design & construction

Designing for Drift…

Is Lateral Drift Accommodation in Exteriors Really Possible?

By Jeffrey S. Kersh, C.E. and Thomas Castle, S.E.

Current practices to accommodate lateral drift of non-bearing light gauge stud exterior systems cannot fully isolate the exterior system to receive no damage in a seismic or high wind event. The magnitude of lateral drift that needs to be accommodated is dependant on the type of lateral resisting system of the structure and the wind or seismic loads, and varies considerably. For example, in a moment frame design under seismic loading, the seismic relative displacement, D_p, can be as large as 2 to 3 inches per floor, whereas in a steel braced frame or concrete shear wall system under the same seismic loading, D_p can be more in the range of 0.5 to 0.75 inches. The exterior non-bearing walls must accommodate this lateral drift in two directions, in-plane and out-of-plane. In-plane lateral drift and out-of-plane lateral drift is accomplished in different ways. Both of these drifts can be achieved with various types of joints, tracks, and slotted clips. However, these methods of drift accommodation are incompatible at perpendicular wall intersections.

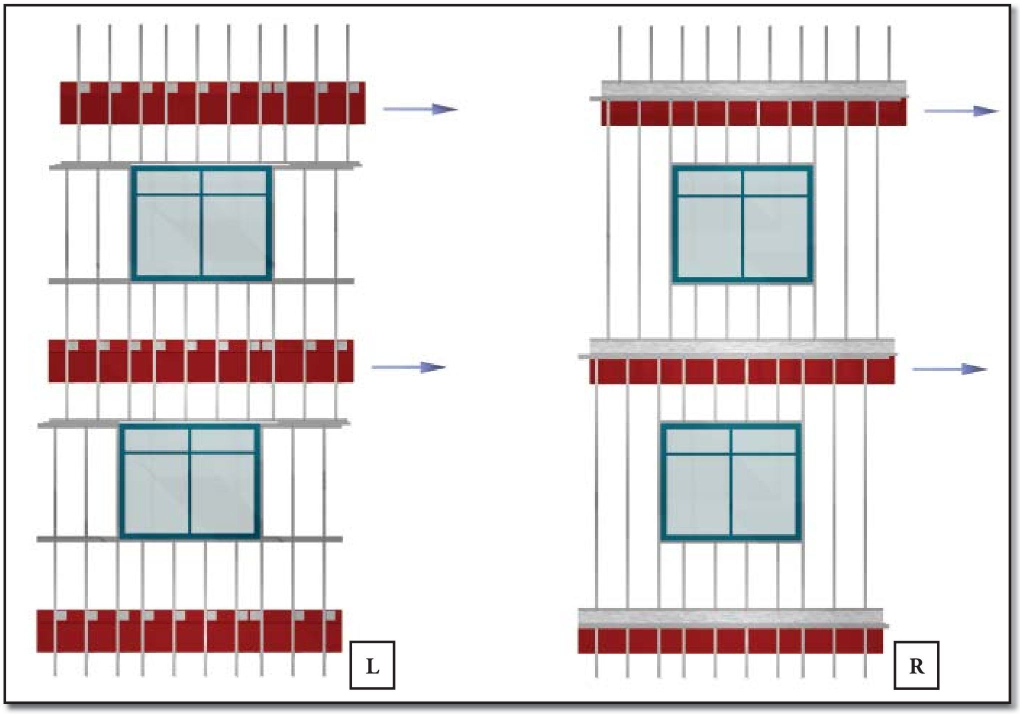


Figure 2: In-plate movement in spandrel framing (L) and floor-to-floor framing (R) with arrows indicating movement direction

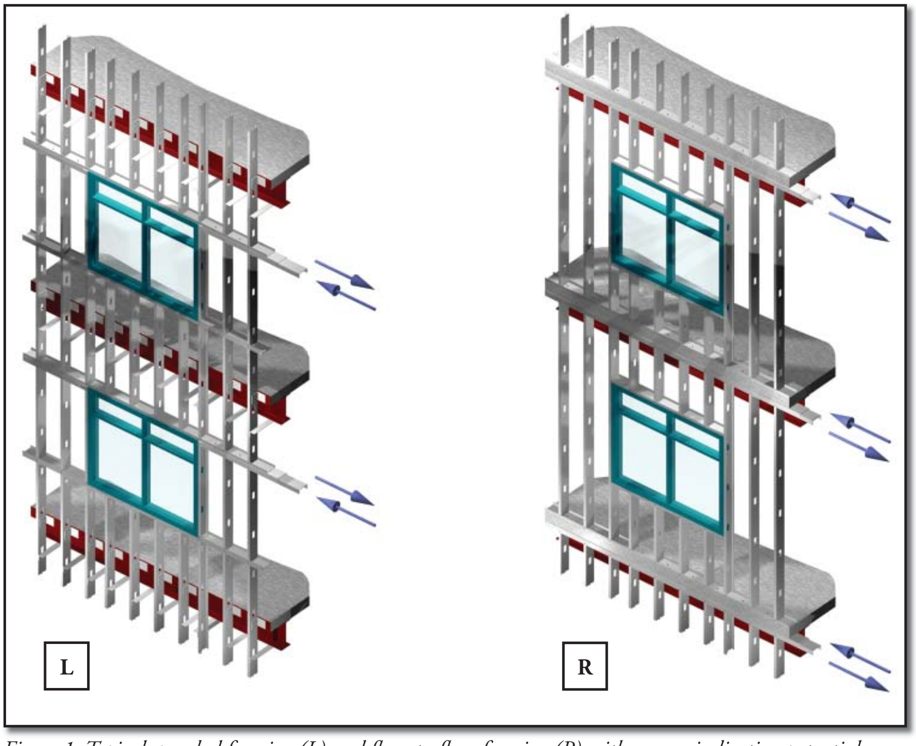


Figure 1: Typical spandrel framing (L) and floor-to-floor framing (R) with arrows indicating potential movement at joint location

There are presently two typical framing methods in the industry that accommodate lateral drift: 1) a floor-to-floor stud system and, 2) a spandrel stud system. In the first method, the exterior stud wall is designed to be rigidly attached to one floor and have a joint capable of both vertical and horizontal movement at the floor above (*See Figure 1-R*). This joint is typically accomplished with either a slotted slip track or a track nested loosely within another slightly wider track. A gap is left above the studs or inner track, depending on the system used, to allow the floor above to move vertically without loading the studs below. The horizontal movement of this joint is accomplished in the slip track by slotted holes in the web of the track in which the fasteners attached to the deck above can move within. In the nested track within a track joint, the horizontal movement is achieved by the inner track fitting loosely inside of the outer track, allowing the inner track to slide along the length of the outer track. Both of these joints allow a section of exterior stud wall to be attached rigidly to the slab below and be independent of the floor above, thus isolating each floor from one another for in-plane movement during a seismic or high wind event (*See Figure 2-R*). For out-of-plane movement during such an event, the wall will rotate in the top and bottom tracks at each level without bending the studs. Since the studs are framed floor to floor, each floor can rotate independent of one another (*See Figure 3-R, pg. 23*).

In the second method, the spandrel stud system, the exterior studs bypass the floor and attach rigidly to the edge of the floor to form a band around each level. Between these bands are either windows or in-fill studs with a joint capable of vertical and horizontal movement, similar to that discussed above, at the top of the windows or in-fill studs (*See Figure 1-L*). This method of framing is commonly used in office buildings where there are long bands of windows that are uniform in height. In this system, the in-plane lateral drift is accommodated through sliding of the joint at the top of the window. Each band of studs will move independent from the band above and below when subjected to lateral drift (*See Figure 2-L*). Out-of-plane lateral drift is accomplished in a similar way as the floor-to-floor method. The studs rotate in the top and bottom track connections between the spandrels (*See Figure 3-L, pg. 23*).

A third less common system is sometimes used. Here the studs bypass each floor with a rigid clip connection to one floor and a vertically and/or horizontally slotted clip connection to the floor(s) above. In this system, the in-plane lateral drift is taken in bending of both the rigid and slotted clips, or slipping of the horizontal slotted clips. The amount of lateral drift capable with this type of system is limited due to the small deflection capability of the clips. This type of system is generally used when the expected lateral drifts are very small, as would be the case in a steel braced frame or concrete shear wall structure. The out-of-plane lateral drift is accomplished by rotating the stud at the slotted clip connection.

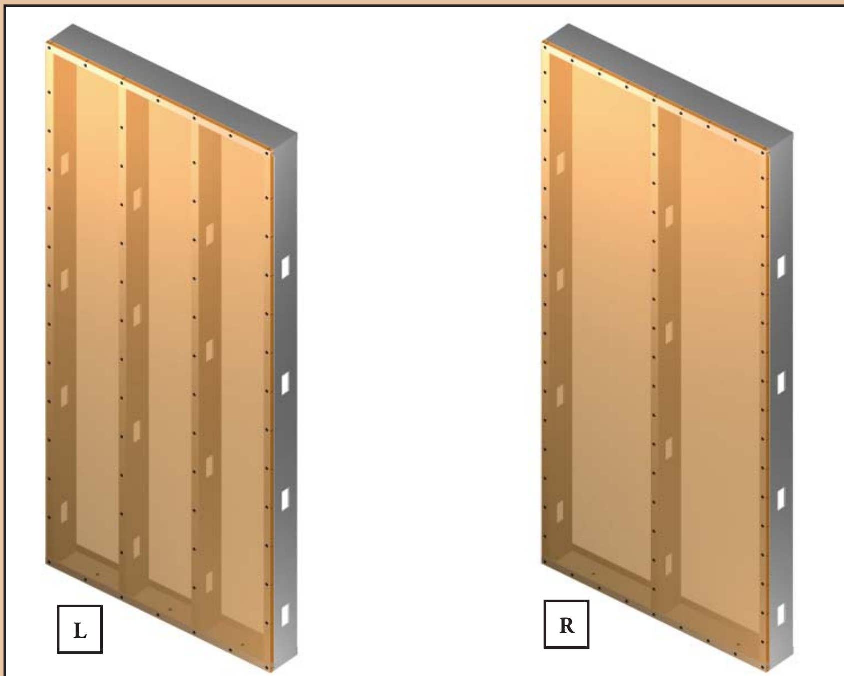
There are several methods of accommodating in-plane and out-of-plane lateral drift, only three of which were described earlier; however,

Continued on next page

16 or 24 inch?

By Thomas Castle, S.E.

No, this is not about the latest tire size for your SUV. It is about the spacing of non-bearing light gauge steel studs on commercial and residential building exteriors. In many cases these studs are specified to be placed at 16 inches on center. Why? This convention has its roots in the wood framing industry. However, many exterior finishes are capable of spanning 24 inches. The placement of the exterior studs at 24 inches on-center, as compared to 16 inches on-center, can significantly reduce the materials and labor involved in the installation of an exterior system. This savings is sometimes partially offset by increased size required due the increased loading, but many times increases in size are not necessary. In addition to the construction cost savings, thermal bridging can be reduced and result in increased energy efficiency, and effort required to place electrical and plumbing lines within the wall cavity can be reduced.



Typical 4 foot wide sheathing with 16-inch on center studs (left) and 24-inch on center studs (right)

The exterior finish deflection requirements and required wind loading will determine if 16- or 24-inch on-center spacing is required. The sheathing itself must be able to safely span without excessive deflection between the stud spacing. In addition, the sheathing must be capable of not pulling off the studs when subjected to the design negative pressures. 24-inch spacing will obviously have greater pullout requirements per stud than the16-inch on-center spacing. However, the number of fasteners can be increased to overcome this increase in force.

One of the most popular materials used on exterior systems is gypsum based panels. Some manufacturers have tested and published data for fastening requirements and allowable wind pressures for 24-inch on-center studs. The allowable wind negative pressures in pounds per square foot are usually in the mid to low 20's. This allowable negative pressure would be adequate for most structures less than 6 stories located in an urban environment, with a three second gust wind speed of 90 miles per hour or less. This system would not generally be acceptable for a gulf coast or eastern seaboard environment or taller structures. However, it is applicable to many urban structures throughout the remainder of the United States. As the trend continues toward more efficient construction, sheathing manufacturers are expected to develop more sheathing products capable of 24-inch spacing when subjected to larger loads. The potential saving in the light gauge framing would support a slightly larger cost of such sheathing products.

As a quick example, an 8 story hotel in San Francisco was constructed with a concrete flat slab design. The floor to floor spacing was 9-foot 6-inches. The exterior walls were required to be 6-inch depth for insulation reasons. The stud size required was a 600S162-33 at either 16- or 24-inches on-center. The decision to design for 24 inch on center spacing had the benefit of decreased material, labor and thermal bridging. The design negative pressure was 18.5 pounds per square foot.

Care should be taken in specifying a 24-inch spacing to meet all structural and serviceability requirements, but in many circumstances the saving over the traditional 16-inch spacing can be significant.▪

Light Gauge Steel Component Testing

By Matthew Stevens, S.E.

The behavior of cold-formed steel (CFS) elements is not always readily predictable or calculable. This is particularly true of connecting devices, where forces must be transferred from CFS framing to building structure through thin sections of irregular geometry, acting in some combination of bending, tension, shear and torsion. The ProX Clip™ shown in *Figure A below* illustrates this point. The back of the clip is screwed to a jamb stud and its tabs nest in and receive screws from a ProX Header™, a modified channel or box shaped header. The clip must transfer the vertical and out of plane loads from header to jamb. Calculating the capacity of complex load transfer mechanisms such as this requires use of either simplifying assumptions or advanced computer modeling, each with their own shortcomings. Section F of the *Specification for the Design of Cold-Formed Steel Structural Members*, published by the American Iron and Steel Institute (AISI), addresses such situations by specifying the test procedure to be followed to determine the structural performance of elements that cannot be evaluated using other Specification Provisions. Use of Section F's alternative procedures enables reliable performance values to be established. The following is a brief synopsis of the procedure with important issues noted.

Careful consideration must be taken when devising the actual test setup. Boundary conditions greatly affect the behavior of the tested specimens, and extra consideration must be given to ensure the desired element of the connection is actually being tested (i.e. the clip's tab capacity vs. that of its connection to the jamb), and that unrealistic behavior does not skew the results.

The AISI Specification states how the test results are to be manipulated to yield allowable design capacities, but the component evaluator must define "failure." Besides the obvious–complete loss of capacity–serviceability deflection limits may also be appropriate. In both the serviceability and ultimate load cases, the results are scaled to account for the deviation of strength and stiffness of the tested specimens from design values. For a given gauge and grade of steel, test values are scaled by ratios of the minimum design values with test specimen values of thickness and material strength. Determining which properties are affecting the test results is a critical part of this step. For example, is loss of capacity reached at the material ultimate stress (f_u dependent) or when support is lost due to excessive deflection (f_y dependent)?

For the ultimate or failure load case, a factor of safety must be calculated to obtain the maximum permissible load. The Factor of Safety calculation in the AISI Specification involves an intimidating formula containing numerous variables. Fortunately, most are provided in Section F, with values depending on type of component or failure mechanism. Variation in test results is perhaps the most significant test-specific variable. For serviceability criteria, the factor of safety equals one–the load that produces the defined deflection is taken as the maximum permissible load.

Once permissible loads for both serviceability and ultimate criteria are determined, the more conservative of the two is taken as the maximum allowable load.

At this point it can be useful to perform a rough "reality-check." After having witnessed the behavior of the component in the testing process, the evaluator should be better equipped to apply simple engineering principles to approximate member capacity.▪

Matthew Stevens, S.E. is a senior project engineer at Ficcadenti Waggoner and Castle Consulting Structural Engineers. He has participated in the physical testing and evaluation of numerous light gauge components. He also is active in the design of both structural and nonstructural light gauge steel structures and components.

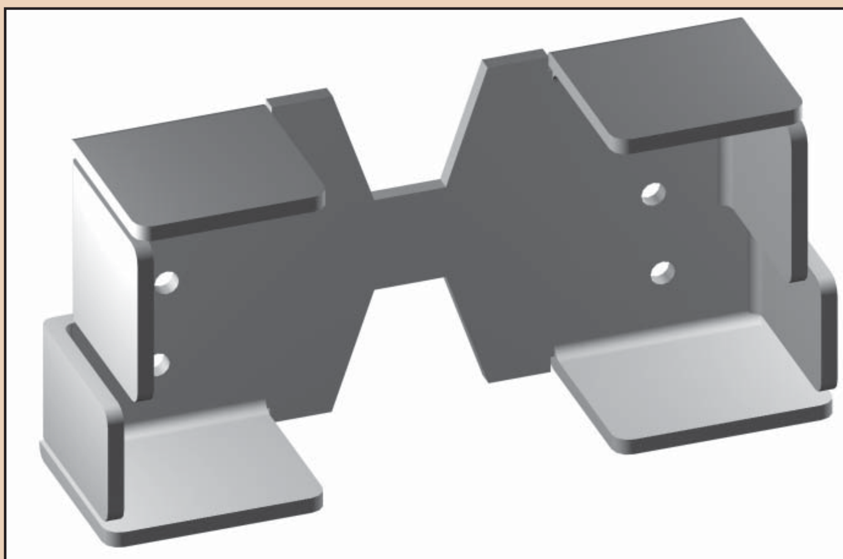


Figure A: Example candidate for testing

all of these systems can be incompatible at wall intersections. For in-plane drift, the movement occurs at a discrete joint location either at the bottom of the slab or top of the window. For out-of-plane drift the movement occurs over the height of the wall with rotation at the top and bottom. When these two drift accommodation methods meet at building corners, the walls will separate from one another or impact each other (*See Figure 4 on pg. 23*).

Current designs acknowledge that damage is expected at the corners during severe events. However, the wall is not anticipated to fall from the building nor cause a safety concern due to the continuity of the top and bottom tracks and ductility of the light gauge framing. This design philosophy is consistent with the stated objectives of national building codes. According to section 101.3 of the *2003 International Building Code*, the purpose of the code is to establish the minimum requirements to safeguard the public health, safety, and general welfare…, and safety to life and property from fire and other hazards attributed to the built environment, and to provide safety to fire fighters and emergency responders during emergency operations.

To limit damage at wall intersections, large vertical joints would need to be provided at all wall intersections. The required width of these joints would be the expected seismic relative displacement or maximum expected deflection under wind loading. Depending on the type of structural system and demand on the structure, the vertical joints could need to be anywhere from 0.5 to 3 inches wide. Since such vertical joints are generally undesirable, they are rarely specified. Frequently, this corner condition is

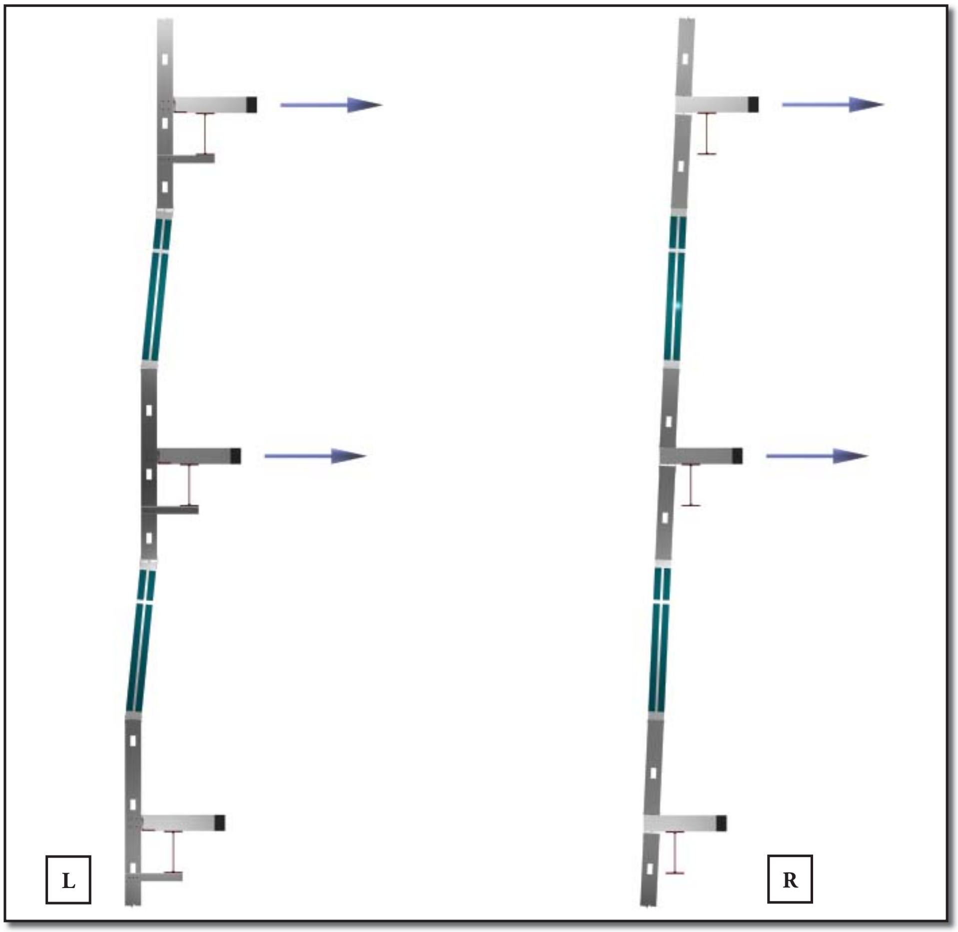


Figure 3: Out-of-plane movement in spandrel framing (L) and floor-to-floor framing (R) with arrows indicating movement direction

not considered nor well understood, and expectations of the performance of the exterior wall lateral drift accommodation system may not be realistic. This is especially true in buildings with short lengths of walls and numerous corners. In structures with long lengths of walls and few corners, the damage is expected to be limited to a small percentage of the total exterior system; however, should the building contain short walls and numerous corners, the percentage of damage could be very large. In fact, the damage to the exterior framing by attempting to accommodate lateral drift in the above manners may be larger then if the exterior system was rigidly attached for lateral movement and the studs were forced to bend from floor to floor in a seismic or high wind event. In recent years, the industry has been detailing more carefully to accommodate lateral drifts. This increased attention to drift accommodation can result in an unreasonable expectation of expected damage. Current design methods accomplish the task of accommodating lateral drift in a reasonable manner by limiting the expected damage to corners. Complete elimination of damage may not be possible without unacceptable measures. Special care should be taken when designing for drift in structures with numerous wall intersections.▪

Jeffrey S. Kersh, C.E., is a project engineer and Thomas Castle, S.E., is a principal at Ficcadenti Waggoner and Castle Consulting Structural Engineers (FW&C), located in Walnut Creek, CA. FW&C specializes in the design of light gauge framing.

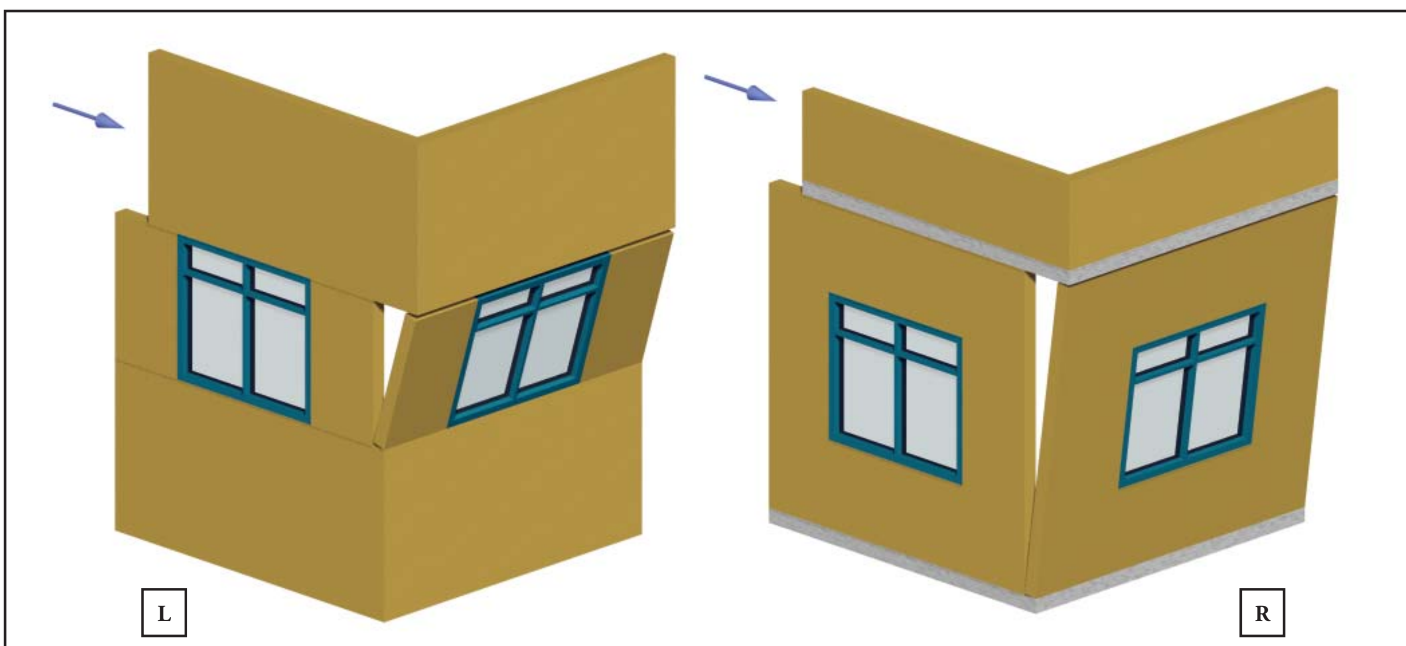


Figure 4: Intersection of walls with spandrel framing (L) and floor-to-floor framing (R) with arrows indicating movement direction