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INVESTIGATION OF UNDESIRABLE BRITTLE FAILURE OBSERVED IN HIGH-CAPACITY SHEAR WALLS

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ABSTRACT: The building height limit for light wood-frame construction has been increased from four to six stories in 2015 National Building Code of Canada (NBCC). In addition, the seismic design spectra in the 2020 NBCC has increased substantially for all site classes. These increases in building height and seismic loads have raised the demand for a stronger shear wall system for construction of mid-rise wood-frame buildings, especially for those located in high seismic zones. To respond to the demand for higher strength shear wall systems, a new high-capacity shear wall system with multiple rows of nails along sheathing edges has been jointly developed by FPInnovations and the University of Victoria. Shear walls with two and three rows of nails along sheathing edges were designed and tested in 2020, 2021 and 2022. In this paper, test results of high-capacity shear walls conducted in previous years were summarized. Brittle failure modes observed in previous test programs were investigated and causes for these brittle failure modes were discussed. New construction details for high-capacity shear walls to prevent these undesirable brittle failures were recommended.

KEYWORDS: Light wood frame construction, high-capacity shear wall, brittle failure modes, reversed cyclic load

1 – INTRODUCTION

Light wood frame construction is the predominant method of construction for single-family and low-rise multi-family dwellings in North America. Since 2009 and 2015, province of British Columbia and federal jurisdictions in Canada have respectively amended its building codes to allow up to 6-storey mid-rise wood frame buildings. The code amendment has expanded the application of light wood frame buildings to new markets which have been dominated by other construction materials. In the past ten years, around 2,500 five- and six-storey light wood frame buildings have been built across Canada. Over 1.2 million cubic meters of wood have been used.

The increase in height in mid-rise wood frame buildings has created additional demand for lateral load resistance. In addition, modern applications of wood construction often feature large floor spans, concrete topping, and heavy tiles on the roof. As a result, the overall lateral loads acting on the lateral load resisting system (LLRS) become greater, which is an especially critical issue for wood buildings in British Columbia and other regions susceptible to severe earthquakes. The seismic design spectra in the 2020 NBCC [1] has been increased substantially for all site classes. The combined effects above make it difficult, if not impossible, for designers to use the existing design solutions to resist the seismic loads in mid-rise wood frame buildings. If new effective solutions to accommodate the increased seismic loads are not developed, designers may completely abandon mid-rise wood frame construction in the highest seismic zones in BC.

The shear strength and stiffness of light wood-frame shear walls in Canadian timber design code CSA O86 [2] is limited to shear walls with single row of nails along sheathing panel edges. Although the sheathing panels can be placed on both sides of the wall (often referred to as double sheathed walls) to double the strength of the shear wall, the double sheathed shear wall can significantly constrain the ability to embed mechanical and electrical services within the wall cavities [3]. As a result, the application of double sheathed shear wall is limited.

To address the above issue, a new high-capacity shear wall system with multiple rows of nails along sheathing edges has been jointly developed by FPInnovations and the University of Victoria. In this paper, test results of high-capacity shear walls were summarized. Brittle failure modes observed from the test programs were discussed and reasons for causing these brittle failure

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modes were analyzed. New construction details for high-capacity shear walls to prevent these undesirable brittle failures were recommended.

2 – SUMMARY OF HIGH-CAPACITY SHEAR WALL TEST PROGRAM

A total of 30 shear walls, consisting of 2 × 4 or 2 × 6 studs and with two or three rows of nails along sheathing panel edges, were tested [4, 5, 6]. Table 1 summarizes the key parameters of shear wall specimens.

The shear wall specimens are 8 ft × 8 ft (2.4 m × 2.4 m) in dimension. Fig 1 illustrates detailed nailing patterns of tested shear walls with two and three rows of nails. The framing members were connected using F1667 NLCMMS69 (76.2 mm in length × 3.05 mm in shank diameter × 7.18 mm in head diameter) power driven common nails, with two rows of nails spaced at 200 mm on center for built-up end studs, built-up interior end studs, and top and bottom plates. To prevent separation of the built-up center studs, two rows of F1667 NLCMMS69 common nails spaced at 75 mm on center were used in 2020-21 test program, and three rows of nails spaced at 100 mm on center were used in 2021-22 and 2022-23 test programs. To prevent the loading beam from interfering with the rotation of the sheathing when the wall is laterally displaced, one additional lumber plate was added on top of the wall.

The shear wall specimens were fabricated and installed according to ASTM E2126 [7]. For all shear wall specimens, continuous tie rods (hold-downs) were used to resist overturning moment. Six anchor bolts with steel plate washers were used to connect both top and bottom plates to load spreader beam (C180 × 18) and foundation steel beam, respectively. Steel plate washers, 75 mm × 75 mm × 6 mm and 127 mm × 127 mm × 6 mm, were used for shear walls with 38 mm × 89 mm and 38 mm × 140 mm framing members, respectively.

Reversed cyclic loading was applied on tested specimens, following the CUREE protocol (method C) in ASTM E2126 [7]. The reference displacement was taken as 63.5 mm (2.5 in.). Each subsequent phase of the CUREE protocol consisted of a primary cycle with an increase in amplitude of α (= 0.5) over the previous primary cycle. A displacement rate of 7.6 mm per second was used for the reversed cyclic loading. The test was terminated when the load dropped by more than 20% of the maximum load.

Fig 2 compares the load-displacement curves of the high-capacity shear walls with their corresponding reference shear walls. Table 2 summarizes the stiffness, yield load and yield displacement, peak load, ultimate displacement and energy dissipation capacity of shear wall specimens, where the secant stiffness K_e is obtained between the origin and the point with 40% of maximum load on the ascending phase; P_{yield} is the yield force based on EEEP method and Δ_{yield} is the corresponding displacement; the ultimate displacement Δ_u is in the post-peak region where the load drops to 80% of the maximum load (P_{peak}) or failure of the specimen happens; μ is the ductility ratio, defined as the ratio between ultimate displacement over the yield displacement; E is the total energy dissipated in the hysteresis loops.

Table 1: Matrix of Shear Wall Test Programs

Test program	Specimen number	Framing member [mm]	Sheathing thickness [mm]	Nail size	Rows of nails	Nail spacing along panel edges [mm]	Plate washer at hold-down	Two rows of nails on single stud / plate	End stud tied to plate
2020-21	1a	38 × 89	9.5	8d	2	75	No	Yes	No
	1b, 1b′	38 × 89	9.5	8d	2	75	No	No	No
	2b-1, 2b-2	38 × 89	9.5	8d	2	100	No	No	No
	3b′, 3c′	38 × 89	15	10d	2	75	No	No	No
	4b′, 4c	38 × 89	15	10d	2	100	No	No	No
	3r′	38 × 89	15	10d	1	75	No	No	No
2021-22	1	38 × 140	11	8d	3	100	No	Yes	No
	1′	38 × 140	11	8d	3	100	Yes	Yes	No
	2	38 × 140	11	8d	3	75	No	Yes	No
	2′-1, 2′-2	38 × 140	11	8d	3	75	Yes	Yes	No
	2r	38 × 140	11	8d	1	75	Yes	No	No
	3-1, 3-2, 3-3	38 × 140	15	10d	3	100	Yes	Yes	No
	4-1, 4-2	38 × 140	15	10d	3	75	Yes	Yes	No
	4r	38 × 140	15	10d	1	75	Yes	No	No
2022-23	1-1, 1-2	38 × 140	12	8d	1	75	Yes	No	No
	2-1, 2-2	38 × 140	12	8d	2	75	Yes	Yes	Yes
	3-1, 3-2	38 × 140	12	8d	3	75	Yes	Yes	Yes
	4	38 × 140	12	8d	3	75	Yes	Yes	Yes

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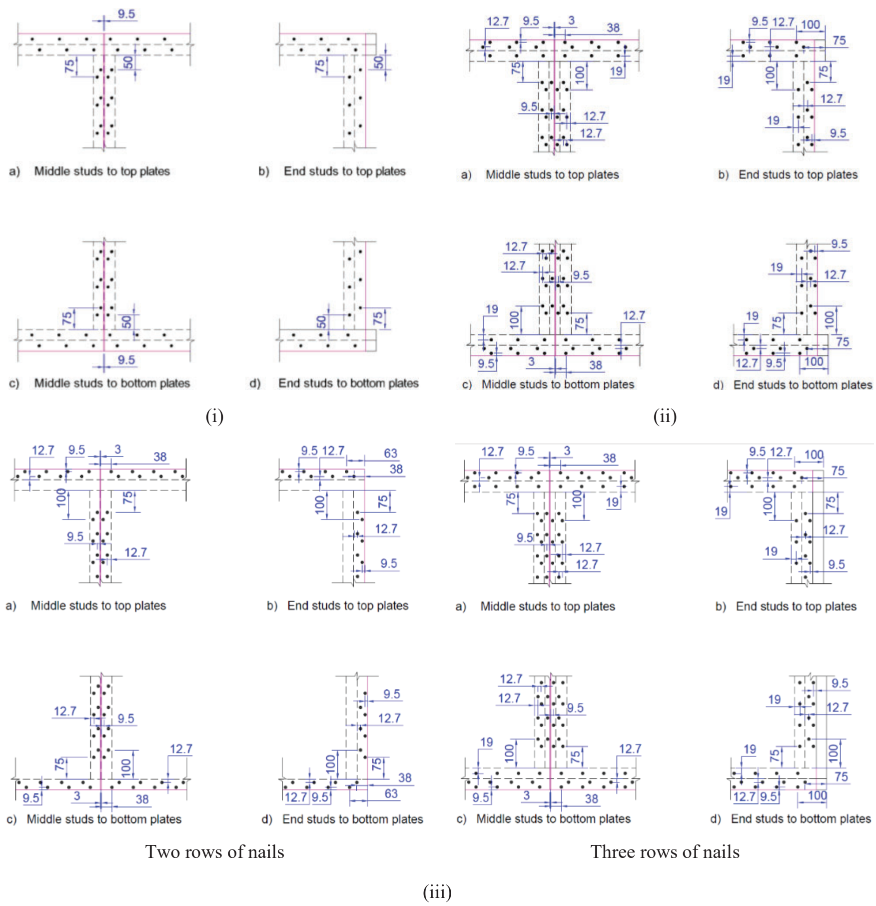

Figure 1. Nailing patterns in shear wall specimens: (i) 2020-21 test program, (ii) 2021-22 test program, (iii) 2022-23 test program.

The results show that in general, the peak load of the shear walls is proportional to the number of rows of nails. However, the ductility ratios of shear walls with multiple rows of nails are lower than the corresponding reference shear wall. This is because while high-capacity shear walls have larger yield displacement and ultimate displacement than a reference wall with the same sheathing panel and nail size and spacing, the increase of yield displacement is greater than that of ultimate displacement in high-capacity shear walls. As a result, the ductility ratio, which is the ratio between ultimate displacement over yield displacement, is reduced.

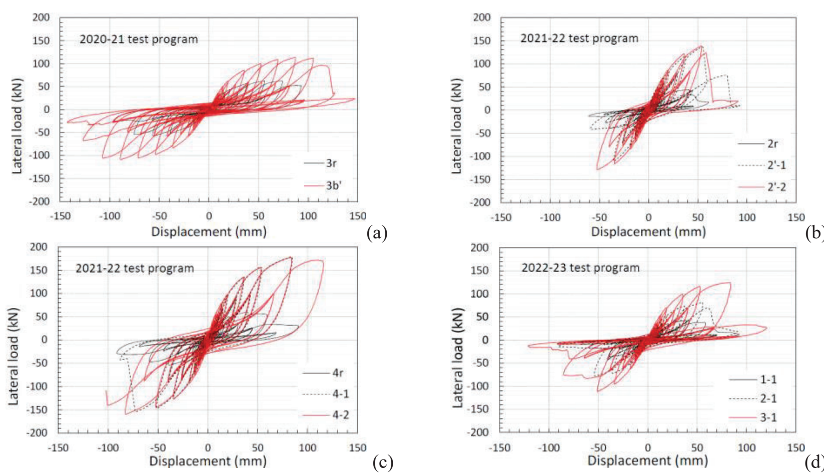

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Figure 2. Comparison of hysteresis loops of high-capacity and reference shear walls: (a) 2020-21 test program, (b) 2021-22 test program, (c) 2021-22 test program, (d) 2022-23 test program.

Table 2. Mechanical Properties of The Tested Shear Walls Based on ASTM E2126 [7]

Test program	Specimen number	K_e *[kN/mm]*	P_{yield} *[kN]*	Δ_{yield} *[mm]*	P_{peak} *[kN]*	Δ_u *[mm]*	μ *[Δ_u/Δ_{yield}]*	E *[kN·m]*
2020-21	1a	3.22	87.3	27.0	95.2	90	3.3	32.5
	1b, 1b′	2.78	85.6	31.0	96.4	94.5	3.0	40.6
	2b-1, 2b-2	2.64	67.0	26.0	74.1	83.5	3.3	26.2
	3b′, 3c′	3.25	100.0	30.5	111.4	122.5	4.0	78.0
	4b′, 4c	2.82	76.2	26.5	85.6	101.5	3.8	39.0
	3r′	2.67	53.3	20.0	59.8	84.0	4.2	23.1
2021-22	1	4.58	74.4	16.2	91.5	35.6	2.2	11.2
	1’	3.50	89.7	25.6	99.0	96.6	3.8	40.5
	2	4.00	96.3	24.1	116.7	54.1	2.2	-
	2′-1, 2′-2	5.41	105.7	19.6	123.9	54.0	2.8	24.3
	2r	5.29	37.4	7.1	41.7	46.9	6.6	8.7
	3-1, 3-2, 3-3	4.75	117.0	24.8	133.6	83.7	3.4	46.6
	4-1, 4-2	5.90	146.1	24.9	167.6	93.0	3.8	51.0
	4r	4.92	49.8	10.1	57.2	62.9	6.2	17.0
2022-23	1-1, 1-2	3.88	31.5	8.2	35.7	51.5	6.5	7.7
	2-1, 2-2	3.51	68.0	19.6	77.8	65.8	3.4	18.3
	3-1, 3-2	4.34	96.1	22.3	109.8	76.4	3.4	28.5
	4	4.9	103.8	21.2	121.0	66.1	3.1	26.1

3 – INVESTIGATION OF UNDESIRABLE BRITTLE FAILURE MODES

For high-capacity shear walls with two or three rows of nails, besides the failure modes observed in regular shear walls such as nail withdrawal from studs, nail head pull through sheathing, nail chip-out at sheathing edge and nail fracture, other failure modes related to framing members and sheathing panels were also observed with the increasing lateral load capacity. As these failure modes are brittle in nature, they should be prevented. In this section, the brittle failure modes observed in the tests are discussed and the causes for these failure modes are analyzed. Recommendations on how to prevent these failure modes are also given.

3.1 BOTTOM PLATE SPLITTING

The splitting of bottom plates was observed in high-capacity shear walls where two rows of nails were located on a single bottom plate, as shown in Fig 3. The splitting of the bottom plate in shear walls is caused by the force component of sheathing-to-framing nailed joints perpendicular to the grain of the bottom plate at the end of the shear wall. Based on the linear elastic theory [8], the lateral shear forces of sheathing-to-framing nailed joints act at an angle around the corners of the sheathing. With force component acting perpendicular to the bottom plate, a bending moment across the grain of the bottom plate (cross-grain bending) is generated and causes the bottom plate to split due to the low tensile strength of wood members perpendicular to grain, as shown in Fig 4 [9].

To reduce the cross-grain bending moment on the bottom plate, it is recommended that one row of nails on each bottom plate be used in high-capacity shear walls. This will reduce the cross-grain bending moment on the bottom plate by half, consequently reducing the possibility of the bottom plate splitting. In addition, plate washers can be placed close to the end of the high-capacity shear walls to further reduce the cross-grain bending moment on bottom plate. Fig 5 shows a plate washer installed at location of hold-down.

Figure 3. Bottom plate splitting (specimen 1a in 2020-21 test program).

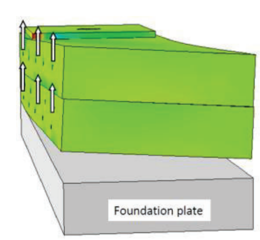

Figure 4. Cross-grain bending moment due to force component perpendicular to bottom plate.

Figure 5. Plate washer at location of hold-down.

Figure 6. Separation between end studs and bottom plates (specimen 4-1 in 2021-22 test program).

3.2 SEPARATION OF STUDS AND PLATES

Separation of end studs from plates was observed in high-capacity shear walls, as shown in Fig 6. This is due to the eccentricities between the lateral force of sheathing-to-framing nailed joints and the center of framing member. As a result, out-of-plane moments are created.

While the out-of-plane moment on top and bottom plates are resisted by the lateral supports installed on both sides of the loading beam and the anchor bolts installed along the centreline of the bottom plate, respectively, the out-of-plane moment on studs are resisted by the plate-to-stud nailed joints and the sheathing which hold the studs and plates together though sheathing-to-framing nailed joints. Once the resistance of plate-to-stud nailed joints and nail head pull-through or withdrawal resistance of sheathing-to-framing nailed joints along the top or bottom plates near the end of shear wall is not enough to resist the out-of-plane force of the studs, sheathing would be pulled away from the plates, causing separation between studs and plates.

Fig 7 shows a schematic of force distribution on end stud. To prevent the separation, studs and plates should be connected by metal straps or connectors. These metal straps or metal connectors should be able to resist the out-of-plane force, F_e, at the ends of end studs, which can be estimated as follows:

$$F_e = \frac{M_e}{h} = \frac{n_r n_u \left(\frac{h}{s}\right) e}{h} = \frac{n_r n_u e}{s} \qquad (1)$$

Where M_e is the out-of-plane moment, h is the clear distance between top and bottom plates, n_u is the lateral resistance of sheathing-to-framing nailed joint, n_r is the number of rows of nails, s is the nail spacing, and e is the eccentricity distance which is half of the stud or plate width.

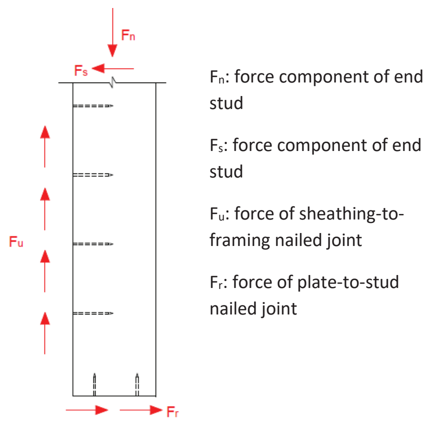

Figure 7. Free body of end stud.

3.3 END STUD SPLITTING

Stud splitting at end studs was observed in high-capacity shear walls. Fig 8 shows the failure of stud splitting. A closer examination reveals that the splitting occurred on studs where two rows of nails were installed on a single stud.

Like the force component perpendicular to the bottom plate that caused the splitting of bottom plate due to cross-grain bending (see Section 3.1), it is believed that the force component perpendicular to studs caused the studs to split. To prevent stud splitting, it is recommended that one row of nails be installed on a single stud. This will effectively reduce the force component perpendicular to studs to half. As stud splitting is hardly noticed in regular shear walls (with one row of nails in a stud), it is expected that stud splitting can be prevented in high-capacity shear walls. Where angle brackets are installed at the ends of studs to prevent out-of-plane movement, it is suggested that they be installed closer to the face of the stud where sheathing is nailed. This would reduce the eccentricities between the shear plane and the

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Figure 8. End stud splitting (specimen 2-2 in 2022-23 test program).

Figure 9. Sheathing rupture (specimen 2′-1 in 2021-22 test program).

angle brackets, consequently reducing the potential cross-grain bending.

3.4 SHEATHING RUPTURE

Sheathing rupture was observed in shear walls with 11 mm (7/16 in.) thick sheathing (specimen 2′-1) in the 2021-22 test program and shear walls with 12 mm (15/32 in.) thick sheathing (specimen 3-2) in 2022-23 test program, as shown in Fig 9. As sheathing rupture did not occur on similarly constructed wall (specimen 2′-2 in 2021-22 test program and specimen 3-1 in 2022-23 test program), this might indicate that the sheathing rupture is caused by some localized weakness in the OSB sheathing. As the rupture occurred at 45° of the sheathing, this indicates that the OSB sheathing strength is not governed by sheathing shear through thickness, but the axial strength at 45° to the sheathing's principle directions.

Based on Table 9.3 of CSA O86 [2], the strong and weak axis tensile strength of 11 mm thick OSB panel is 60 N/mm and 30 N/mm, respectively. The shear through thickness strength is 46 N/mm in both directions. As the weak axis tensile strength is lower than the shear through-thickness strength, panel rupture may be governed by axial strength, rather than shear through thickness. Assuming plastic theory is applicable and that the sheathing panel in a shear wall is under pure shear [10], the maximum axial stress in the panel will be at an angle of 45° based on Mohr's circle. Assuming the axial strength of OSB panel at an angle follows Hankinson's formula, the axial strength for 11 mm thick OSB panel at 45° is roughly 40 N/mm, which is smaller than the shear through-thickness strength. This means that for an 11 mm thick OSB panel, the failure mode will likely be in axial tensile.

Sheathing rupture was also observed in shear walls after the bottom plate splitting, separation of end studs from bottom plates, and sheathing buckling. It is believed that the rupture is caused by tearing or compression of sheathing. They can be prevented once the root cause is prevented.

3.5 SHEATHING BUCKLING

Sheathing buckling was observed in shear walls with 9.5 mm (3/8 in.) thick sheathing (specimen 1b′) in 2020-21 test program, shear wall with 11 mm (7/16 in.) thick sheathing (specimen 1′) in 2021-22 test program, and shear walls with 12 mm (15/32 in.) thick sheathing (specimens 2-1 and 4) in 2022-23 test program. Fig 10 shows the sheathing buckling failure.

Although design calculation in accordance with CSA O86 [2] indicates that the tested shear walls are governed by sheathing buckling, test results show that the lateral load resistances are still governed by the lateral load capacities of sheathing-to-framing nailed joints. The sheathing buckling was observed after shear wall reached peak lateral load resistance when the stiffness of sheathing-to-framing nailed joint is significantly reduced. While the sheathing buckling does not govern the lateral load resistance of shear wall, it may affect the ultimate displacement of shear wall, causing shear wall to fail earlier than anticipated.

The equation for panel buckling resistance used in CSA O86 [2] is developed based on the assumption of simply supported thin plate on four edges [11]. Since the contribution of nailed joints located in the intermediate studs of the panel is ignored, the panel buckling resistance based on the equation in CSA O86 [2] is overly conservative. In addition, the multiple rows of nails installed along sheathing edges would likely restrain the rotation of panel edges, therefore further increasing the panel buckling resistance.

Based on Källsener & Lam [12], the critical buckling stress of a panel under pure shear stress can be calculated as:

$$\tau_{cr} = k \frac{\pi^2 E}{12(1-\nu)^2}\left(\frac{t}{b}\right)^2 \qquad (2)$$

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Where t is the thickness of the panel and ν is Poisson's ratio. The value of the coefficient k depends on the length-to-width ratio of the panel and on the boundary conditions. For a panel simply supported along all four edges, an approximate expression for the coefficient k is given by

$$k = 5.35 + 4\left(\frac{b}{a}\right)^2 \tag{3}$$

For a panel clamped along all four edges the corresponding expression for the coefficient k is given by

$$k = 8.98 + 5.6\left(\frac{b}{a}\right)^2 \tag{4}$$

For a wall panel with intermediate studs, Källsener & Lam [12] suggested that b be taken as the spacing between adjacent studs since the panel is supported along the studs. As critical buckling stress of a panel clamped on four edges is not provided in the report by Dekker et al. [10], the k values in (3) and (4) are used to get a glimpse of the influence of boundary conditions on panel buckling resistance. Table 3 shows the k values with different boundary conditions based on (3) and (4).

It shows that for a panel clamped along all four edges, the critical buckling stress is approximately 67% greater than that of a panel simply supported along all four edges. For a panel with intermediate stud, the k values are about 10% smaller than those of a panel without intermediate stud. Although the k value is reduced for a panel with intermediate stud, the critical buckling stress is increased due to smaller panel dimension b. With panel dimension b reduced to half, the critical buckling stress of a panel with intermediate stud is approximately 3.6 times that of a panel without intermediate stud.

In practice, the number of nails installed at intermediate studs are much less than those installed along panel edges. This indicates that the actual critical buckling stress for a panel with intermediate stud is likely somewhere between the buckling strengths with and without full supports at studs. More study needs to be conducted to estimate the effect of intermediate studs on critical buckling stress of a panel.

Figure 10. Sheathing buckling (specimen 1' in 2021-22 test program).

Table 3. k Value Under Different Boundary Conditions

Panel size		**Boundary condition**		**Clamped / Simply supported**
a	***b***	***Simply supported***	***Clamped***	
[mm]	***[mm]***	***[-]***	***[-]***	***[-]***
2400	1200	6.35	10.38	1.63
2400	600	5.60	9.33	1.67

4 – RECOMMENDATIONS ON DESIGN AND CONSTRUCTION DETAILS OF HIGH-CAPACITY SHEAR WALLS

Based on the findings obtained from the high-capacity shear wall test programs, it is recommended that the following design aspects be considered when designing high-capacity shear walls:

1. The axial tensile strength along the weak axis of OSB panel is smaller than the shear through-thickness strength. To prevent OSB panel rupture, the OSB panel rupture strength should be the lesser of OSB shear through-thickness strength and the axial strength at 45° of the sheathing's principle direction.

2. Test results show that the panel buckling resistance determined based on the equation in CSA O86 [2] is overly conservative. As a result, the buckling equation should not be used for determining the lateral load capacity of high-capacity shear walls. Until a more accurate equation for determining panel buckling resistance is developed, simplified equation for preventing sheathing buckling in Eurocode 5 should be used.

To prevent plate and stud splitting and separation of studs and plates, it is recommended that the following construction details be used for high-capacity shear walls:

1. One row of nails spaced at 2 inches on center or greater should be used on a single framing member. Results indicate that two rows of nails on a single framing member will likely cause plates or studs to split.

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2. Steel plate washers covering the bottom plate to within 1/4″ of the edges should be installed at the locations of hold-downs. This is to reduce the cross-grain bending due to cantilever effect as shown in Fig 4, therefore preventing the splitting of bottom plate.

3. Metal straps or angle brackets connecting end studs and plates should be used to prevent separation of studs from top or bottom plates. Where possible, metal straps or angle brackets should be installed closer to the sheathing to reduce the eccentricity between the sheathing and metal straps or angle brackets.

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