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.

References

American Forest and Paper Association (AF&PA). 2010. Design for Code Acceptance 6: Prescriptive Residential Wood Deck Construction Guide. AF&PA, Washington, DC

American Wood Council (AWC). 2008. Special Design Provisions for Wind and Seismic. ANSI/AF&PA SDPWS-2008, Washington, D.C.

American Society of Civil Engineers (ASCE). 2010. Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-10, Reston, Virginia.

Homan, S.W., Boase, A.J., Raider, C.J., Jensen, H., Matthews H.W., Smith, G.B., and Wetzel, C.H. 1932. Horizontal forces produced by movements of the occupants of a grandstand. American Standards Association Bulletin 3(4).

ICC. 2009. International Building Code (IBC). http://publicecodes.cyberregs.com/icod/ibc/2009/index.htm

ICC. 2009. International Residential Code for One-and-Two Family Dwellings (IRC). http://publicecodes.cyberregs.com/icod/irc/2009/index.htm

Parsons, B.J., Bender, D.A., Dolan, J.D., Tichy, R.J., and Woeste, F.E. 2013a. Lateral load path and capacity of exterior decks. ASCE Practice Periodical on Structural Design and Construction (in review).

Parsons, B.J., Bender, D.A., Dolan, J.D., and Woeste, F.E. 2013b. Deck and porch floor lateral loading by occupants. ASCE Practice Periodical on Structural Design and Construction (in review).

Garrett, G.H and Bender, D.A. 2013. Wind load determination for residential decks. Wood Design Focus. 23 (2):3-8.

Garrett, G.H, Bender, D.A., and Dolan, J.D. 2013. Seismic load determination for residential decks. Wood Design Focus. 23(2):9-14.

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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

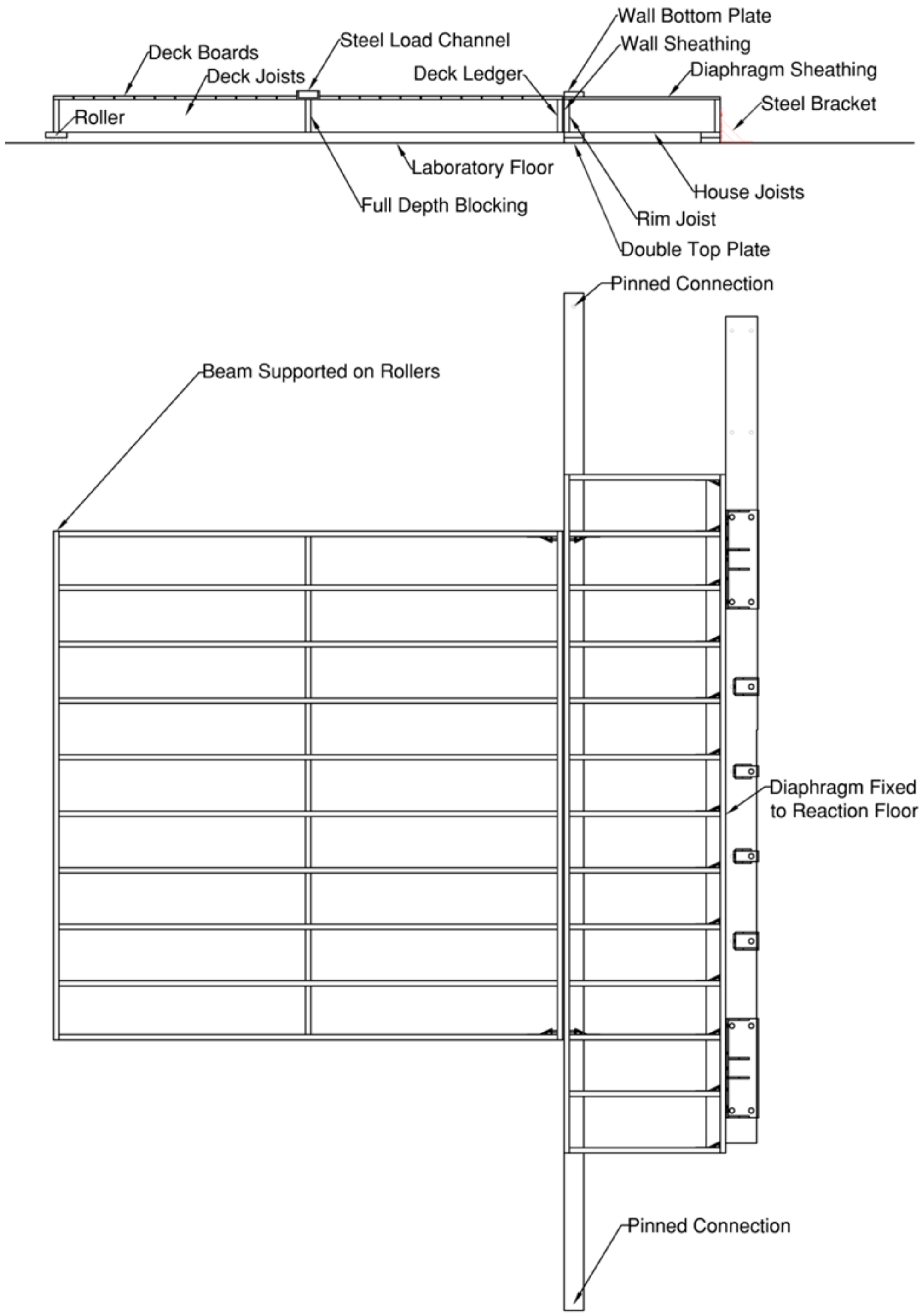

Figure 1. Elevation and Plan Views of Test Setup Construction

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, 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.

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^{TM} 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

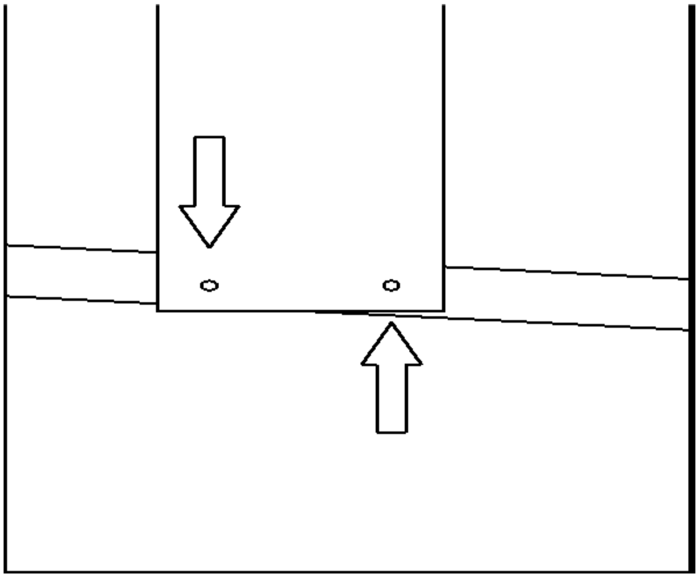

Figure 3. Deck Board to Joist Connection and Resisting Couple Providing Lateral Resistance

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

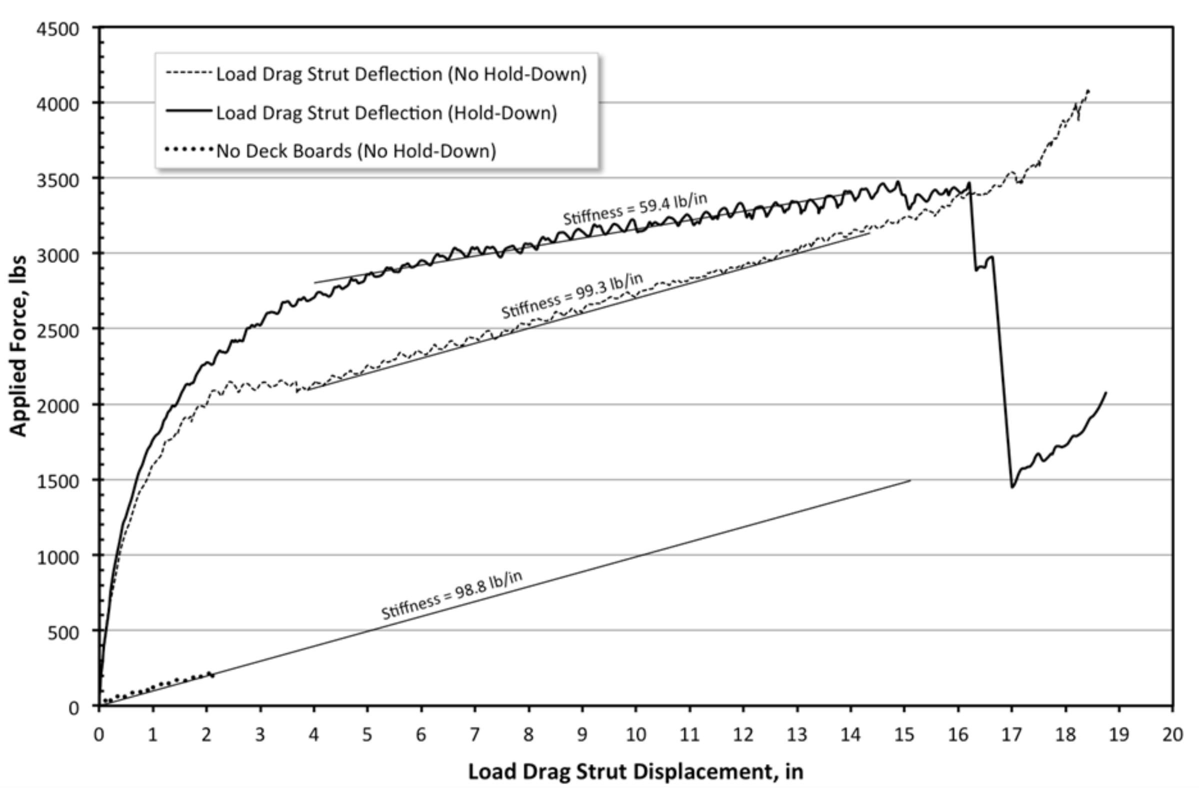

Figure 4. Load-Displacement Curves for Deck With and Without Hold-downs

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

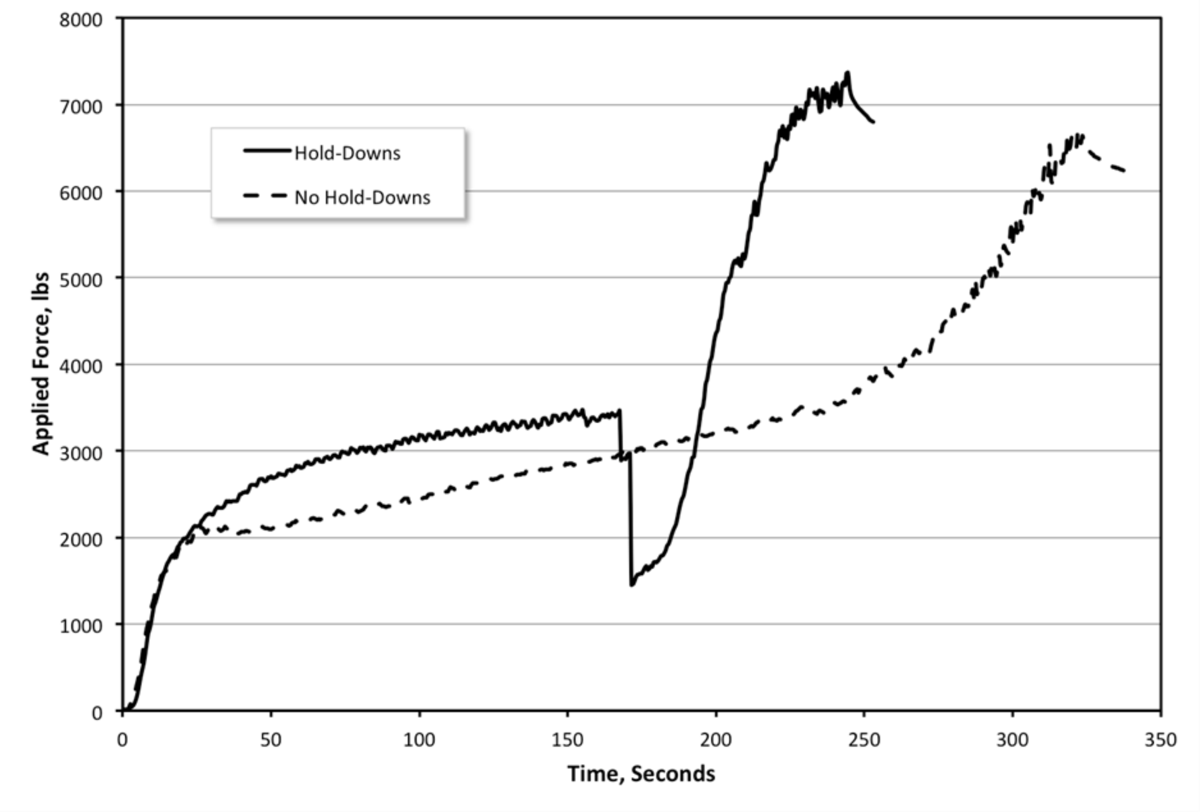

Figure 5. Load-Time Curves for Deck With and Without Hold-downs

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 pretension 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-

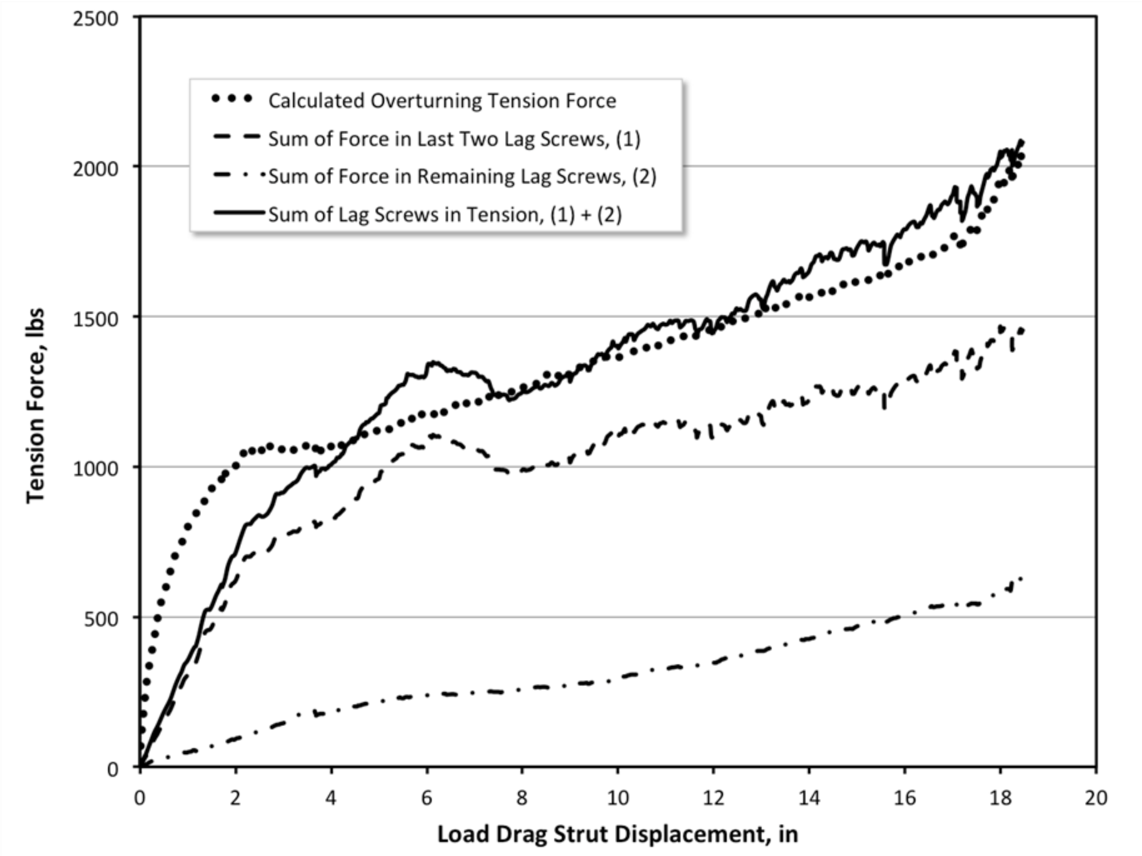

Figure 6. Lag Screw Forces on Deck Without Hold-downs. Overturning Tension Force Calculated Assuming End Joist Resists Full Overturning Moment

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

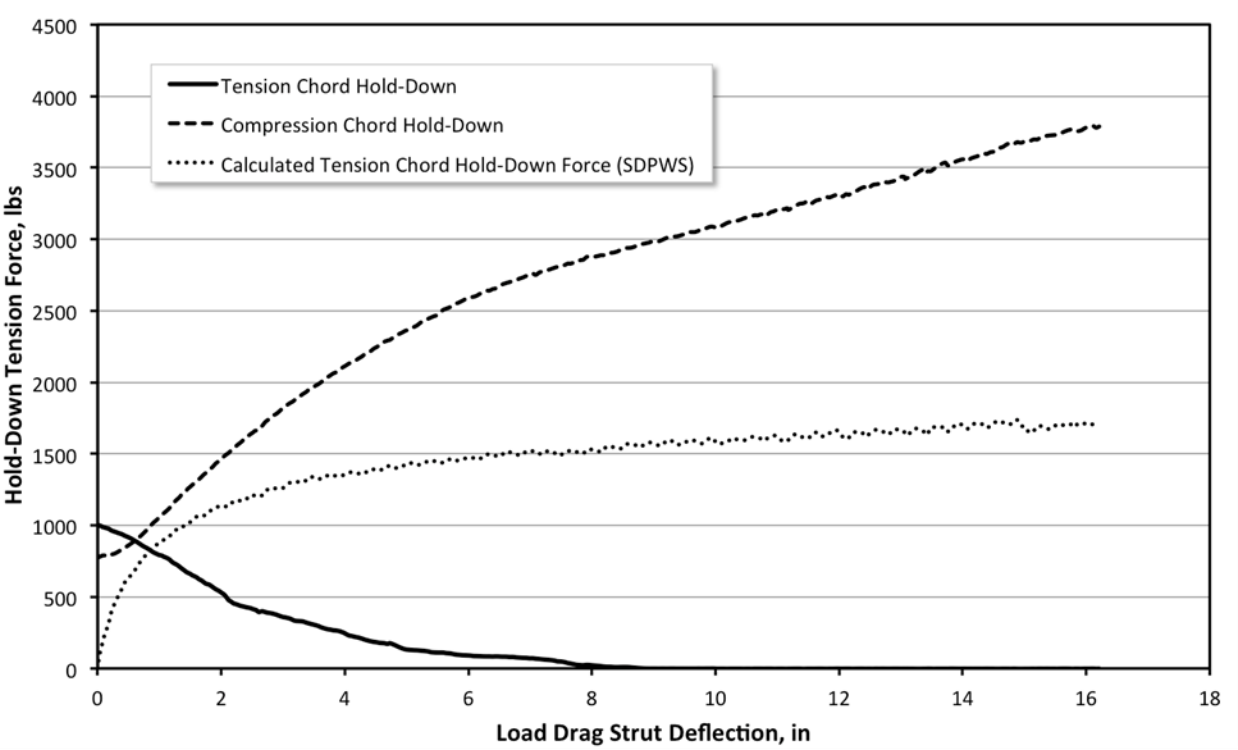

Figure 7. Recorded Hold-down Force Versus SDPWS Calculations

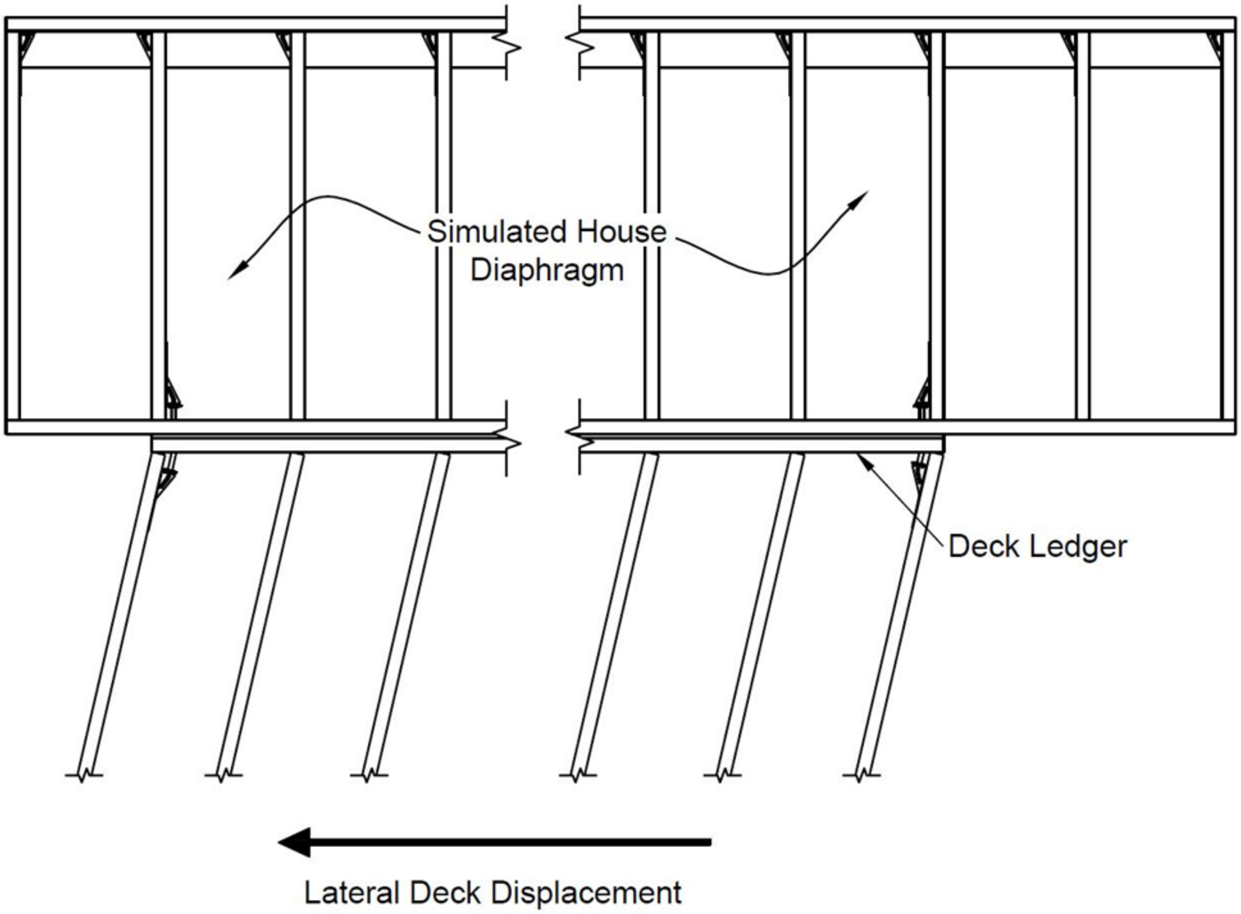

Figure 8. Plan View of Deck Joist Rotation and Resulting "Prying" Effect on Hold-down

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.

References

American Wood Council (AWC). 2008. Special Design Provisions for Wind and Seismic. ANSI/AF&PA SDPWS-2008, Washington, D.C.

American Forest & Paper Association (AF&PA). 2005. National Design Specification for Wood Construction. ANSI/AF&PA NDS-2005, Washington, D.C.

American Forest & Paper Association. 2001. Wood Frame Construction Manual for One- and Two-Family Dwellings. ANSI/AF&PA WFCM-2001, Washington, D.C.

Carradine, D.M., Bender, D.A., Woeste, F.E., and Loferski, J.R. 2007. Development of Design Capacities for Residential Deck Ledger Connections. Forest Products Journal 57(3), 39-45.

Carradine, D.M., D.A. Bender, J.R. Loferski and F.E. Woeste. 2008. Lessons learned: residential deck ledger connection testing and design. Structure magazine (May):53-56. www.structuremag.org/Archives/2008-5/C-LessonsLearned-DeckLedger_Carradine-May08.pdf

ICC. 2009a. International Building Code (IBC). http://publicecodes.cyberregs.com/icod/ibc/2009/index.htm

ICC. 2009b. International Residential Code for One-and-Two Family Dwellings (IRC). http://publicecodes.cyberregs.com/icod/irc/2009/index.htm

Legacy Services. 2010. Outdoor deck and porch injury study. http://www.buildingonline.com/news/pdfs/Outdoor-Deck-and-Porch-Injury-Study.pdf

Lyman, G.H. and Bender, D.A. 2013. Wind load determination for exterior decks and porches. Wood Design Focus. 23(2):3-8.

Lyman, G.H., Bender, D.A., and Dolan, J.D. 2013. Seismic load determination for exterior decks and porches. Wood Design Focus. 23(2):9-14.

Parsons, B.J. 2012. Understanding the lateral strength, load path, and occupant loads of exterior decks. MS Thesis, Department of Civil and Environmental Engineering, Washington State University, Pullman, WA.

Parsons, B.J., Bender, D.A., Dolan, J.D., Tichy, R.J., and Woeste, F.E. 2013. Lateral load path and capacity of exterior decks. ASCE Practice Periodical on Structural Design and Construction (in review).

Schutt, C.A. 2011. Improving deck safety. LBM Journal (May/June): 26-28. http://mydigimag.rrd.com/publication/frame.php?i=71105&p=28&pn=&ver=flex

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