2}{g} = \frac{28\text{kips}}{32.2\frac{\text{ft}}{\text{s}^2}} = 0.87\,\text{kip second}^2/\text{ft}^2$$

$$a_2 = \frac{F_2}{m_2} = \frac{2.87\,\text{kips}}{0.87\,\frac{\text{kips}\left(\text{s}^2\right)}{\text{ft}^2}} = 3.3\text{ft/s}^2$$

Note: Mass and acceleration were determined at the 2nd floor of the building to distribute to the deck.

Step 12: Determine the seismic load on the deck

$A_d = 144 \text{ ft}^2$

$$W_d = 50\left(\frac{\text{lb}}{\text{ft}^2}\right)A_d = \left(50\,\frac{\text{lb}}{\text{s}^2}\right)\left(144\,\text{ft}^2\right) = 7{,}200\,\text{lbs}$$

Note: To be conservative the deck weight included a 10 psf dead load and a 40 psf live load.

$$m_d = \frac{W_d}{g} = \frac{7200\,\text{lbs}}{32.2\,\frac{\text{ft}}{\text{s}^2}} = 224\,\text{slugs}$$

$F_d = m_d a_2 = (224 \text{ slugs})(3.3\text{ft/s}^2) = 738$ lbs

$F_{ASD} = 0.7 F_d = (0.7)(738) = 517$ lbs

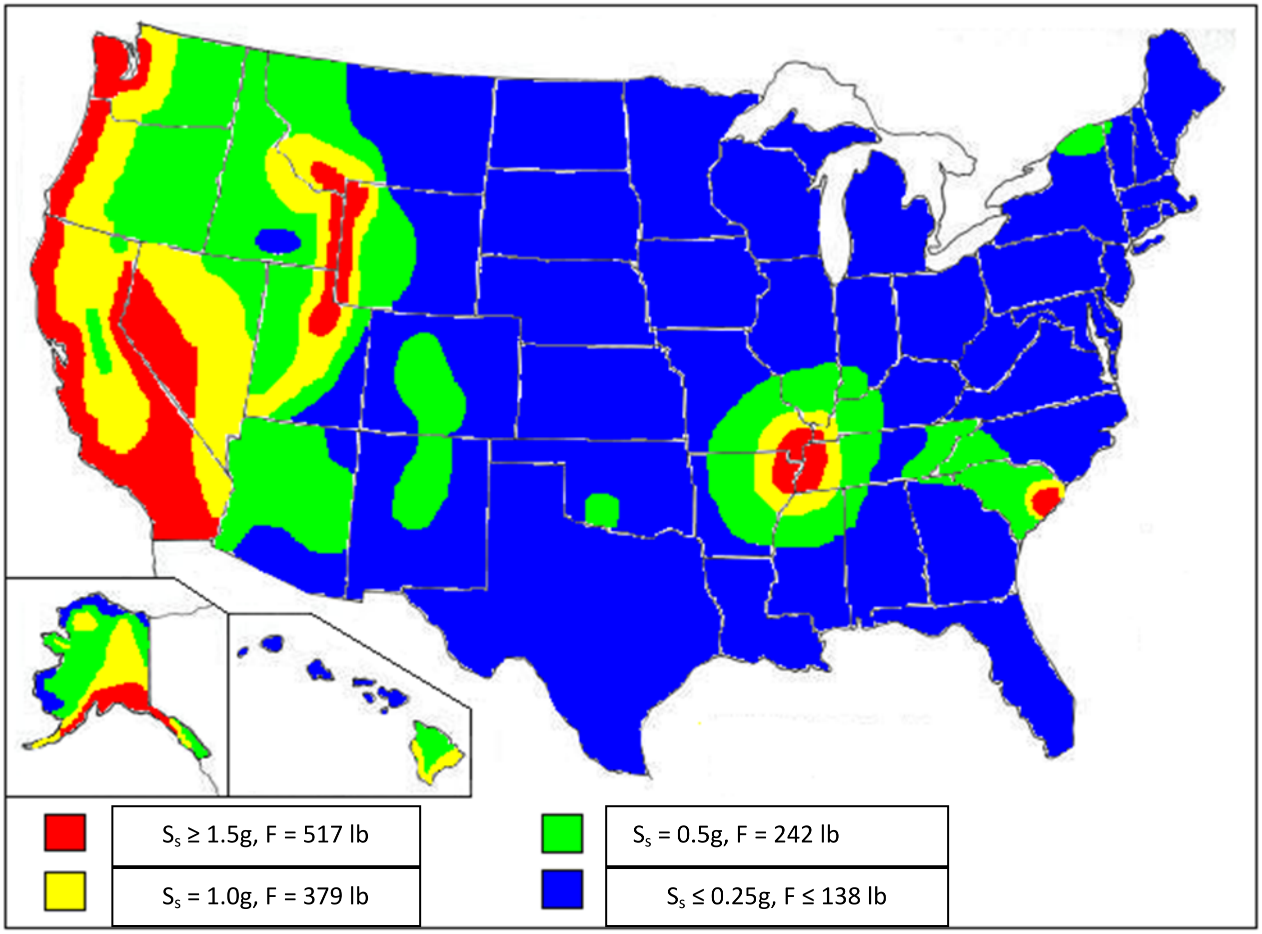


Figure 2. Approximate ASD Seismic Deck Loads for Site Class D

Table 2: Hold-down Forces Due to ASD Seismic Load for Different Deck Ratios

Deck ratio	Hold down forces (lb)
1.5:1	388
1:1	259
1:1.5	173

Therefore the ASD-factored load on the deck is 517 lb. To analyze the hold-down force at the reactions, the seismic load can be placed at the center of mass, which is typically near the center of the deck (Figure 1). The effect of the posts resisting lateral loads will be conservatively neglected. The reaction forces were assumed to occur at the hold-down tension devices that were attached at the corners of the deck. To gain an understanding of decks with different length-to-width ratios, hold-down forces with different deck aspect ratios using a deck area of 144 ft^2 are summarized in Table 2.

Parameter Sensitivity Analysis

To gain an understanding of the typical seismic loads across the United States, an investigation of seismic loads for different spectral response acceleration parameters and site classes was performed (Table 3 and Figure 2). Seismic loads were calculated using the assumptions from the above example. The largest ASD-factored seismic loads occurred for Site Classes D, C, and B with a spectral response acceleration parameter of 1.5g. The maximum load calculated was 517 lb. The maximum value of 1.5g was used for the S_S parameter because ASCE 7-10 Section 12.8.1.3 permits a value of 1.5g for regular structures five stories or less.

According to the permitted lateral load connection in the 2009 IRC, Figure R502.2.2.3, there needs to be a minimum of at least two hold-down tension devices with an allowable stress design capacity of not less than 1,500 lb. Based on our seismic analyses with the stated assumptions, and using the equivalent lateral load provisions in ASCE 7-10, hold-down requirements significantly lower than 1,500 lb can be justified when seismic loads govern. From our analyses, a maximum ASD-factored seismic load of 1,250 lb would be reasonable, resulting in hold-down requirements of approximately 625 lb. This can be achieved through a variety of hardware solutions.

Table 3. ASD Seismic Loads for Different Response Acceleration Parameters and Site Classes

Spectral response acceleration parameters		Seismic Force, F (lb)			
S_s	S_1	Site class E	Site class D	Site class C	Site class B
0.25	0.1	216	138	104	86
0.5	0.2	293	242	207	172
1	0.3	310	379	344	344
1.5	0.6	466	517	517	517
1.5	1.25	466	517	517	517

Summary and Conclusions

Seismic loads were calculated using the equivalent lateral force procedure in ASCE 7-10. An example was presented along with a summary of the calculation steps involved. Seismic loads for different seismic zones were determined using the assumptions from the example presented herein. The largest ASD-factored seismic load calculated was 517 lb. After analysis of this load on a 12 ft by 12 ft deck, the reaction hold-down force was 259 lb. This load is smaller than the permitted hold-down tension devices that require an allowable stress design capacity of 1,500 lb each in the 2009 IRC, Section 502.2.2.3. An allowable design capacity of 625 lb would be sufficient to resist the seismic lateral loads based on the assumptions and calculations given in this paper. By accurately characterizing the lateral loads on decks, design professionals can pursue a range of rational, economical solutions to resist lateral loads.

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Notation:

a_2 – second floor acceleration

A_d – area of the deck

C_s – seismic response coefficient

C_t – building period coefficient

C_{vx} – vertical distribution factor

C_{v2} – 2nd floor vertical distribution factor

C_{vr} – roof vertical distribution factor

g – acceleration due to gravity (32.2 ft/s^2)

I_e – the seismic importance factor

F_a – short-period site coefficient

F_{ASD} – allowable stress design load on the deck

F_d – load on the deck

F_v – long-period site coefficient

F_x – portion of the seismic base shear, V, induced at Level *i*, respectively

F_2 – portion of seismic base shear on second floor

h_i,h_x – the height above the base to level *i*

h_n – structure height

k – distribution exponent

m_2 – mass on the second floor of building

m_d – mass of the deck

R – response modification coefficient

S_s – mapped MCE_R, 5 percent damped, spectral response acceleration parameter at short periods

S_1 – mapped MCE_R, 5 percent damped, spectral response acceleration parameter at period 1s

S_{DS} – design, 5 percent damped, spectral response acceleration parameter at short periods

S_{D1} – design, 5 percent damped, spectral response acceleration parameter at a period of 1s

S_{MS} – the MCE_R, 5 percent damped, spectral response acceleration parameter at short periods adjusted for site class effects

S_{M1} – the MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1s adjusted for site class effects

T – fundamental period of the building

T_a – approximate fundamental period of the building

T_L – long-period transition period

V – total design lateral force or shear at the base

W – effective seismic weight of the building

W_d – weight of the deck

w_i,w_x – portion of W that is located at or assigned to Level *I*, respectively

w_2 – weight of second floor of building

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Deck and Porch Lateral Loading by Occupants

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Introduction

When engineers consider lateral loading on structures, typically the loads considered are from wind and seismic events. One source of lateral load that is not commonly considered, and has no calculation methodology in ASCE/SEI 7-10 *Minimum Design Loads for Building and Other Structures* (ASCE 2010), is occupant lateral movement. Preliminary research at Washington State University revealed that forces generated by occupants are significant, and in many cases greater than wind or seismic forces. The objective of this study was to quantify lateral loads caused by dynamic actions from the occupants. Two deck configurations and two dynamic load cases were investigated.

Deck Configuration 1: Deck boards oriented parallel to the ledger

Deck Configuration 2: Deck boards oriented 45 degrees to the ledger

Load Case 1: Cyclic

Load Case 2: Impulse

It was expected that the two deck board orientations would result in dramatically different stiffnesses in the lateral loading plane since according to the ANSI/AF&PA *Special Design Provisions for Wind and Seismic* (AWC 2008), diaphragms and shear walls sheathed with diagonally oriented boards compared to horizontal boards results in a four-fold increase in stiffness. The two dynamic load cases were chosen to represent the types of occupant behavior that might result in the greatest lateral loads. The full details of the research reported herein can be found in Parsons et al. (2013b).

Background

The 2009 *International Building Code* (IBC) and the ASCE/SEI 7-10 *Minimum Design Loads for Building and Other Structures* are silent on the subject of lateral loads from occupants, with one exception. Table 4-1 in ASCE 7-10 gives gravity loads for reviewing stands, grandstands and bleachers, along with Footnote k which stipulates *lateral loads* of "… 24 lbs per linear ft of seat applied in the direction parallel to each row seats…". Footnote k was based on empirical research by Homan et al. (1932) where the lateral forces caused by the movement of a group of people on a simulated grandstand were studied. The lateral load provision in Footnote k is a convenient benchmark for comparing the deck loads reported in this paper. For example, assuming each row of grandstand seats is approximately 2 ft apart, this lateral load provision would be equivalent to 12 psf of plan area.

Materials

Both deck floor configurations were 12 ft by 12 ft using similar materials, with the orientation of deck boards being the only factor that differed. Decks were built according to *Design for Code Acceptance 6 (DCA 6)* (AF&PA, 2010), which is based on the 2009 *International Residential Code (IRC)*. The deck ledger was constructed of 2x12 lumber; joists were 2x10 spaced 16 inches on center; and deck boards were 2x6 installed with no gapping. Deck boards were not gapped due to their high moisture content at time of installation. All lumber was incised and pressure preservative treated (PPT), with a grade of No. 2 and Better, and species grouping of Hem-fir. The PPT formulation was Alkaline Copper Quaternary Type D (ACQ-D) with a retention level of 0.40 pcf.

Table 1. Forces Generated by Occupants From Impulse Loading.

Occupant Load Level, (psf)	Deck Board Orientation to Ledger	Total Force, (lbs)	Uniform Lateral Load, (psf)
Impulse loading perpendicular to ledger			
10	Parallel	384	2.7
10	45 Degrees	443	3.1
Impulse loading parallel to ledger			
10	Parallel	428	3.0
10	45 Degrees (East)	1,297	9.0
10	45 Degrees (West)	1,351	9.4

The hangers used to connect the deck joists to the ledger were Simpson Strong-Tie Model No. LU210, which use 20-gauge steel and 16 fasteners; 10 into the header and 6 into the joist. This hanger was selected because the fastener pattern (all fasteners installed perpendicular to the member faces) performed well when joists were loaded in tension (pulling away from the hanger). The manufacturer's joist hanger that was recommended for corrosive environments had a toe-nail type fastening pattern for attaching to the joists, which did not perform well in preliminary tests when the joists were loaded in withdrawal from the hanger. Of course, before any connection hardware is used in an actual deck, the appropriate corrosion protection must be satisfied.

The joist hanger manufacturer permits their joist hangers to be installed with either nails or screws as specified in their technical literature. Screws were used with the joist hangers to meet the provisions of the model building codes. IRC-2009 Section R507.1 and IBC-2009 1604.8.3 both state that the deck attachment to an exterior wall *shall not be accomplished by nails subject to withdrawal*. These provisions have been widely interpreted as applying to the deck ledger attachment; however, these provisions also should apply to deck joist hanger attachment to the deck ledger to complete the lateral load path from the deck to house. The joist hanger screws were #9 (0.131 inch diameter, 1-1/2 inch long) Simpson Strong-Tie Structural-Connector Screws (Model No. SD9112). These screws have a Class 55 2006 IRC compliant mechanical galvanized coating to mitigate corrosion due to the preservative chemicals in the lumber and wet use conditions. The deck boards were attached to the top of each joist with two 3-inch #8 wood screws rated for outdoor use.

Test Methods

Standard test methods are not available for occupant-induced lateral loading, so two testing protocols were developed to represent worst-case conditions. Each person participating in the study was weighed, allowing us to evaluate occupant densities of 10, 20, 30, and 40 psf. A conservative assumption was made that other than the attachment at the ledger, the deck substructure would provide negligible lateral resistance; therefore, the deck was supported on rollers as shown in Figures 1 and 2. In reality, many decks have some degree of lateral support provided by stairs, braces or other configurations that provide resistance to lateral movement. Lateral stiffness of decks differs substantially when loaded parallel versus perpendicular to the ledger; hence, loadings in both directions were conducted for all cases.

The first load case was an *impulse*. For this type of loading, the occupants were instructed to start at one end of the deck and run and jump, in unison, towards the opposite side of the deck. Impulse loading was conducted with an occupant density of 10 psf to allow occupants ample room to run and jump. The second load case was *cyclic,* in which the occupants were instructed to sway, in unison, following visual and audible cues, back and forth at an approximate frequency of 1 Hz.

All impulse and cyclic tests were performed with motion parallel and perpendicular to the deck ledger. Forces were recorded at the two corners where the deck was anchored to the laboratory floor with steel brackets (simulating the building). In an actual building, the load path would differ from this test set-up since deck ledger boards are typically connected to the house along the entire length. The rationale for attaching the deck at two discrete points was to obtain a conservative (high) load

Table 2. Forces Generated by Occupants from Cyclic Loading.

Occupant Load Level, (psf)	Deck Board Orientation to Ledger	Total Force, (lbs)	Uniform Lateral Load, (psf)
Cyclic loading perpendicular to ledger (stiffest direction)			
10	Parallel	224	1.6
10	45 Degrees	226	1.6
20	Parallel	398	2.8
20	45 Degrees	543	3.8
30	Parallel	411	2.9
30	45 Degrees	482	3.3
40	Parallel	651	4.5
40	45 Degrees	502	3.5
Cyclic loading parallel to ledger			
10	Parallel	320	2.2
10	45 Degrees	567	3.9
20	Parallel	983	6.8
20	45 Degrees	862	6.0
30	Parallel	1,431	9.9
30	45 Degrees	995	6.9
40	Parallel	1,747	12.1
40	45 Degrees	1,020	7.1

estimate by attracting all load to the two attachment points. Load path from the deck into the house floor diaphragm was investigated in a separate study reported in a companion paper (Parsons et al. 2013a).

Results & Discussion

Results of this study were reported as equivalent uniform lateral surface tractions in psf generated by occupant actions. These values were determined by dividing the total force generated by the surface area of the deck floor. Loads in this form can easily be applied to decks of any size for design purposes. For the perpendicular to ledger load cases, the total force was taken as the sum of the two load cells. For the parallel to ledger load cases, the total force was taken as two times the maximum load cell value by applying basic equilibrium principles.

Impulse Loading

Forces generated on both deck configurations are shown in Table 1 for the perpendicular and parallel to ledger load cases. All tests were recorded with high-definition video and retained by the authors. A sample still shot from the video can be seen in Figure 1 for the impulse loading.

Perpendicular to ledger: Impulse loads were similar for both decking configurations since deck stiffness was primarily controlled by axial stiffness of the joists rather than the decking orientation. The stiffness of the deck resulted in many short duration pulses as each person landed, but was not flexible enough to allow the pulses to accumulate into one large force.

Parallel to ledger: When impulse loading was directed parallel to the deck ledger, as shown in Figure 1, decking orientation controlled the stiffness of the system. Table 1 shows that the less stiff deck (with decking oriented parallel to the ledger) experienced lower loads as the pulse duration was relatively long at impact, and the occupants velocities were reduced by the deck movement as the occupants pushed off to accelerate. The greatest loads were observed for diagonal decking. Apparently this scenario “hit the sweet spot” of a deck with just enough flexibility to allow the individual impacts to act additively in a long enough time interval. In any case, the maximum traction load of 9.4 psf was less than the value of 12.1 psf for cyclic loading.

Figure 1. Impulse Loading Caused by Occupants Leaping/Stopping in Unison

Figure 2. Cyclic Loading Caused by Occupants Swaying Side to Side in Unison

Cyclic Loading

Figure 2 shows a sample still shot from the video for the cyclic side-sway motion.

The highest lateral load observed in all tests was 12.1 psf as shown in Table 2. In this case, deck boards were oriented parallel to the deck ledger, resulting in a very flexible deck that swayed back and forth approximately 7 inches each way at a frequency of approximately 1 Hz. These large displacements caused significant inertial forces from the mass of the deck and also allowed the occupants to "feel" the deck movement, making it easier for them to synchronize their movements. As displacements of the deck reached maximum values of approximately 7 inches, the occupants started pivoting their hips (like downhill skiers) with the deck while leaving their upper body nearly motionless. At this point, it could be argued that the majority of the force generated is coming from deck inertial forces rather than from the occupants. This would imply that if lateral sway/acceleration of a deck is adequately restrained, these inertial forces could be reduced or eliminated. For example, when the cyclic motion was perpendicular to the deck ledger (the stiffest orientation), the maximum traction load was 4.5 psf. In summary, it could be argued for design that 12 psf would provide a reasonable upper estimate of lateral loads from occupants for flexible decks.

Conclusions

When deck boards were oriented parallel to the ledger and occupant loading was applied parallel to the ledger, large side-to-side displacements were observed when a cyclic action was performed by the occupants. These large displacements produced significant inertial forces with a maximum equivalent uniform lateral surface traction of 12.1 psf. When cyclic actions were perpendicular to the ledger (i.e. the stiffest lateral direction), it was difficult for the occupants to synchronize their movements and the resulting maximum uniform surface traction was 4.5 psf. The maximum recorded impulse load resulted in a uniform lateral surface traction of 9.4 psf as compared to the maximum surface traction of 12.1 psf for cyclic loading.

A design lateral load of 12 psf of plan area is recommended, which conservatively includes inertial forces from a flexible deck. The 12 psf observed in the laboratory is similar to the lateral load specified in Table 4-1, Footnote k (ASCE/SEI 7-2010) for reviewing stands, grandstands and bleachers, which call for 24 lb/linear ft of seat (assuming seats are 2 ft apart, the resulting load would also be 12 psf). One surprising outcome of this research is that measured lateral loads from occupancy exceeded the calculated worst-case lateral loads from wind or seismic hazards (Garrett and Bender, 2013; Garrett et al., 2013). Furthermore, extreme occupant loading can occur *anywhere* in the US, while extreme wind and seismic events are limited to smaller geographic regions.

The testing protocol and conclusions reported herein are based on the assumption that the proposed deck or porch sub-structure has no auxiliary lateral support to resist occupant loading. The design professional is encouraged to include lateral support structures to resist all or part of the lateral loads produced by occupant loads (as well as other design loads such as wind or seismic). It should be noted that the weak link in the load path might be the fasteners used in the joist hangers. Our test assemblies were fabricated with screws to prevent premature withdrawal of nails in the joist hangers. The first step in any lateral load analysis, when required, should be to address the lateral design capacity of the joist connections (hangers) as nails would likely not be adequate in resisting lateral loads produced by occupants.

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Lateral Load Path and Capacity of Exterior Decks

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Introduction

The safety of exterior elevated decks and porches is an important national issue due to numerous documented collapses and resulting injuries and, in some cases, deaths (Shutt 2011; Legacy Services 2010). The 2009 *International Residential Code (IRC)* Section R502.2.2 (ICC 2009b) requires decks to be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Designing decks for vertical (gravity) loads is well understood, but less is known about lateral loads and designing decks to resist these lateral loads. This issue of *Wood Design Focus* illustrates how to calculate wind and seismic lateral loads on decks, and presents original research on lateral loads from occupants. The next obvious question is to quantify how the lateral loads transfer from a deck floor to the house structure.

A prescriptive lateral hold-down concept was introduced into the 2009 *International Residential Code* (IRC Figure 502.2.2.3) as a means of resisting chord forces of a deck diaphragm subjected to lateral loading. This paper aims to define the load paths of a commonly constructed exterior deck and evaluate the effectiveness of the current prescriptive detail for resisting lateral loads. A common deck construction that followed IRC provisions was investigated with and without hold-down tension devices. The full details of the research reported herein can be found in Parsons et al. (2013).

Materials and Deck/Diaphragm Construction

Two identical 12 ft by 12 ft decks were constructed using similar materials; one with a tension hold-down at two corners, and one without. The decks were built in accordance with *Design for Code Acceptance 6 (DCA 6)* which is based on the 2009 *International Residential Code (IRC)*. The deck ledger was a 12 ft 2x10; joists were 2x10 spaced 16 in on center; and deck boards were wood-plastic composite (nominal 1x6) Trex Accents installed with ¼ in gaps. All lumber used for the deck joists and ledger was incised and pressure preservative treated (PPT), No. 2 and Better Hem-Fir. The preservative treatment was alkaline copper quaternary Type D (ACQ-D) with a retention level of 0.40 pcf. Moisture content and specific gravity was measured for all framing lumber and are reported in Parsons (2012).

The simulated house diaphragm assembly was constructed to be approximately 16 ft long by 3.8 ft deep. The diaphragm assembly consisted of a double top plate connected to the laboratory reaction floor (simulating the resistance of an exterior wall), floor joists, rim boards, and floor sheathing. The joists were 2x10's spaced 16 in oc; double top plates were two 2x6's with splices constructed no closer than 4 ft; rim boards were continuous 2x10's; and the bottom plate was constructed of 2x6's. All lumber used for the house diaphragm was untreated, No. 2 and Better Douglas Fir-Larch. Elevation and plan views of the test set-up are given in Figure 1.

All nailing used in the construction of the simulated house diaphragm followed IRC Table R602.3(1) and the *Wood Frame Construction Manual* (AF&PA 2001). OSB Rated Sheathing used for the house floor diaphragm was 23/32-in nominal thickness with a 24 inches on center floor span rating and Exposure 1 adhesives. The sheathing was glued and nailed to the joists using construction adhesive designed for subfloor and deck applications. Nails, 2.5 inches by 0.131 inches, were used per IRC Table R602.3(1) to fasten the

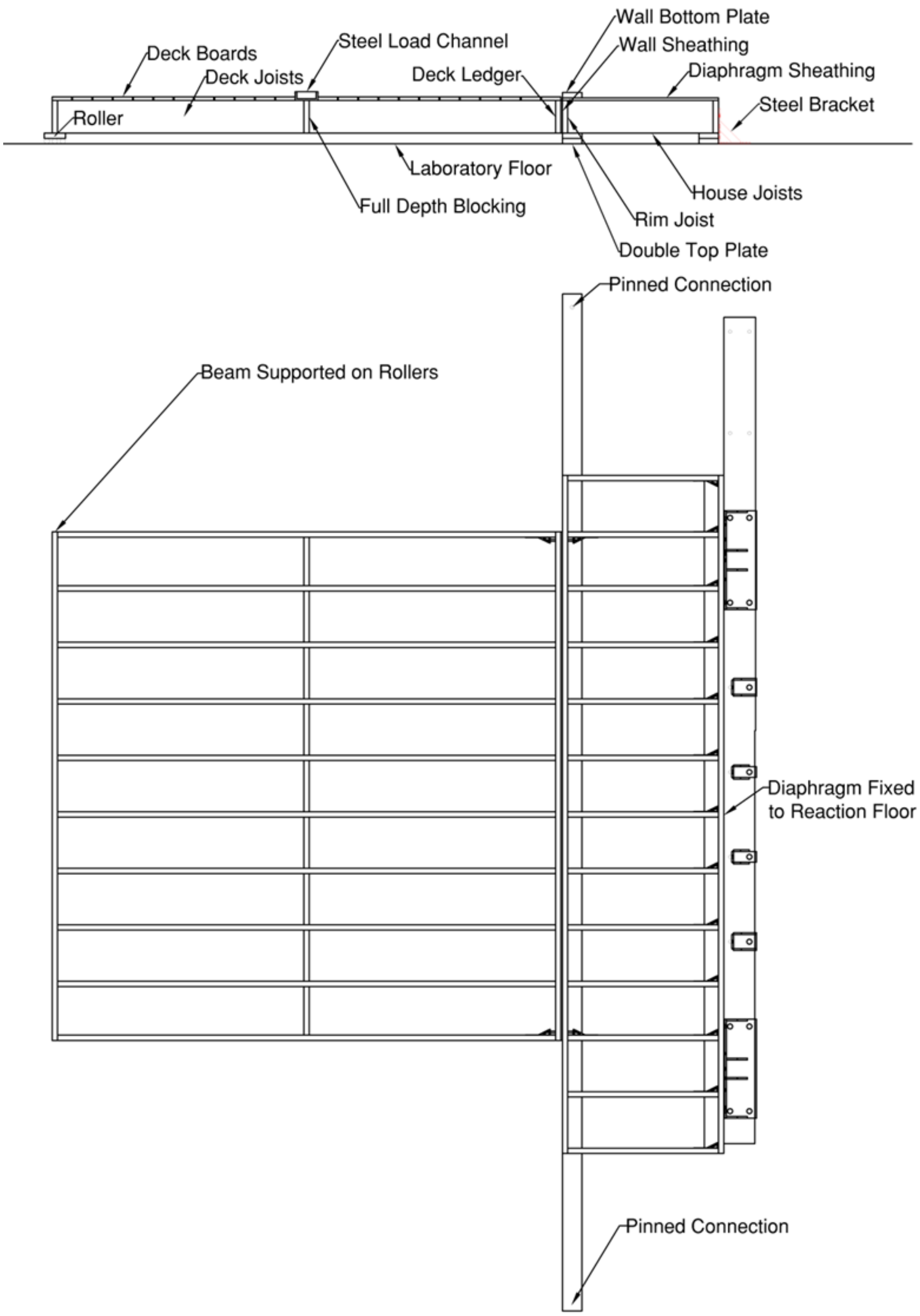

Figure 1. Elevation and Plan Views of Test Setup Construction

7/16 inch thickness category, and Exposure 1 adhesives. Simulated wall sheathing was included since it acts as a spacer between the house rim board and the deck ledger and could influence the lag screw connection performance. Lag screws were selected to fully penetrate through the house rim board plus an additional 0.5 inches, therefore transferring the load through the wall sheathing and into the rim board.

Two types of joist hangers were used for deck construction - Simpson Strong-Tie (SST) Model No. LU210 and Model No. LUC210Z. LU210 hangers were 20-gauge steel and used a total of 16 fasteners; 10 into the ledger and six into the joist (three on each side, driven perpendicular to the joist). LU210 hangers had a standard G90 zinc coating, which SST classifies as a low level of corrosion resistance. This hanger was selected because the fastener pattern (all fasteners installed perpendicular to the member faces) performed well when joists were loaded in tension (pulling away from the hanger). The LUC210Z hangers were 18-gauge steel and used a total of 16 fasteners; 10 into the header and six into the joist. The LUC210Z had a "ZMAX" coating, which is classified as a medium level of corrosion resistance. Based on the environment, the design professional should take care to specify appropriate corrosion protection for all hardware used in a deck.

sheathing to the joists. Floor sheathing nailing was installed immediately after the adhesive was applied at 6 inches on center along sheathing perimeter and 12 inches on center along intermediate supports. When hold-downs were used, nails were spaced 6 inches on center on the diaphragm joist to which the hold-down was attached.

Rated Sheathing used between the diaphragm rim board and deck ledger board had a 24/16 span rating,

Lag screws with 0.5-inch diameter full body and a length of 7 inches (to accommodate the load cell) and a root diameter of approximately 0.370 in were used. Lag screws were installed 15 inches on center in a staggered pattern as specified in IRC Table R502.2.2.1. Per

Figure 2. Load Application Setup Showing Framing and Blocking

the DCA 6, each lag screw was thoroughly tightened, without over-tightening to prevent wood crushing, which resulted in a tensile force of approximately 500 lb in each lag screw. Due to stress relaxation, this force was slightly less at the initiation of tests.

While the joist hanger manufacturer permits their hangers to be installed with either nails or screws as specified in their technical literature, screws were used in this study. IRC-2009 Section R507.1 (ICC 2009b) and IBC-2009 1604.8.3 (ICC 2009a) both state that the deck attachment to an exterior wall shall not be accomplished by *nails subject to withdrawal*. These provisions have been widely interpreted as applying to the deck ledger attachment; however, they should also apply to deck joist hanger attachment to the deck ledger needed to complete the lateral load path from the deck to house. Joist hanger screws were #9 (0.131 inch diameter, 1.5 inches long) SST Structural-Connector Screws (Model No. SD9112) and #10 (0.161 inch diameter, 1.5 inches long) SST Structural-Connector Screws (Model No. SD10112). These screws have a Class 55 2006 IRC compliant mechanical galvanized coating which is required to resist corrosion. The deck boards were attached to each deck joist with two #9 SST Composi-Lok™ Composite-Decking Screws (Model No. DCLG212). Each deck board screw was installed approximately 1 inch from the deck board edge, and each deck board was cut to length (no splices).

The hold-down connectors used on the second deck configuration were SST DTT2Z with a "ZMAX" protective coating. The hold-down was 14-gauge steel and a 0.5 inch diameter threaded rod was used to connect the hold-downs from the deck to the house. The screws used with the hold-down were (0.25 inches by 1.5 inches) Simpson Strong-Tie Strong-Drive screws (Model No. SDS25112). These screws had a double-barrier coating, which SST rates as equivalent corrosion resistance to hot-dip galvanized.

Test Methods

Occupant loads were idealized as a resultant line load acting through the centroid of the deck surface, simulating the resultant force that would be present from a uniformly distributed lateral load applied to the deck boards. The deck board loading was accomplished by installing full-depth blocking along the centerline and attaching a steel channel to the deck surface with lag screws in to the joists. The load was then applied to this channel. The steel channel acted as a drag strut to evenly distribute the force along the length of the deck. Since large displacements were anticipated, force was applied with a come-along as shown in Figure 2.

A conservative assumption was made that the deck substructure would provide minimal lateral resistance; therefore, the deck was supported on rollers along the outer beam. The simulated house diaphragm was securely anchored to the laboratory reaction floor.

A 10 kip load cell was installed in-line with the come-along to record the force applied to the deck. Load cells made out of steel sleeves and strain gages were used to record forces in lag screws connecting the deck ledger to the diaphragm rim board and hold-downs. Parsons (2012) gives a detailed description of these load cells and other experimental details. Seven string potentiometers were used to measure various deck displacements.

Results and Discussion

Lateral Force Resisting Mechanism

A large portion of lateral resistance was provided by moment couples formed by the screws in the deck board-to-deck joist connection, as shown in Figure 3. A test was conducted without deck boards installed to determine the initial stiffness of the bare frame (Figure 2), which resulted in a value of 98.8 lb/in. This low amount of stiffness was primarily provided by the rotational stiffness of the joist hangers and the supporting rollers. The initial stiffness determined after the deck boards were installed was approximately 2,600 lb/in for both decks. Therefore, 96% of the initial lateral stiffness was provided by the deck board-to-joist connections. The magnitude of each resisting couple is a function of the distance between the two screws and capacity is limited by the screw strength and joist strength in tension perpendicular to grain.

Observed Damage

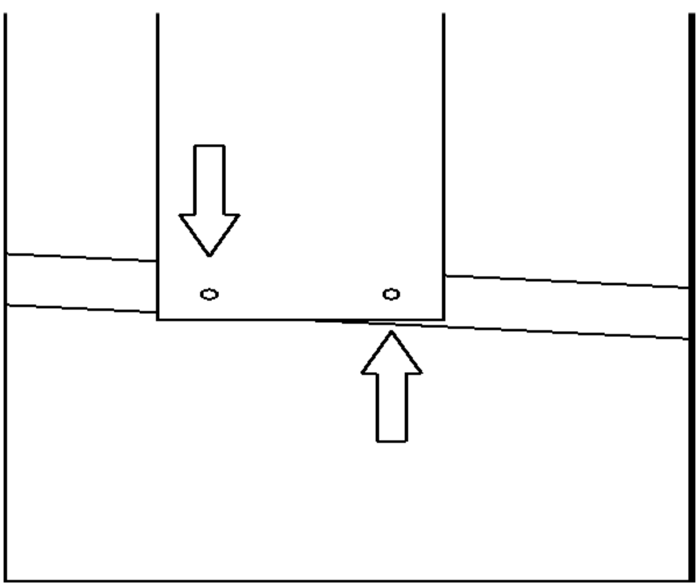

Figure 3. Deck Board to Joist Connection and Resisting Couple Providing Lateral Resistance

In both tests, splitting of the top edges of the deck joists was the main source of damage, and was caused by the couple from the deck screws that induced stresses perpendicular to the grain. Splitting propagated along the longitudinal axis of the wood. Each deck joist completely split, to the depth of screw penetration, from the load drag strut to the ledger board. Significant yielding and fracture of deck board screws was also observed in this region. Minimal joist splitting and screw yielding was seen in the region from the load drag strut to the outer deck beam. In both tests, no damage was observed in the deck ledger to house rim board connection. A maximum separation of 0.1 inches when hold-downs were used and 0.15 inches when hold-downs were not used was recorded between the deck ledger and diaphragm rim board at the tension chord of the deck. No damage was observed in the simulated house diaphragm.

In the test that used hold-down tension connectors, deck joists fractured in weak axis bending due to the hold-down installed on the compression chord producing larger rotational joist stiffness at the ledger connection than the joist hangers provided on the other joists. This caused load from the other deck joists to be attracted to the end joist, resulting in fracture. Once the end joist fractured, the remaining joists fractured due to progressive failure.

Load-Displacement Curves

For the test with no hold-down, the load displacement curve at the load drag strut, shown in Figure 4, can be divided into three segments. The first segment was a softening curve that is seen in tests of many mechanically connected structural assemblies as slip occurs and damage initiates. At a displacement of approximately 3.5 inches, significant joist splitting has occurred and most of the diaphragm stiffness from the deck board attachment is lost. The second segment of the load-displacement curve from 3.5 to 17 inches is approximately linear, with stiffness nearly equal to that of the bare frame (shown at bottom of Figure 4). After 17 inches, the third segment shows an unexpected large increase in stiffness.

For the test with hold-downs, slightly higher stiffness and load at 4 inch displacement were observed due to the hold-downs resisting rotation of the deck joists. Similar to the first test, the second segment from 4 to 15 inches reflects the frame stiffness with deck boards contributing little. At a displacement of approximately 16 inches, the outer deck joists ruptured in weak-axis bending, followed by a sharp drop-off in load. In the third

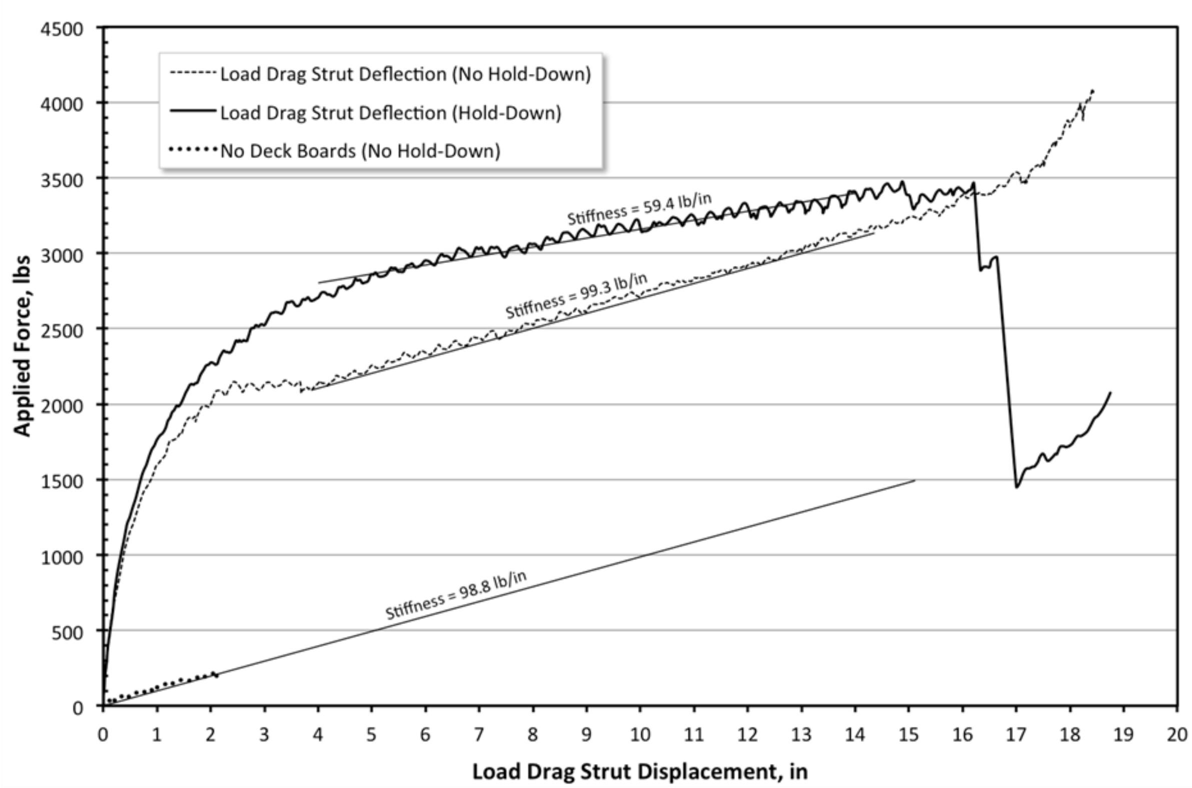

Figure 4. Load-Displacement Curves for Deck With and Without Hold-downs

segment, a large increase in stiffness was once again seen at approximately a displacement of 17 inches even after deck joists had severely fractured.

When displacements reached approximately 17 inches at the load drag strut, a large unexpected increase in stiffness was seen in both decks (Figure 4). This large change in stiffness is not fully understood, but could be due to two phenomena. The increase in stiffness is most likely caused by large lateral deflections and the resulting rotation of the deck joists. This caused increased portions of the lateral load to be resisted by axial tension of the joists and hangers (recall the joist hangers were attached with screws, thereby provided significant withdrawal resistance). A second explanation could be a function of deck board spacing. The stiffness increase could occur at the point where deck boards began to bear against each other (i.e., the gap between deck boards has closed), causing a large portion of the force to be resisted by compression between deck boards. Determining the exact reason for this large increase in stiffness is probably not practically significant since it occurred at extreme levels of displacement that would most likely cause column instability under gravity loads. Also, at this point significant damage was present in the joists, which would compromise the safety of the deck. From a practical standpoint, deck failure could be defined as the point when the diaphragm stiffness was lost by joist splitting at a displacement of approximately 4 inches.

Lag Screw Forces

The lag screws to one side of the ledger board centerline were in tension and the other side compression, as expected. The two outermost lag screws in tension resisted most of the chord force and the sum of the forces in all the lag screws located in the tension region of the deck agree well with the calculated overturning tension force (Figure 6). Furthermore, even though the two outermost lag screws carried most of the force, these lag screws did not show any visible signs of withdrawal at a maximum load of approximately 7,000 lbs (Figure 5)

Hold-Down Behavior and Geometric Effects

If the deck behaved as a rigid body, the tension chord forces can be calculated using simple statics as given in Equation 4.3-7 of the 2008 *Special Design Provisions for Wind and Seismic* (AF&PA, 2008), and are shown in Figure 7. However, due to the flexibility of the deck, the measured forces in the hold-down connectors were dramatically different than expected. The hold-down expected to resist overturning tension forces actually diminished to zero as the deck deformed. The hold-down

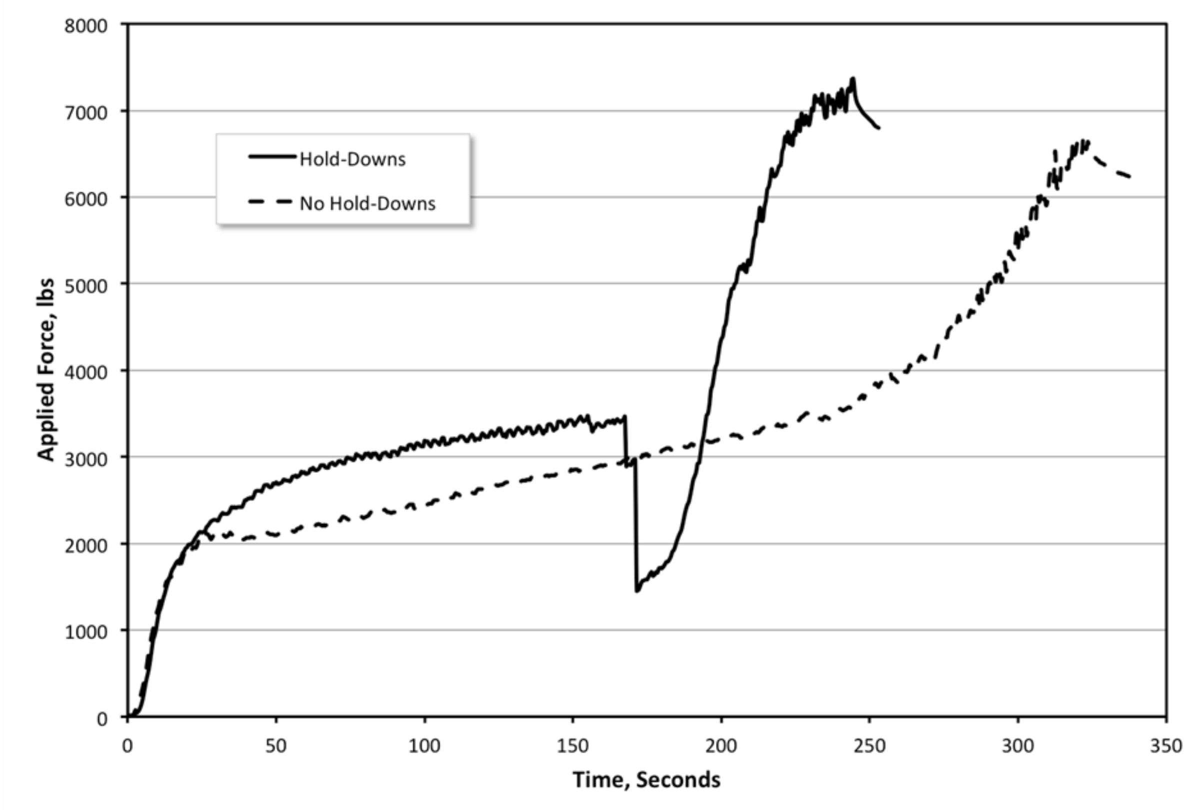

Figure 5. Load-Time Curves for Deck With and Without Hold-downs

installed on the compression chord, which was expected to resist no tension forces, actually had significant **tension** force due to a geometric prying effect caused by joist rotation.

Significant rotations of the joists occurred due to large displacements. Figure 8 illustrates how the tension chord rotation caused a gradual loss of hold-down pre-tension force until there was zero tension force in the hold-down. This outcome demonstrated that the geometric effect that was reducing the force in the hold-down was larger than any tension force in the joist from overturning moments. At this point, the joist hanger was resisting the entire tension force in the joist, bypassing the hold-down altogether. It can also be seen that the hold-down on the compression chord is moving away from the ledger as deck joist rotations increased. Eventually, the result was a significant tension force that caused yielding of the hold-down. These same effects are not seen in typical light-frame shear walls because the chord framing members experience much smaller rotations.

Due to this geometric effect, the hold-downs in their installed locations, behaved in a way that was completely counterintuitive. The hold-downs might be more effective if the deck stiffness was increased, by installing the decking diagonally. According to the 2008 *Special Design Provisions for Wind and Seismic* (AF&PA, 2008), shear walls and diaphragms sheathed with diagonally oriented boards compared to horizontal results in four-fold increase in stiffness. Also, if the joist connections to the ledger had low withdrawal capacity, such as when nails are used in the hangers, or toe-nails, then the tension hold-down connection would be expected to function as intended.

Design Implications

Joist hangers -- Joist hangers are typically rated for gravity (vertical) loads. When a deck is loaded laterally, the outermost joists are loaded in tension. Joist hangers are not load-rated in tension (i.e. joist withdrawal from the hanger). Preliminary experiments revealed that joist hangers that utilized a toe-nailed fastener orientation did not perform well when the toe-nailed connection was subject to tension loads. As such, hangers used in this project had fasteners installed perpendicular to the joist faces.

Joist hanger manufacturers generally permit joist hangers to be installed with either nails or screws as speci-

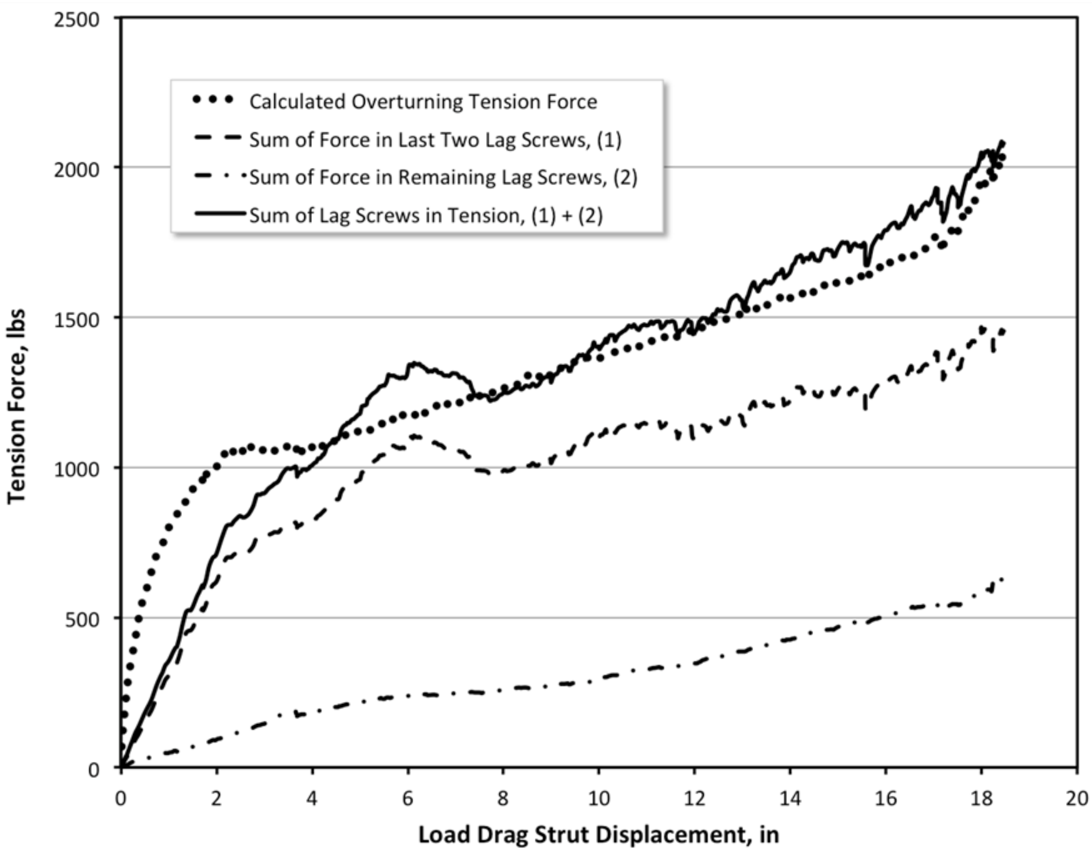

Figure 6. Lag Screw Forces on Deck Without Hold-downs. Overturning Tension Force Calculated Assuming End Joist Resists Full Overturning Moment

fied in appropriate technical literature. In this project, screws were used with the joist hangers to meet the provisions of IRC-2009 Section R507.1 and IBC-2009 1604.8.3, which both state that the deck attachment to an exterior wall shall not be accomplished by nails loaded in withdrawal. These provisions have been widely interpreted as applying to the deck ledger attachment; however, they should equally apply to deck joist hanger attachment to the deck ledger needed to complete the lateral load path from the deck to house.

Parsons (2012) performed calculations to determine the allowable withdrawal and lateral capacity of fastener groups (10d common nails versus #9 SST SD screws) that attach the hangers (10 fasteners into the ledger, six fasteners into the joist). The calculated design capacity for screws was 750 lb; whereas, the capacity for nails was 150 lb – a five-fold difference. One reason for the large difference in design capacity is the 75% reduction in withdrawal capacity for smooth-shank nails subject to wet/dry cycling specified in Table 10.3.3 of the NDS (AF&PA 2005).

Relying on any withdrawal capacity of joist hanger connections having nails subjected to tension is a potentially unsafe practice, in violation of model code provisions, and does not provide an element of structural redundancy. Some level of structural redundancy is recommended, even though in *ideal laboratory conditions* it was shown that sufficient withdrawal capacity could be provided by joist hanger connections when screws are used. It is important to note that both deck tests were conducted in a laboratory setting where materials were not exposed to environmental factors such as wet/dry cycles, and there was no wood decay or fastener corrosion present.

Ledger attachment -- Deck ledgers were attached with 0.5-inch diameter lag screws in a staggered pattern as specified in IRC Table R502.2.2.1. The research basis for the IRC provisions was Carradine et al. (2007; 2008). The deck ledger-to-house attachment appeared to be adequate for the conditions studied. When no tension hold-down connectors were used, the outer two lag screws carried most of the withdrawal load with no visible signs of failure (Figure 6).

Tension hold-down -- Tension hold-downs behaved in a counterintuitive way for the deck investigated. The

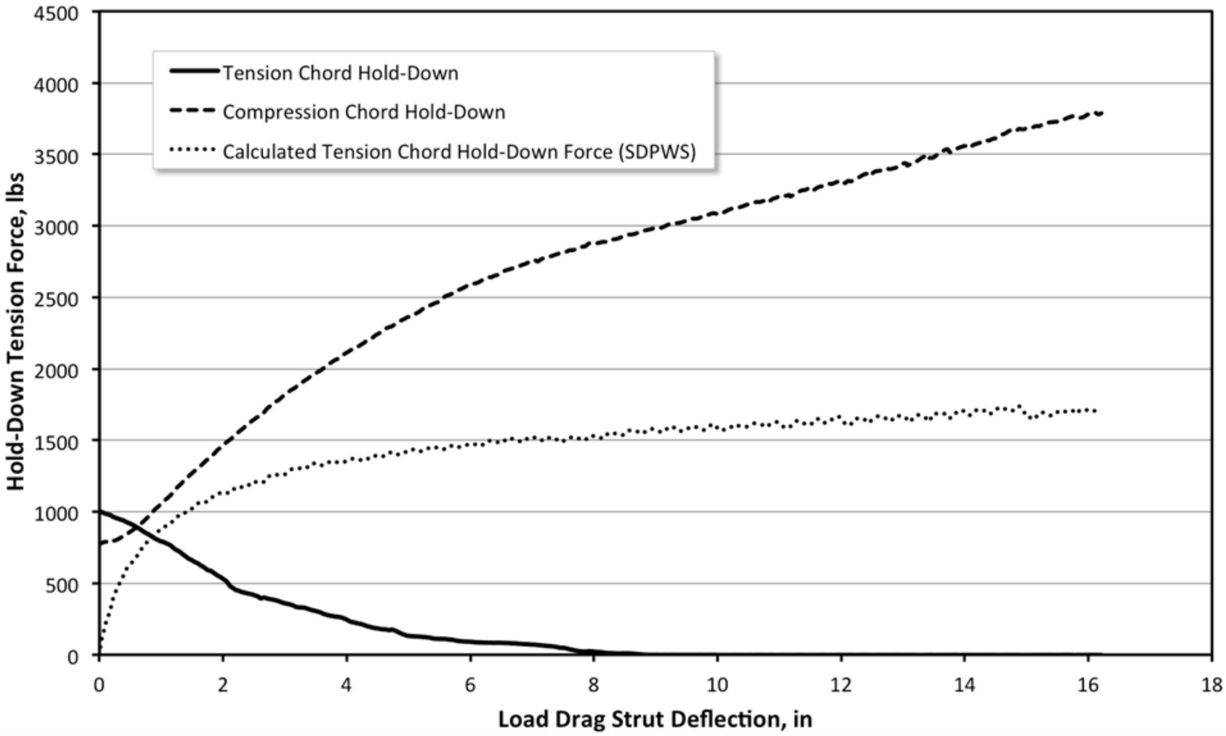

Figure 7. Recorded Hold-down Force Versus SDPWS Calculations

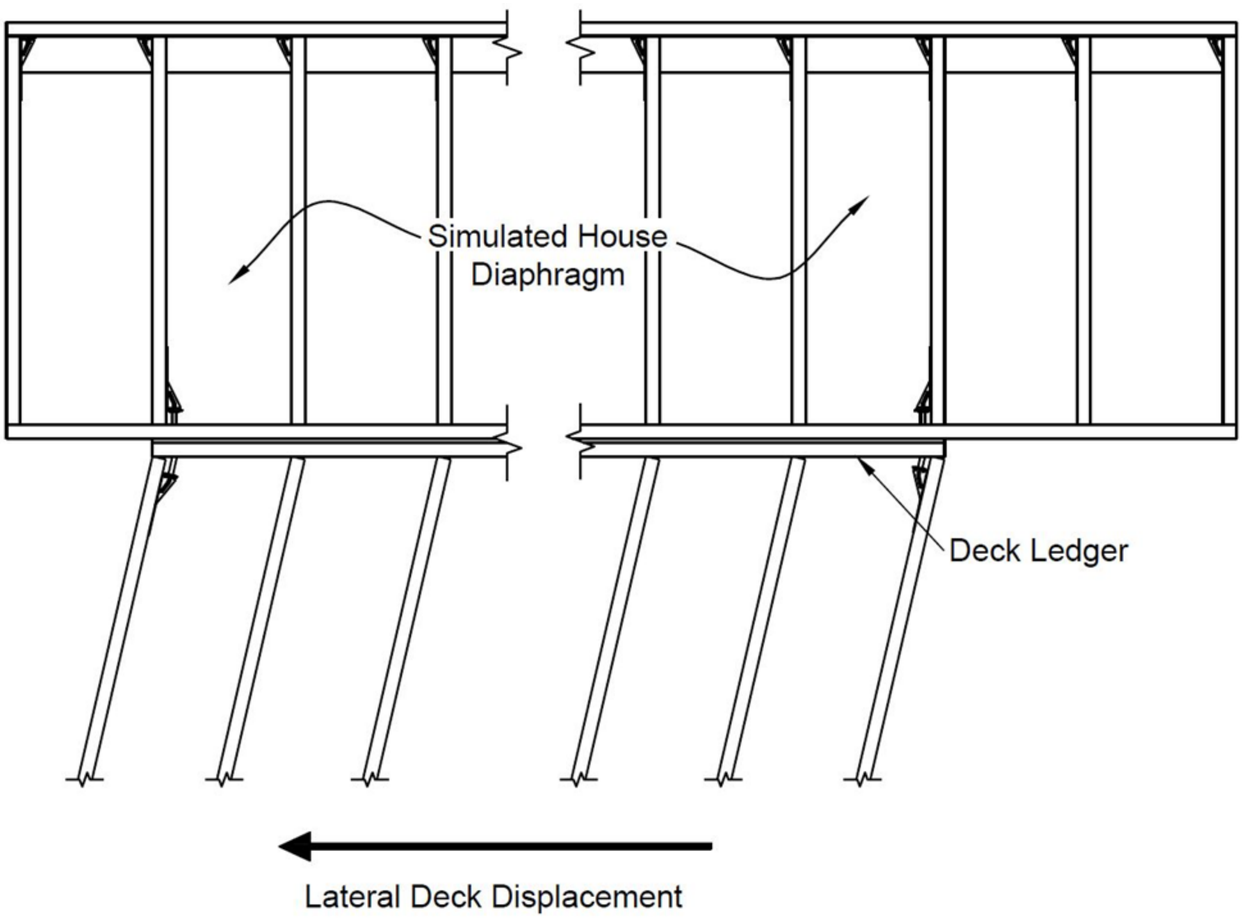

Figure 8. Plan View of Deck Joist Rotation and Resulting "Prying" Effect on Hold-down

flexibility of the deck allowed significant rotation of the deck joists within the joist hangers. This resulted in a geometric “prying” effect that caused zero tension in the “tension hold-down” and significant tension in the “compression hold-down” as shown in Figures 7 and 8. The hold-down connectors would behave in a more intuitive manner as the deck lateral stiffness is increased. While hold-down devices did not appear to significantly improve deck performance in the two decks tested that utilized screws in the hangers, hold-down devices do provide some level of structural redundancy for decks in service that naturally experience different levels of deterioration.

Conclusions

Prior to this study, little was known about the lateral performance and load path of exterior wood decks. To learn more about lateral strength and load path of decks, two 12 ft by 12 ft decks were attached to a simulated house diaphragm and laterally loaded to failure. One deck was constructed with a tension hold-down connection as described in IRC Section R507.2.3 and one without. The following conclusions have been reached based on simulated full-scale lateral load tests:

For two specific laboratory deck configurations *that utilized screws in the deck joist hangers*, no significant impact on short-term deck strength and stiffness was observed when two tension hold-downs were installed. A similar result would not be expected had nails been used in the joist hangers, since wet/dry cycling causes nails to lose 75% of withdrawal capacity as specified in Table 10.3.3 of the NDS (AF&PA 2005).

While code-conforming hold-down devices did not appear to significantly improve lateral-load deck performance in the two decks tested, these devices do provide a level of structural redundancy that improves in-service deck safety.

Hold-downs used in lateral load deck tests exhibited significant counterintuitive behavior. This outcome was due to geometric effects caused by large lateral deck displacements and rotations of deck joists in their hangers.

Testing was terminated before an ultimate strength was achieved at a load of approximately 7,000 lb for both decks. The two lag screws nearest the deck tension chord experienced the largest forces, yet did not fail in withdrawal. These results point to the effectiveness of 0.5-in diameter lag screws when selected and installed per the IRC deck ledger connection provisions in Table R502.2.2.1 (ICC 2009b).

The results obtained in this study should generally apply to decks with an aspect ratio of 1:1 and less, where aspect ratio is defined as the deck dimension perpendicular to the house divided by the dimension parallel to the house. The study results should not be applied to decks having an aspect ratio greater than 1:1 as the failure modes and deck behavior may substantially change.

Additional research is needed to study other deck constructions and aspect ratios and to investigate other methods to achieve lateral stiffness and load capacity, and structural redundancy for new and existing decks.

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