Innovative Strategies for Light-Frame Mid-Rise Buildings in High-Seismic Regions

Design Example

WoodWorks / WHM Structural Engineers
2025

CANADIAN WOOD COUNCIL
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Acknowledgements

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Executive Summary

BC's west coast has always been a region of high seismic activity, in the current building codes, the seismic forces have increased significantly. Design of mid-rise residential buildings in the Vancouver region and especially Vancouver Island have become more difficult due to these higher seismic demands. Current practice in the design and construction of light wood frame shearwall systems is not able to meet the demand in certain circumstances without the addition of shearwalls, reducing floor area and adding cost.

This guide presents alternative shearwall strategies that can achieve the forces required, especially when combined with lighter weight floor topping strategies, traditionally used to increase acoustic performance and aid in floor leveling. Midply shearwall design is a codified alternative shearwall construction that achieves 50% higher capacities than a traditional shearwall. This is a great alternative for buildings needing higher capacity shearwalls without using research-based strategies or Alternative Solutions, and does not require extra wall length or the doubling up of framing. The second option is the use of double rows of nails at panel sheathing edges, a strategy that has been tested but is not yet included in the building code. This achieves similar capacities to Midply with a construction more similar to a traditional shearwall but requires an Alternative Solution for acceptance by jurisdictions.

Mid+Std and Double Nail Walls are particularly well-suited for offsite construction. Cost estimates provided by industry members suggest that Mid+Std Walls incur a 30% framing cost increase over baseline, while Double Nail Walls incur a 20% increase. Both of these have the potential to be more efficient than doubling corridor wall lines.

High-strength walls combined with lighter floor topping strategies will enable light wood frame construction up to six stories to remain viable in even the highest seismic regions in BC and Canada. These strategies can ensure wood construction remains viable and competitive in a time of increasing costs and housing shortages. Appendix C contains a detailed example to allow designers to follow the analysis and design using these wall systems.

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1 Introduction

The 2020 update to the National Building Code of Canada introduced substantial revisions to the structural requirements for seismic design. The most notable revision was the widespread increase in Design Spectral Acceleration across multiple regions. As a result, seismic loading has increased by more than 50% on average (Popovski, Baheri, Chen, & Ni, 2021).

Multi-family residential construction in high-seismic regions, like Vancouver Island, is expected to face greater challenges, increasing the complexity and cost of light-frame mid-rise buildings. There are a number of strategies available to designers to mitigate these increased risks. The most effective options are summarized below:

1. Site Conditions: Using Shear Wave Velocity measurements in lieu of a traditional Site Classification can reduce the seismic demand by up to 60%.
2. Light-Weight Assemblies: Replacing the non-structural topping with lighter materials can yield significant savings on building weight and thus seismic demand.
3. High-Strength Shearwalls: Adopting novel light-frame shearwall types can be a more cost-effective solution compared to increasing the number of shearwalls in the building.

This document explores innovative lateral design solutions to address emerging risks through a design example of a fictitious residential building in Victoria, BC. The example primarily focuses on the feasibility of high-strength shearwalls, with light-weight toppings considered to maintain the viability of conventional systems only. The feasibility of each design option was determined based on the minimum Shear Wave Velocity (highest base shear) that could be accommodated. All design calculations were completed by WHM Structural Engineers and are included as appendices in this guide.

1.1 Previous Work

The effects of these code changes were examined in a report commissioned by the BC Ministry of Housing titled *"Space and Cost Impact Report"*. A team of practitioners investigated a worst-case scenario for a typical six-storey residential light-frame building located in Victoria, BC. The solution focused on increasing the lateral system strength by providing multiple rows of shearwalls. This approach resulted in an overall building cost increase of approximately 20% compared to current practices. The structural consultant highlighted the importance of re-evaluating current practices to maintain economic viability. They noted that *"Conventional shearwalls are no longer adequate to handle the forces and require new solutions that are not normally used in structural design"* (GHL, Public, WHM, BTY, 2024).

1.2 Site Location and Floor Plan

Victoria is the largest population centre in Canada’s highest seismic risk region. Thus, it was chosen as the location for this design example. The seismic data used corresponds to the increased accelerations found in the BC Building Code 2024. A variety of soil conditions were considered for all possible scenarios.

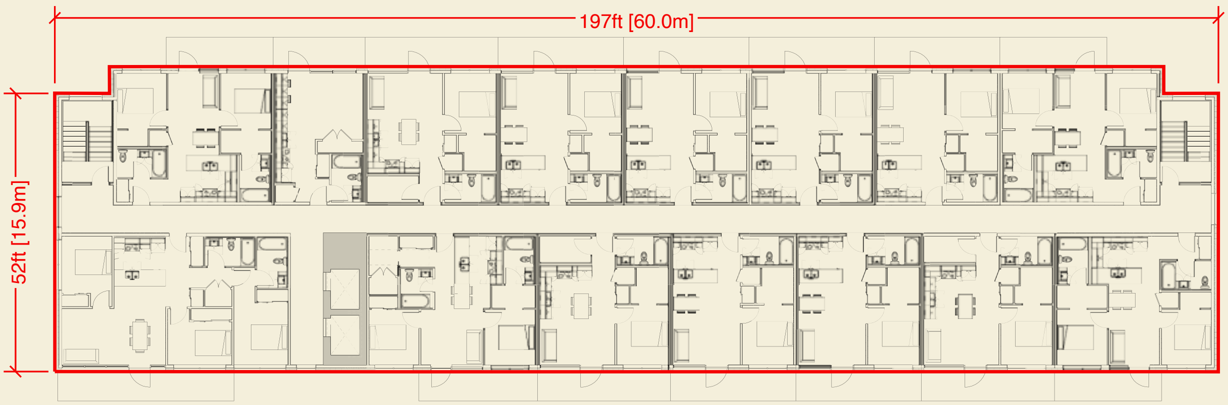

Figure 1: Floor Plan (credit: Public Architecture, courtesy of Ministry of Housing)

The floor plan in Figure 1 is representative of typical multi-residential buildings common in urban and suburban areas in BC. This layout was prepared by Public Architecture as part of the previously-mentioned report. Units of measurement are primarily U.S. customary units as that system is most common for this building type. Metric units are shown in brackets.

2 Seismic Weight

Three floor assemblies were considered for this example. The assembly breakdown can be found in Appendix A with a summary shown in Table 1. A typical asphalt roof assembly with 20psf [1kPa] seismic weight was considered for all cases.

Table 1: Assemblies Considered

Description	Topping Weight
1½" Concrete Topping	19psf [0.91kPa]
1½" Gypcrete Topping	13psf [0.62kPa]
"Dry" (Proprietary acoustic components)	5psf [0.24kPa]

Reduced-weight assemblies are one of the most promising strategies to derisk projects in high-seismic zones. This can be achieved by swapping the non-structural concrete topping with alternatives. In this study we have reviewed gypsum concrete (gypcrete), as well as dry, sheet-based assemblies. There are several proprietary options available that meet or exceed Code requirements, but specification of these assemblies is outside of the scope of this example. More information about cost considerations can be found in Section 4.

Photo Credit: Canada Wood Japan

3 Design Iterations

This example examines various light-frame shearwall strategies for this building type. Three wall construction types have been chosen; they represent both conventional practices and state-of-the-art techniques based on the latest research. The shearwall types used in the study are described in Table 2.

Table 2: Shearwall Design Options

Design Option	Description & Commentary	Code Compliance	Shearwall ID	Nail Diameter, df (mm)	Panel Edge Nail Spacing, s (mm)	No. of Panel Edge Rows	No. of sheathing layers, ns	Sheathing Thickness, ts (mm)	Linear Shear Capacity, vr	
									kN/m	plf
1 – Double-Ply (Conventional)	Standard stud wall with OSB/Ply sheathing with 1-row of nails. Sheathing may be applied from one or both sides of the studs. Indicated (2) where both sides sheathed.	Yes	SW4	3.33	100	1	1	12.5	8.3	569
			SW3	3.33	75	1	1	12.5	10.6	726
			SW2	3.33	50	1	1	12.5	13.7	939
			SW2-H	3.66	50	1	1	15.5	16.8	1,151
			(2)-SW2	3.33	50	1	2	12.5	27.4	1,877
			(2)-SW2-H	3.66	50	1	2	15.5	33.6	2,302
2 - Mid+Std	Midply2.0 variant » Not commonly used in BC » Optional exterior layer of sheathing to increase capacity where indicated*	Yes	Midply	3.66	50	1	1	15.5	33.7	2,309
			Mid+Std*	3.66	50	1	2	15.5	50.5	3,460
3 – Double Nail	Standard stud wall with 2-layers of OSB/Ply sheathing with 2-rows of nails » Similar construction to standard walls with extra nailing » Undergoing testing at the time of this report	No – Only preliminary research values available	(2)-SW1	3.33	50	2	2	12.5	45.9	3,145

Note: 'H' indicates larger nail diameter and thicker sheathing

In this example, studs are made from SPF and wall plates from D. Fir-L, both common species in BC construction. For panels, a variety of types are available for use based on both the published code values and the testing that has been performed to date. This example considers OSB sheathing due to superior panel Shear-Through Thickness rigidity values that control seismic drift.

3.1 Option 1: Double-Ply Walls

Standard double-ply shearwalls for lower levels and single-ply shearwalls for upper levels are a common construction practice. This system was considered as a base case for this example, similar to the *"Space and Cost Impact Report"* mentioned previously. This conventional approach has adequate capacity for a wide range of soil profiles but allowances for lighter toppings must be made for poor soil conditions as shown below.

- Concrete: V_{s30}=560m/s minimum
- Gypcrete: V_{s30}=480m/s minimum
- Dry: V_{s30}=300m/s minimum

Although swapping toppings is an effective strategy, it is not the only one that can be implemented. Non-structural elements can significantly contribute to the seismic loading due to their weight. Below are general recommendations that can help achieve measurable savings:

- » Cladding: Avoid heavy materials such as brick veneers and use light-weight alternatives whenever possible. If required, limit their extent both on plan and elevation as much as possible.
- » Elevator Shafts: Consider wood-based materials such as MLT or CLT. These have the added benefit of reducing or eliminating the need for seismic separation between the shaft and primary structure required for CMU shafts.
- » Firewalls: Similar to above, consider MLT or CLT for weight reduction and speed of construction benefits. Note that Alternative Solutions may be required.
- » Roofs: Avoid green roofs and use lightweight decking instead of concrete pavers whenever possible.

3.2 Option 2: Mid+Std Walls

Mid-Panel (Midply) shearwalls were introduced to CSA O86 in 2015 but have seen limited use in BC due to few benefits over double-ply walls. This approach was used as the basis for the higher-strength variant used in this example. We have considered a Midply shearwall with an additional layer of plywood on the opposite face for improved performance (referred to as Mid+Std).

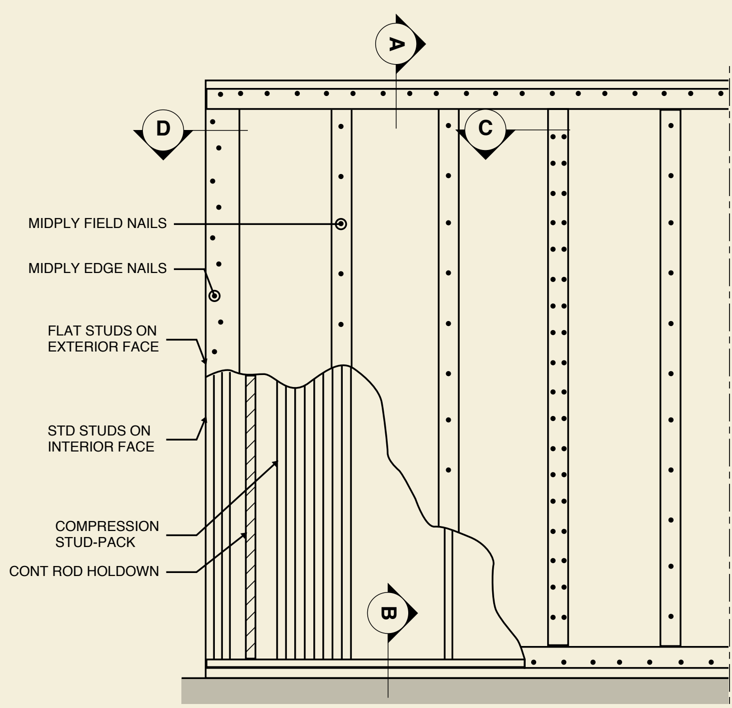

Figure 2: Mid+Std Wall Elevation

The construction of Mid+Std Shearwalls is similar to traditional walls, as shown in Figure 2. The main difference is that sheathing nails are driven through an additional member (2x on flat) to develop double shear in the nails. This increases the wall capacity with a slightly more complex load path. Specific details are required to ensure the wall behaves as designed, as covered in Section 3.2.3.

Adding this exterior layer of plywood increases the capacity beyond traditional approaches. Figure 3 shows a plan detail of this combined assembly.

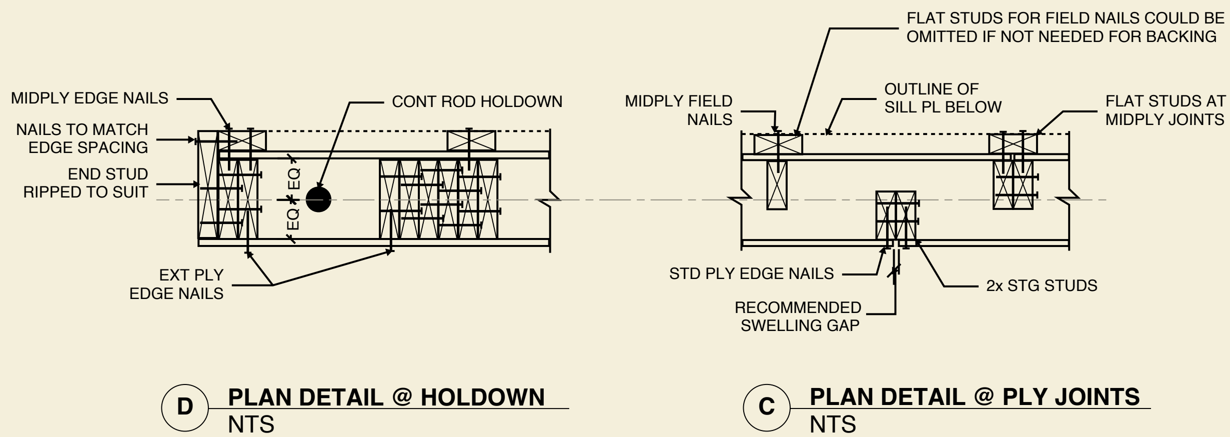

Figure 3: Mid+Std Plan Detail

The prevailing standard for high-strength walls is a doubled wall assembly, which increases overall wall thickness by approximately 7 inches. By contrast, a Mid+Std Wall requires only an additional 1½ inches due to the inclusion of an extra framing layer. This method results in a comparatively modest increase in wall thickness, which is essential for preserving the project's feasibility.

3.2.1 Code Compliance

A Mid+Std shearwall is a fully code-compliant solution. Fasteners in double shear are recognized in CSA O86 Clause 11.3.2.3, with design values being included in the Wood Design Manual derived based on a Mechanics Approach (Canadian Wood Council, 2020). Furthermore, Clause 11.4.3.2 allows for linear strength addition of wood panel systems on opposite sides of a stud wall. This approach is similar to combining wood- and gypsum-based panels on the same wall considering strength and stiffness differences. It should be noted that although Midply shearwalls have undergone thorough testing (Ni & Chen, 2021), Mid+Std walls have not been tested.

3.2.2 Design Summary

Mid+Std Walls in this design example are adequate for use within all site conditions by varying the toppings. For the building studied, they can be used to the minimum V_{s30} values shown below:

- Concrete: V_{s30}=300m/s
- Gypcrete: All values
- Dry: All values

Detailed design calculations can be found in Appendix C. A summary of final design results at V_{s30}=300m/s with concrete topping is presented in Table 3 below.

Table 3: Mid+Std Design Summary

	Short-Direction (Party Walls)			Long-Direction (Corridor Walls)		
Level	Shearwall ID	End Post	Tie-Down	Shearwall ID	End Post	Tie-Down
6	SW2	2-2x6	0.75" ø A307	SW2-H	4-2x6	0.75" ø A307
5	(2)-SW2	2-2x6	0.75" ø A307	(2)-SW2-H	4-2x6	1.25" ø A307
4	(2)-SW2	4-2x6	1.25" ø A307	Mid+Std	8-2x6	1.75" ø A307
3	(2)-SW2-H	6-2x6	1.5" ø A307	Mid+Std	8-2x6	1.75" ø A307
2	Mid+Std	8-2x6	1.75" ø A307	Mid+Std	14-2x6	2" ø B7
1	Mid+Std	10-2x6	2" ø A307	Mid+Std	14-2x6	2" ø B7

Note:
1. End Post stud count is sum of end studs on either side of tie-down rod, i.e. total number on each wall end
2. (2) indicates sheathing on both sides of stud wall
3. Mid+Std walls in short direction could be replaced by (3)-SW2-H, i.e. three layers of sheathing on party wall consisting of two walls of 2x4 studs

3.2.3 Construction Considerations

Due to their high capacity, standard connection details need to be adjusted for Mid+Std shearwalls. This construction will be a change for many builders across BC. However, the significant increase in capacity combined with code-compliant design values make this a viable option when dealing with very high seismic forces. Concept details are provided in Appendix D. Designers are encouraged to use them as a base to develop their own project-specific approaches.

3.2.3.1 Top and Bottom Connections

A suitable option is presented in Figure 4 using common nails and framing clips. The vertical sill plate allows the bottom row of edge nails to act in double shear. To complete this load path, the vertical plate should be connected to the blocking below. This connection can be made by using a double horizontal sill plate, with the lower plate being oversized to accommodate the vertical portion. A similar method applies to the top plate, which can include two continuous plates. The larger plate must be ripped to keep the overall wall thickness consistent.

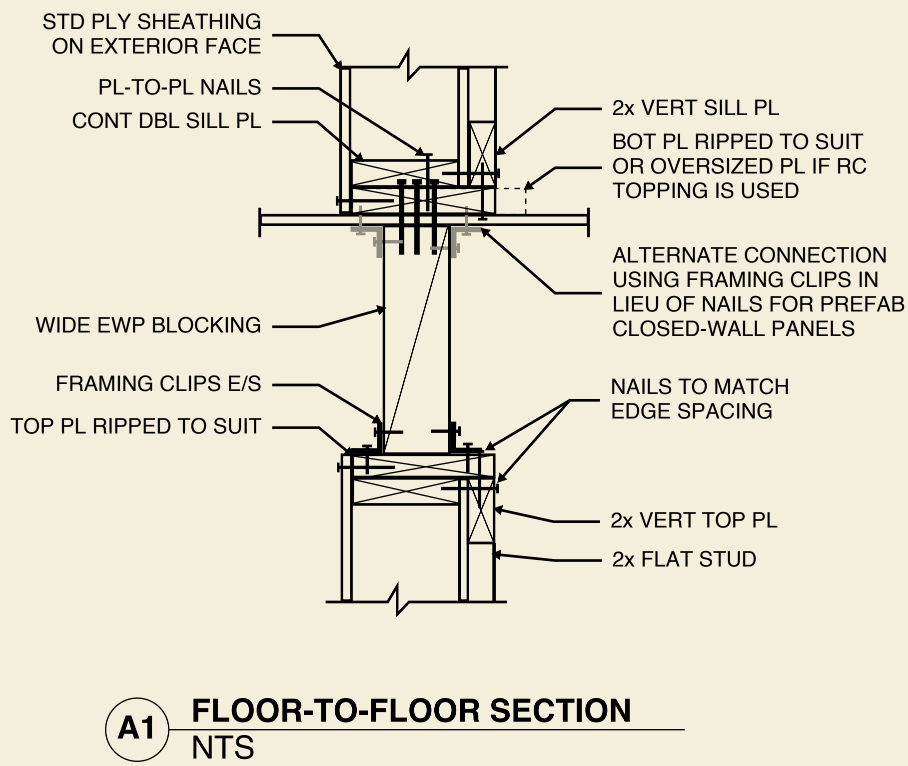

Figure 4: Mid+Std Floor-Floor Connections

When designing for framing clips, it is important to account for the geometry of the connectors, as the spacing is typically tighter than standard. Selecting narrower clips can help reduce the potential for overlapping when placement is close. Self-tapping fully threaded screws are also a suitable alternative for faster installation as shown in Appendix D. Driving these screws in two rows at a 45-degree angle, with one screw in tension and the other in compression, can provide a stiff and strong connection. Large end stud packs limit sill plate connection length; consider alternate underside connections if necessary.

3.2.3.2 Concrete Connections

The same sill plate connection can be adapted at the bottom of the wall fastening to the slab or foundation. Figure 5 shows a typical concrete connection detail.

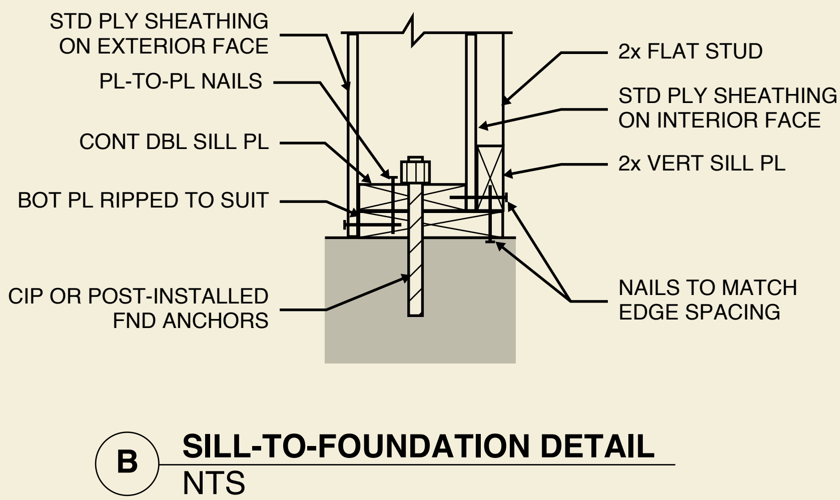

Figure 5: Mid+Std Sill-Foundation Connection

Similar to the top plate detail, the larger sill plate would need to be ripped unless it can be concealed within the floor assembly. Traditional cast-in-place or post-installed anchors can be designed following current practices. Designers must ensure that the tighter spacing does not conflict with the stud spacing. They must also consider that end-posts reduce the length available for the connection. A detail with anchors below the stud pack can also be developed as an option. This can be a considerable loss for shorter walls with large stud packs.

3.2.3.3 Prefabrication Opportunities

In a field-built scenario, it would be possible to prebuild the 'L' portion for the sill plate and fasten it to the subfloor before building the wall itself, similar to typical construction techniques. Figure 6 shows a possible erection sequence following this approach.

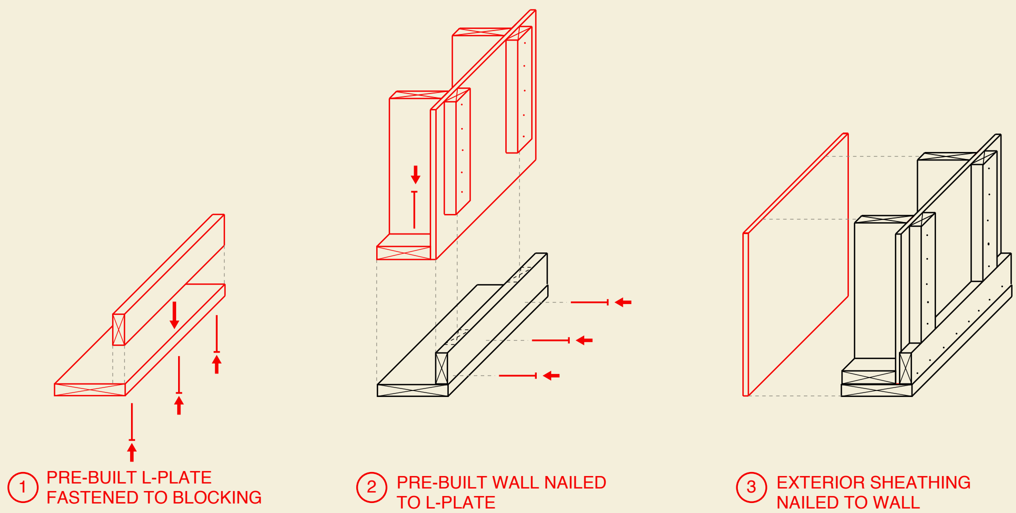

Figure 6: Mid+Std Erection Sequence

Framing the outer layer of 2x material could be done either before or after wall installation. As these are not bearing vertical loading, the ends of members do not need to have a tight fit. However, the placement is critical to ensure proper nailing to the narrow side of the load-bearing stud. Top plates could be built more traditionally, with the wider upper top plate fastened to the lower top plate and vertical outer layer either before or after wall installation.

In a factory-built scenario, it would be possible use a similar sill detail with the L-plate shipped loose to site. Alternatively, the studs could be fastened directly to the wider plate with blocking between studs for the sheathing nails. With this construction, the wall would go to site with an effective double sill plate, requiring the connection to the subfloor/wall below to either use self-tapping screws, or framing clips fastened to the sill plate from the bottom.

3.3 Option 3: Double Nail Walls

Double Nail Walls are an innovative concept that relies on two rows of edge nails to increase the shear capacity of the wall. Only preliminary testing has been completed at the time of this report, so it is considered to be a work-in-progress solution (refer to Section 3.3.1). The initial tests were conducted by FPInnovations based on the assembly shown in Figure 7.

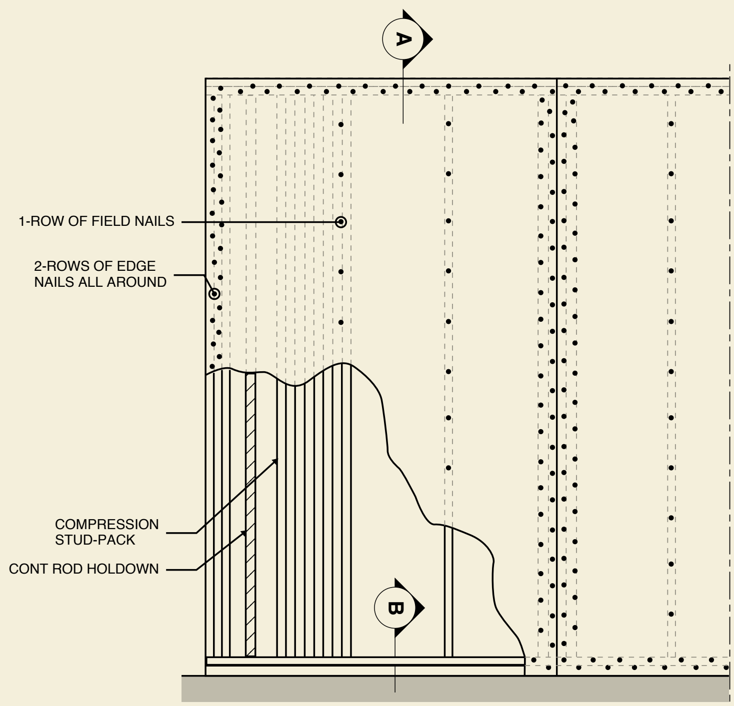

Figure 7: Double Nail Wall Elevation

Two rows of nails are used because the tight spacing in a single row is not feasible due to the risk of splitting the sheathing edge and framing. Thus, this wall requires additional framing to allow for additional nails. The testing used vertically-oriented panels to eliminate the need for mid-height blocking. Vertical panel joints would require four studs, with the nailing between them designed to transfer the panel joint shear load.

3.3.1 Code Compliance

The design values used for this example were derived with assistance from FPInnovations based on their preliminary testing. Results for single- and double-sided shearwalls can be found in their April 2025 – Info Note 1 (Tung & Ni, 2025) and May 2025 – Info Note 2 (Tung & Ni, 2025), respectively. Future research will focus on developing design values and ductility and overstrength equivalency to conventional shearwalls per ASTM D7989.

Although more work is needed before building code adoption, designers will have the option of following these recommendations through an Alternative Solution. Technical assistance from FPInnovations and WoodWorks will be available to navigate project-specific requirements and to provide design values based on testing results. The testing on this wall assembly used ½" [12.5mm] thick sheathing and 8d nails, so the specifications will follow this same assembly. Further testing would be required to review the use of thicker panel sheathing products.

3.3.2 Design Summary

Double Nail Walls in this design example are adequate for use within all soil conditions by varying the toppings, with similar capacities to Mid+Std Walls. For the building studied, they can be used to the minimum V_{s30} values shown below:

- Concrete: V_{s30}=300m/s
- Gypcrete: All values
- Dry: All values

A summary of final design at V_{s30}=300m/s with concrete topping is presented in Table 4.

Table 4: Double Nail Design Summary

	Short-Direction (Party Walls)			Long-Direction (Corridor Walls)		
Level	Shearwall ID	End Post	Tie-Down	Shearwall ID	End Post	Tie-Down
6	SW2	2-2x6	0.75" ø A307	SW2-H	4-2x6	0.75" ø A307
5	(2)-SW2	2-2x6	0.75" ø A307	(2)-SW2-H	4-2x6	1.25" ø A307
4	(2)-SW2	4-2x6	1.25" ø A307	(2)-SW1	8-2x6	1.75" ø A307
3	(2)-SW2-H	6-2x6	1.5" ø A307	(2)-SW1	8-2x6	1.75" ø A307
2	(2)-SW1	8-2x6	1.75" ø A307	(2)-SW1	14-2x6	2" ø B7
1	(2)-SW1	10-2x6	2" ø A307	(2)-SW1	14-2x6	2" ø B7

Note:
1. End Post stud count is sum of end studs on either side of tie-down rod, i.e total number on each wall end
2. (2) indicates sheathing on both sides of stud wall

3.3.3 Construction Considerations

The connections between wall and floor assemblies for a Double Nail wall are similar to a standard connection while having the capacity to transfer much higher forces. Concept details are provided in Appendix D.

3.3.3.1 Top and Bottom Connections

Nails through the sill plate into the blocking below may be adequate but will require wider blocking (3.5"/89mm minimum). Figure 8 shows this detail.

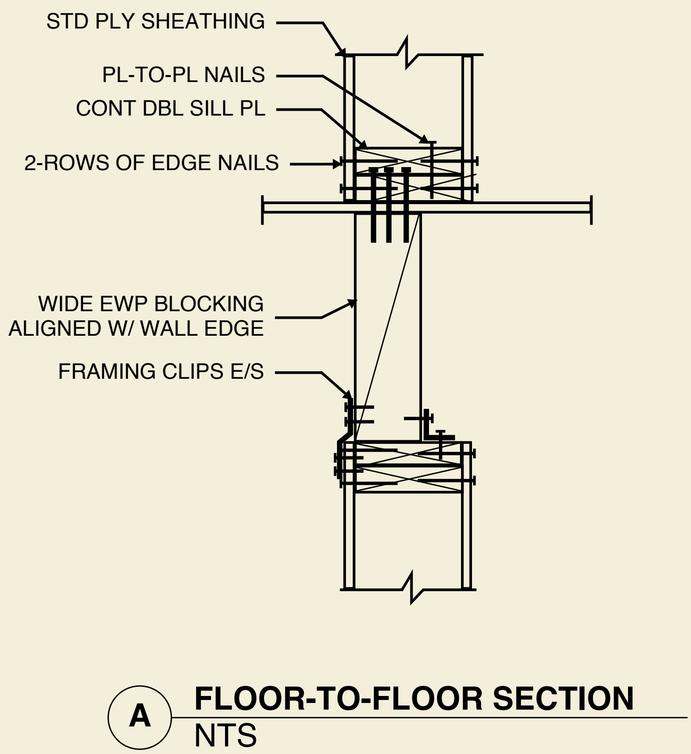

Figure 8: Double-Nail Floor-Floor Connection

As there are two sill plates, the first plate will need to be nailed down first before fastening the rest of the wall to this plate. This would work for both field, and factory-built walls but would require the lower rows of sheathing fasteners to be installed while the wall is standing. Alternatively, framing clips or screws could be used, similar to the details for Mid+Std walls. See the comments in Section 3.2.3 when detailing with these fastener types.

3.3.3.2 Concrete Connections

For foundation connections, standard details of anchors into concrete can be used, similar to traditional construction techniques.

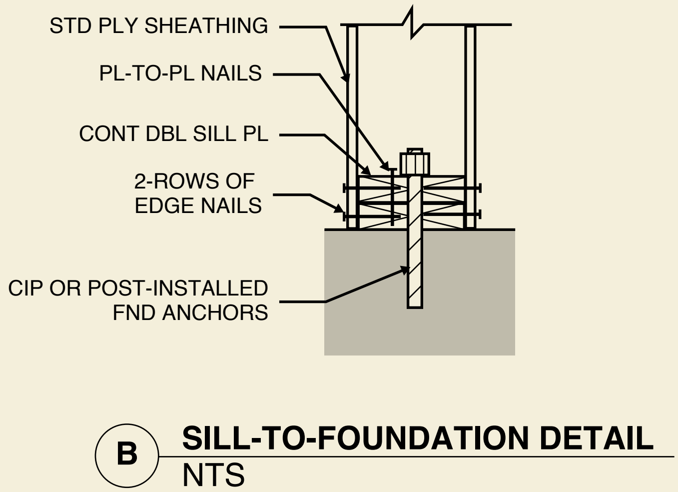

Figure 9: Double-Nail Sill-Foundation Connection

Tighter spacing of fasteners to support the higher loads will be required. See the comments in Section 3.2.3 for additional considerations.

3.3.3.3 Prefabrication Opportunities

The construction process and detailing for Double Nail Walls are similar to Double-Ply shearwalls. The main difference is that both top plates and sill plates will require nailing, which may be more difficult to coordinate with the connection to the subfloor. Review of the sequencing to ensure that fasteners can be installed easily is critical for both site and factory construction.

4 Cost Considerations

Dry toppings can reduce seismic loads to levels suitable for a single row of walls along corridor lines. Material costs are similar, but dry toppings generally involve additional labour due to their panelized format. Industry partners estimate relative costs for lightweight alternatives at approximately x1.0-3.0 compared to traditional wet toppings. These costs should be considered alongside the expense of adding shearwall lines and accounting for other variables such as floor framing (sawn lumber vs. I-Joists) and acoustic performance.

If heavier toppings are required with minimal wall thickness increase, high-strength wall options might be more appropriate. Mid+Std and Double Nail Walls are most suitable for off-site approaches due to the increased framing complexity. Ron Anderson and Sons Ltd was engaged to provide general costing for prefabricating all options. The values below were calculated for a single wall segment based on the worst-case design scenario for each option. One layer of sheathing was assumed to be field-installed, following current practices. No further considerations for connections or dry toppings were made.

1. Double-Ply Walls: Baseline
2. Mid+Std Walls: x1.3 increase over baseline
3. Double Nail Walls: x1.2 increase over baseline

They observed that most variances were due to extra materials, with minimal impact from additional labour. This breakdown would vary between suppliers depending on their level of automation. They also noted that there were no significant obstacles to the prefabrication of the high-strength options compared to a standard system. These values should be compared with the standard double corridor wall (x2.0). Designers can explore various walls and topping options to find the most cost-effective solution.

5 Summary

Increasing seismic loads brings challenges that are sometimes greater than traditional solutions can solve. This example provides alternative strategies that can be used when conventional systems do not have the capacity required. The findings show that both Mid+Std walls and Double Nail walls can provide the extra capacity required to maintain the feasibility of light wood frame construction even for poor soil conditions.

1. Double-Ply Walls: When designing in locations with V_{s30} > 560m/s, traditional walls will likely be viable; however, with poorer soil conditions lighter weight toppings may be required. Locations with V_{s30} values < 300m/s will likely not be viable with this shearwall assembly.
2. Mid+Std Walls: Where the design team and/or the jurisdiction does not want to pursue an alternative solution, a Mid+Std assembly may be the best choice. This allows a code-compliant design with the ability to choose the sheathing size and type that works best for the project. In the example provided, sites with V_{s30} > 300m/s will be viable with normal weight topping, and V_{s30} < 300m/s are possible with lighter weight alternatives.
3. Double Nail Walls: Where the design team wants to keep a wall assembly that more closely resembles a traditional approach, the double nail system can increase the capacity of a double-sided wall assembly; however, the team will need to prepare and present an alternative solution to the jurisdiction. In the example provided, sites with V_{s30} > 300m/s will be viable with normal weight topping, and V_{s30} < 300m/s are possible with lighter weight alternatives.

Reducing building weight will always be a benefit when seeking to reduce seismic loads. The use of heavy cladding systems, roof assemblies and floor toppings all contribute to increased forces. Designers, Contractors, and Owners should review the alternatives to heavy assemblies like normal weight concrete topping when faced with high seismic demand.

This guide and its calculation example should support the design of high-capacity LWF lateral systems in even the highest seismic regions. If you have further questions, please reach out to the staff at WoodWorks.

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Photo Credit: Canada Wood Japan

Appendix A: Structural Scheme Description

A1 Structural Scheme Description

The building is assumed to be a six-storey wood-frame structure with a rectangular floor plate over a suspended slab or foundation. The analysis of a rectangular floor plate can be extended to more complex L-shaped or C-shaped configurations with some modifications. The building has a typical floor-to-floor height of approximately 9'-0", giving clear heights in units of 8'.

A1.1 Gravity Scheme

Framing for a typical level is assumed to use floor joists spanning from party wall to party wall, with interior load-bearing walls parallel to party walls as supports. The assumed joist arrangement is shown in Figure A1 below.

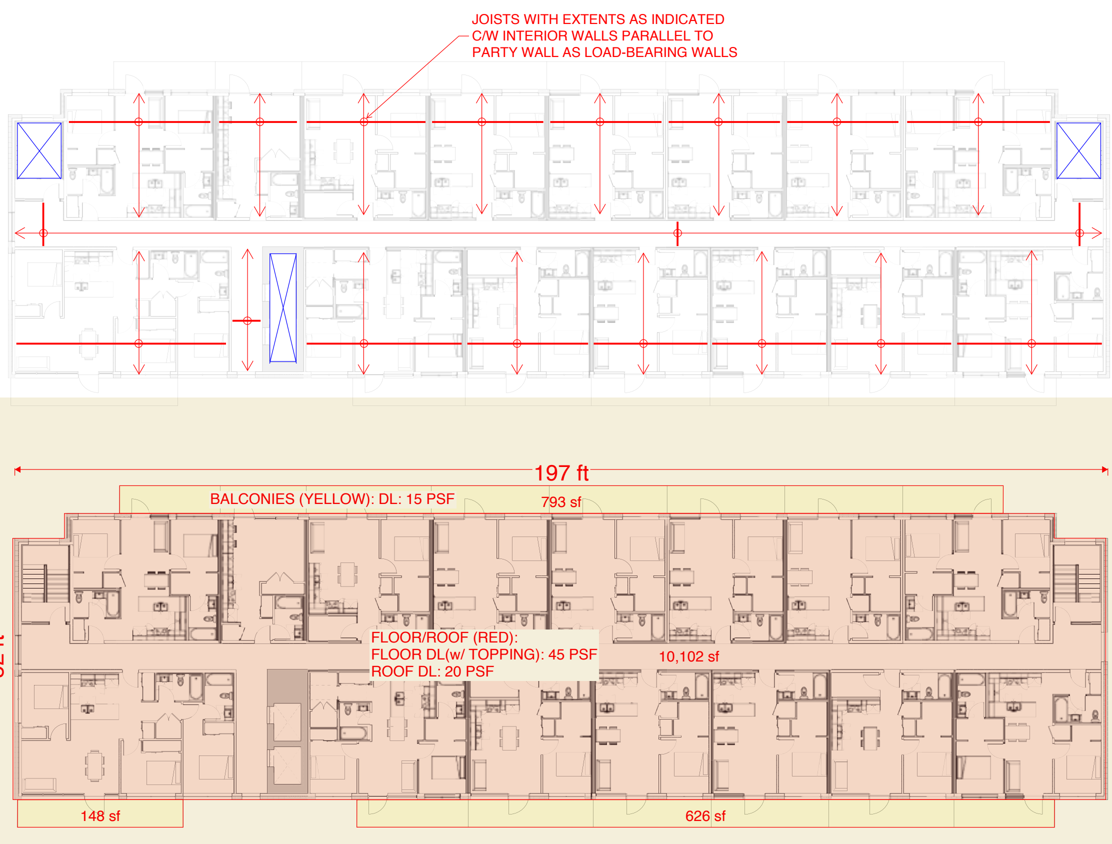


Figure A1: Floor plan showing framing (top) and dead loads assuming concrete topping (bottom)

The loading plan of the building, given in Figure A1, is for a typical floor buildup that includes a 1½" non-structural concrete topping. Furthermore, use of gypcrete and a lighter dry acoustic assembly (referred to as Dry Topping throughout) were also considered as a mitigation strategy to reduce the seismic loading on the building.

A1.2 Lateral Scheme

The shearwall layout consists of corridors and demising walls between units. This layout provides a central and approximately symmetrical layout, reducing torsion on the building and keeping the critical structural behaviour localized, making it easier to ensure quality control.

A1.2.1 Long-Direction Shearwalls

The structural performance of these buildings is often controlled by the corridor walls. The layout for a typical level is shown in Figure A2. The use of additional walls (exterior or in-unit) parallel to the corridor is often difficult due to short wall lengths decreasing effectiveness. The use of these walls is not included in this analysis.

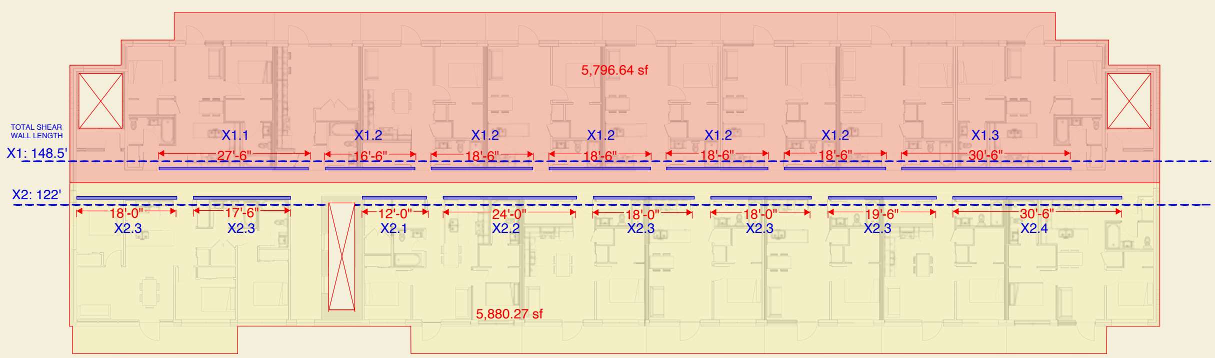


Figure A2: Floor plan showing corridor shearwalls

The analysis considers two primary wall lines: X1 and X2. Segments within each line are connected by continuous elements such as a spliced double top plate. Therefore, they are assumed to deflect uniformly. For discontinuities similar to the front of the elevator at X2, a continuous drag strut (collector) lapped with shearwalls on either side is assumed to act as a collector. The floor diaphragm areas directly engaged by a shearwall line are highlighted in the red for the North line, and yellow for the South line.

A1.2.2 Short-Direction Shearwalls

The shearwalls between units for a typical level are shown in Figure A3. Walls are aligned along unit party walls highlighted in blue.

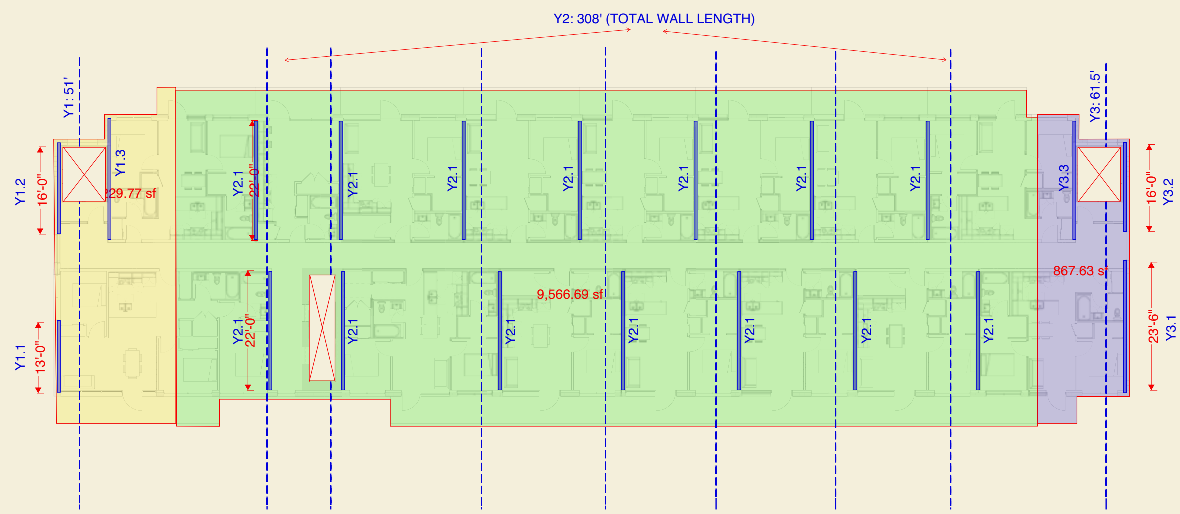


Figure A3: Floor plan showing demising walls used as shearwalls

The analysis is divided into three wall types depending on the expected behaviour: Y1, Y2, and Y3. Group 1 walls have similar length and diaphragm tributary areas and hence would experience similar forces and deflections (Y2.1). Group 2 walls are connected by a continuous element such as double spliced top plates on exterior walls (Y1.1 and Y1.2). Group 3 walls can be forced to share loads and deflect similarly by using drag struts and diaphragm sheathing with other walls in the same line (Y1.3 with other Y1 walls). As with the X-direction walls, diaphragm areas directly engaged by each shearwall line are highlighted.

Appendix B: Dead Load and Seismic Load Calculations

B1. Assemblies

Roof Assembly:

Typical Flat Roof – Tar and Gravel Roofing:		w_{DL}	
		psf	**kPa**
4 ply tar and gravel (modified bitumen)	=	6	0.29
Drywall ceiling (16mm)	=	3	0.14
Mechanical allowance	=	3	0.14
Misc. allowance/back-framing for ceiling	=	2	0.10
Superimposed Dead Load	=	**14**	**0.67**
½" plywood sheathing	=	1.5	0.07
2x tapered sleepers @ 24" o/c	=	2	0.10
I-joists or 2x @ 24" o/c	=	2	0.10
Wood beam allowance	=	neglect	
Total Dead Load	=	**20**	**0.96**

Floor Assemblies:

Typical Floor with 1-1/2" concrete topping		w_{DL}	
		psf	**kPa**
Partitions (see general section for notes)	=	13	0.62
Flooring allowance	=	2	0.10
1-1/2" Concrete Topping @ 150pcf (23.5kN/cu-m)	=	19	0.91
2 layers Type X drywall (32mm total)	=	5	0.24
Mechanical allowance	=	2	0.10
Wood beam allowance	=	neglect	
Superimposed Dead Load	**=**	**41**	**1.96**
5/8"plywood sheathing	=	2	0.10
I-joists or 2x @ 16" o/c	=	2	0.10
Total Dead Load	**=**	**45**	**2.15**

Typical Floor with gypcrete topping		w_{DL}	
		psf	**kPa**
Partitions (see general section for notes)	=	13	0.62
Flooring allowance	=	2	0.10
1-1/2" gypcrete topping	=	13	0.62
2 layers Type X drywall (32mm total)	=	5	0.24
Mechanical allowance	=	2	0.10
Wood beam allowance	=	neglect	
Superimposed Dead Load	**=**	**35**	**1.68**
5/8"plywood sheathing	=	2	0.10
I-joists or 2x @ 16" o/c	=	2	0.10
Total Dead Load	**=**	**39**	**1.87**

Typical Floor with dry topping		w_{DL}	
		psf	**kPa**
Partitions (see general section for notes)	=	13	0.62
Flooring allowance	=	2	0.10
Dry topping	=	5	0.24
2 layers Type X drywall (32mm total)	=	5	0.24
Mechanical allowance	=	2	0.10
Wood beam allowance	=	neglect	
Superimposed Dead Load	**=**	**27**	**1.29**
5/8"plywood sheathing	=	2	0.10
I-joists or 2x @ 16" o/c	=	2	0.10
Total Dead Load	**=**	**31**	**1.48**

B2. Building Geometry and Design Properties

Summary of the building geometry:

Level	Diaphragm Area			Diaphragm Dim.		Level Height	Extra Seismic Weight	Elevation
i	$A_{roof,i}$	$A_{balcony,i}$	$A_{floor,i}$	$B_{D,i}$	$L_{D,i}$	H_i	$W_{Extra,i}$	Z_i
	ft^2	ft^2	ft^2	ft	ft	ft	kip	ft
6	10102	0	0	197	52	9	0	54
5	0	1567	10102	197	52	9	0	45

4	0	1567	10102	197	52	9	0	36
3	0	1567	10102	197	52	9	0	27
2	0	1567	10102	197	52	9	0	18
1	0	1567	10102	197	52	9	0	9

Design Properties:
Location = Victoria, BC
Site Class = D
Vs30 (m/s) = 300
Coordinates (Lat, Lon) = 48.42832, -123.36495
Building Info NBC 2020 (Vs30 = 300 m/s):

Sa(0.2): = 1.860
Sa(0.5): = 1.820
Sa(1.0): = 1.090
Sa(2.0): = 0.646
Sa(5.0): = 0.167
Sa(10.0): = 0.060
PGA: = 0.817
PGV: = 1.040

Importance Category = Normal
Rd = 3.0
Ro = 1.7

B3. Design Loads

Although three different floor assemblies are presented in the main body of report, the detailed calculations are only provided for 1-1/2” normal weight concrete topping.

$$W_{Roof} = 20psf$$

$$W_{Balcony} = 15psf$$

$$W_{Floor}(incl\ partitions) = 45psf$$

$$S_s = 1.1kPa = 23.0psf$$

$$S_r = 0.2kPa = 4.2psf$$

$$SL = 1.1kPa = 22.6psf$$

$$25\%SL = 0.3kPa = 5.6psf$$

$$W_{Roof+Snow} = 26psf$$

B4. Building Weight

Level	Total Diaphragm Area	Level Weight	Elevation	Lateral Force Dist.	
i	$A_{T,i}$	W_i	Z_i	$W_i Z_i$	$W_i Z_i / \sum W_i Z_i$
	ft^2	kip	ft	kip-ft	
6	10102	310	54	16713	20%
5	11669	487	45	21912	27%
4	11669	487	36	17529	21%
3	11669	487	27	13147	16%
2	11669	487	18	8765	11%
1	11669	487	9	4382	5%
Total		2744		82448	100%

B5. Equivalent Static Force

$$T_a = T_{code} = 0.41\ , S_{(Ta)} = 1.83$$

Minimum Lateral Force

V	=	$S(T_a)\ M_v I_E W / (R_d R_o)$	=	0.359	W			

Minimum for walls, coupled walls and wall-frame systems

V	=	$S(4.0)\ M_v I_E W / (R_d R_o)$	=	0.064	W			

Minimum for walls, coupled walls and wall-frame systems

V	=	$S(2.0)\ M_v I_E W / (R_d R_o)$	=	0.126	W			

Maximum for sites other than Class F and $R_d \geq 1.5$

V	=	$2/3\ S(0.2)\ I_E W / (R_d R_o)$	=	0.243	W			
	and	$S(0.5)\ I_E W / (R_d R_o)$	=	0.357	W	**Governing Case**		

Lateral Force

Governing Case			=	**0.357**	W	=	**979**	kip

Note: Mv and F(t) are not considered for six-storey building.

B6. Load Distribution

Seismic Force Distribution per level:

Level	Elevation	Total Diaphragm Area	Lateral Force	Storey Shear	**Storey Shear per Unit Area**
i	H_i	$A_{T,i}$	$F_{D,i}$	$V_{D,i} = \Sigma \mathrm{F}_{D,i}$	$\boldsymbol{v_{D,i}}$
	ft	ft^2	kip	kip	**psf**
6	54	10102	198.5	198.5	**19.7**
5	45	11669	260.3	458.8	**22.3**
4	36	11669	208.2	667	**17.8**
3	27	11669	156.2	823.1	**13.4**
2	18	11669	104.1	927.2	**8.9**
1	9	11669	52.1	979.3	**4.5**

$v_{D,i}$ is storey shear per unit area of diaphragm. This value informs load distribution for the shear wall design in next section.

Appendix C: Detailed Design Calculations for Option 2

C1. Overview and Key Plans

Option 2 refers to the mid+std shear wall option in the main body of the text. This consists of a mid-ply sheathing on one side and a standard exterior sheathing on the other side of the wall studs. The design properties of mid+std wall assembly and the standard assemblies considered for upper levels are provided again here for easy reference.

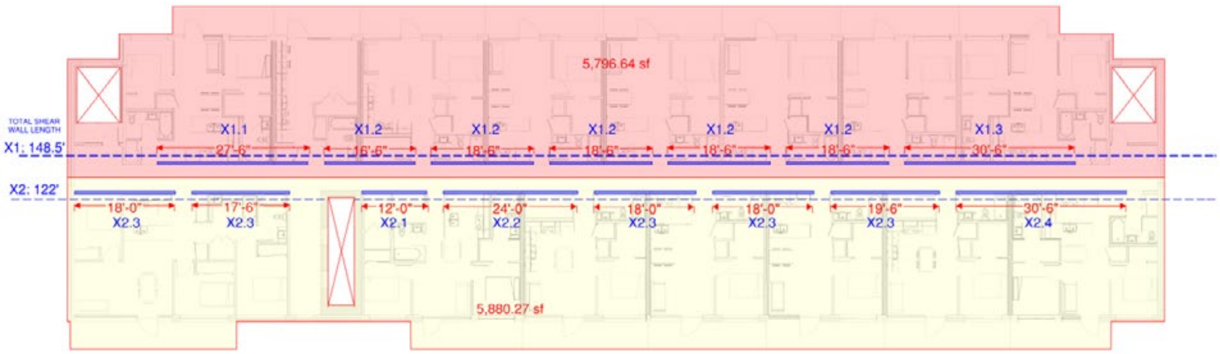
Shear Wall type	Nail Diameter	Panel Edge Nail Spacing	No. of sheathings	Sheathing thickness	Shear Capacity		Shear-Through-Thickness Rigidity of Sheathing
	d_f	s	n_s	t_s	v_r		B_v (OSB)
	mm	mm		mm	kN/m	plf	N/mm
SW4	3.33	100	1	12.5	8.3	569	11000
SW3	3.33	75	1	12.5	10.6	726	11000
SW2	3.33	50	1	12.5	13.7	939	11000
SW2-H	3.66	50	1	15.5	16.8	1151	12000
(2)-SW2	3.33	50	2	12.5	27.4	1877	11000
(2)-SW2-H	3.66	50	2	15.5	33.6	2302	12000
MidPly	3.66	50	1	15.5	33.7	2309	12000
Mid+Std	3.66	50	2	15.5	50.5	3460	12000

The overall calculation steps followed in this design example:

1. Strength calculations at code-based time-period to obtain trial selection of sheathing, tie-down and end post sizes for all walls.
2. Iterative deflection calculations at rational time-period for the trial assemblies. Force re-distribution per wall stiffnesses based on CSA-O86 and 6 storey wood design guide by EGBC.
3. Revised sheathing, tie-down and end post selection based on rational period and re-distributed forces.

The key plans of the building, discussed in detail in appendix A, are provided here for quick reference.

Corridor Walls (X-Direction):



Party Walls (Y-Direction):

C2. Wall Sheathing

This section estimates the wall sheathing requirements for each shear wall line. Appendix A of the report provides key plans for both X direction walls and Y direction walls showing the discrete zones of the floor diaphragm supported by specific wall lines. For level *i*, the total shear force in a wall line is calculated and then converted into force per unit length:

$$V_{T,i} = A_{Trib,i} * v_{D,i} + V_{T,i+1}$$

$$v_i = V_{T,i} / \mathrm{L}_{T,i}$$

Wall Sheathing Utilization:

$$Utilization = \frac{v_i}{v_r} * 100\%$$

where,

$V_{T,i}$ is total shear force in a shear wall line at level i,

$A_{Trib,i}$ is diaphragm tributary area associated with shear wall line at level i,

v_i is shear force per unit wall length for level i,

$L_{T,i}$ is total length of shear wall line for level i.

The following tables summarize the sheathing requirements and utilization for the X- and Y-direction walls, respectively. The appropriate shear wall type (e.g., SW3, (2)SW2-H, Mid+Std) is selected based on the required capacity and utilization efficiency.

Corridor Walls (X-Direction)										
Wall Line	Level	Wall Line Length	Wall Line Load Above	Diaphragm Tributary Area	% Area	Wall Line Shear Demand	Shear Demand per Unit Length	**SW Estimate**	Shear Capacity	Utilization
	i	$L_{T,i}$	$V_{T,i+1}$	$A_{Trib,i}$		$V_{T,i}$	v_i	**SW**	v_r	$\frac{v_i}{v_r}$
		ft	kip	ft^2		kip	plf		plf	%age
X1	6	148.5	0	5051	50%	99.3	668	**SW3**	726	92%
X2	6	157	0	5051	50%	99.3	632	**SW3**	726	87%
X1	5	148.5	99.3	5796	50%	228.5	1539	**(2)-SW2**	1876	82%
X2	5	157	99.3	5879	50%	230.4	1467	**(2)-SW2**	1876	78%
X1	4	148.5	228.5	5796	50%	331.9	2235	**(2)-SW2-H**	2302	97%
X2	4	157	230.4	5879	50%	335.3	2136	**(2)-SW2-H**	2302	93%
X1	3	148.5	331.9	5796	50%	409.5	2758	**Mid+Std**	3459	80%
X2	3	157	335.3	5879	50%	413.9	2637	**Mid+Std**	3459	76%
X1	2	148.5	409.5	5796	50%	461.2	3106	**Mid+Std**	3459	90%
X2	2	157	413.9	5879	50%	466.4	2971	**Mid+Std**	3459	86%
X1	1	148.5	461.2	5796	50%	487.1	3280	**Mid+Std**	3459	95%
X2	1	157	466.4	5879	50%	492.6	3138	**Mid+Std**	3459	91%

Party Walls (Y-Direction)										
Wall Line	Level	Wall Line Length	Wall Line Load Above	Diaphragm Tributary Area	% Area	Wall Line Shear Demand	Shear Demand per Unit Length	**SW Estimate**	Shear Capacity	Utilization
	i	$L_{T,i}$	$V_{T,i+1}$	$A_{Trib,i}$		$V_{T,i}$	v_i	**SW**	v_r	$\frac{v_i}{v_r}$
		ft	kip	ft^2		kip	plf		plf	%age
Y1	6	50	0	1111	11%	21.8	437	**SW4**	568	77%
Y2	6	308	0	8284	82%	162.8	528	**SW4**	568	93%
Y3	6	50	0	707	7%	13.9	278	**SW4**	568	49%

Y1	5	50	21.8	1230	11%	49.3	985	**SW2-H**	1151	86%
Y2	5	308	162.8	9567	82%	376.2	1221	**(2)-SW2**	1876	65%
Y3	5	50	13.9	867	7%	33.2	665	**SW3**	726	92%
Y1	4	50	49.3	1230	11%	71.2	1424	**(2)-SW2**	1876	76%
Y2	4	308	376.2	9567	82%	546.9	1776	**(2)-SW2**	1876	95%
Y3	4	50	33.2	867	7%	48.7	974	**SW2-H**	1151	85%
Y1	3	50	71.2	1230	11%	87.7	1754	**(2)-SW2**	1876	93%
Y2	3	308	546.9	9567	82%	674.9	2191	**(2)-SW2-H**	2302	95%
Y3	3	50	48.7	867	7%	60.3	1206	**(2)-SW2**	1876	64%
Y1	2	50	87.7	1230	11%	98.6	1973	**(2)-SW2-H**	2302	86%
Y2	2	308	674.9	9567	82%	760.2	2468	**Mid+Std**	3459	71%
Y3	2	50	60.3	867	7%	68	1361	**(2)-SW2**	1876	73%
Y1	1	50	98.6	1230	11%	104.1	2083	**(2)-SW2-H**	2302	90%
Y2	1	308	760.2	9567	82%	802.9	2607	**Mid+Std**	3459	75%
Y3	1	50	68	867	7%	71.9	1438	**(2)-SW2**	1876	77%

C3. Tie-Downs and End Posts

The following assumptions are made for calculating tie-down and end post sizes:

1. Each end post consists of two posts (one on each side of the tie-down rod), with a clear spacing of 150 mm (6"). (See figure below)
2. Studs in the posts are asymmetrically distributed. A maximum of three studs are placed at the outer end, and the remainder on the interior side.
3. The moment arm is taken as the distance between the tie-down rods at either end of the wall.
4. Floor joist + sheathing depth is assumed to be 250 mm (10") at all levels.
5. Two top and bottom plates are included per wall.
6. Studs: SPF No.1/No.2; Plates: D. Fir No.2.

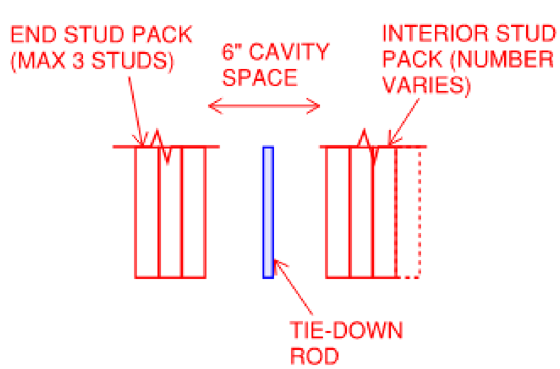


Figure: Compression Posts and Tie-down Rod arrangement

Stud Capacities:

For 2x6 walls, the stud capacities are calculated as below:

$f_c = 11.5MPa\ (SPF\ No. 1/2)$

$K_D = 1.15\ (Short\ Duration)$

$K_H = 1.1\ (Case\ 2\ System)$

$K_{Sc} = 1\ (Dry\ Service\ Conditions)$

$K_T = 1\ (No\ treatment)$

$F_c = f_c(K_D K_H K_{Sc} K_T) = 14.5MPa$

$Effective\ stud\ height = 2337mm$

$Area\ (stud) = 5320mm^2$

$K_{Zc,b} = 1.3$

$K_{Zc,d} = 1.3$

$C_{c,b} = 0\ (Restained\ in\ plane\ of\ the\ wall)$

$C_{c,d} = 16.7$

$K_{c,b} = 1$

$K_{c,d} = 0.7$

$\emptyset = 0.8$

$P_r = \min(P_{r,b}, P_{r,d}) = \min(80.5kN, 55.0kN) = 55kN$

Bearing capacities:

$f_{cp} = 7MPa\ (D. Fir\ No. 1/2)$

$K_D = 1.15\ (Short\ Duration)$

$K_{Scp} = 1\ (Dry\ Service\ Conditions)$

$K_T = 1\ (No\ treatment)$

$F_{cp} = f_{cp}(K_D K_{Scp} K_T) = 8.05MPa$

$K_{Zcp} = 1.15$

$\emptyset = 1$ (Mean value of bearing strength of wood plates is considered for the seismic loads)

$Q_{rp} = 49.2kN$

Tie down and end post forces:

The shear force and moment per shear wall is given by:

$$V_i = v_i * \mathrm{L}_i$$

$$M_i = V_i * \mathrm{H}_i + M_{i+1}$$

Tie down forces at level i:

$$T_{i,L/R} = 1.2 * \frac{M_i}{\mathrm{l}_i} - P_{DL,L/R}$$

where,

$$l_i = L_i - Tiedown\ offset(L) - Tiedown\ offset(R)$$

$$P_{DL,L/R} = w_{DL} * L_i/2$$

For end posts at level i,

$$C_{i,L/R} = \frac{M_i}{\mathrm{l}_i}$$

where,

V_i is shear force in a shear wall at level i,

L_i is the length of the shear wall at level i,

M_i is moment in the shear wall at level i,

$T_{i,L/R}$ is tie down force demand at level i.

w_{DL} is counteracting dead load on shear wall for level i.

l_i is the moment arm of the shear wall at level i.

$P_{DL,L/R}$ is effective counteracting dead load on wall ends for level i.

The tables below summarize tie-down and end post requirements for wall lines X1.1 and Y2.1. Analysis for other walls would follow similar procedures and is not shown here.

Wall #	X1.1	Wall Line:	X1	Stud Size:	2x6	TD Offset:	7.5	in
Level	Height	Wall Length	Counteracting DL Trib.	Shear Demand Per Unit Length	Tie-Down Demand	End Post Demand	**Tie-Down Estimate**	**End Post Estimate**
i	H_i	L_i	$w_{DL}\ trib$	v_i	$T_{i,L/R}$	$C_{i,L/R}$		
	ft	ft	ft	plf	lb	lb		**Count**
6	9	27.5	3	668	6505	6302	**0.75" ⌀**	**2**
5	9	27.5	3	1539	22060	20811	**1" ⌀**	**2**
4	9	27.5	3	2235	45494	41887	**1.5 ⌀**	**4**
3	9	27.5	3	2758	74838	67887	**1.75" ⌀**	**8**
2	9	27.5	3	3106	108122	97171	**1.5" ⌀ H**	**10**
1	9	27.5	3	3280	143376	128095	**1.75" ⌀ H**	**12**

Wall #	Y2.1	Wall Line:	Y2	Stud Size:	2x6	TD Offset:	7.5	in
Level	Height	Wall Length	Counteracting DL Trib.	Shear Demand Per Unit Length	Tie-Down Demand	End Post Demand	**Tie-Down Estimate**	**End Post Estimate**
i	H_i	L_i	$w_{DL}\ trib$	v_i	$T_{i,L/R}$	$C_{i,L/R}$		
	ft	ft	ft	plf	lb	lb		**Count**
6	9.0	22.0	8	528	3796	5043	**0.75" ⌀**	**2**
5	9.0	22.0	8	1221	13820	16697	**0.75" ⌀**	**2**
4	9.0	22.0	8	1776	30191	33639	**1.25" ⌀**	**4**
3	9.0	22.0	8	2191	51321	54547	**1.5 ⌀**	**6**
2	9.0	22.0	8	2468	75624	78100	**1.75" ⌀**	**8**
1	9.0	22.0	8	2607	101514	102975	**2" ⌀**	**10**

C4. Intermediate Design Summary

Below is the summary of trial selection of all shear walls. This design is based on the strength check on the shear walls at code-based time-period, and the forces are distributed to walls proportional to wall length. These forces will be recalculated for final design by stiffness compatibility, where applicable. The tie-down sizes are rationalized to keep the net rod area higher or equal to the area on the upper level. This design forms the first iteration for the deflection analysis.

Corridor Walls (X-Direction):

		Wall #						
Wall Line:	X1	X1.1	X1.2	X1.3	Stud Size:	2x6	TD Offset:	7.5 in
Level	Height	Wall Length	Wall Length	Wall Length	Shear Demand Per Unit Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i	L_i	L_i	v_i			
	ft	ft	ft	ft	plf			Count
6	9	27.5	18	30.5	668	SW3	0.75" ⌀	2
5	9	27.5	18	30.5	1539	(2)-SW2	1" ⌀	2
4	9	27.5	18	30.5	2235	(2)-SW2-H	1.5 ⌀	4
3	9	27.5	18	30.5	2758	Mid+Std	1.5" ⌀ H	8
2	9	27.5	18	30.5	3106	Mid+Std	1.5" ⌀ H	10
1	9	27.5	18	30.5	3280	Mid+Std	1.75" ⌀ H	12

		Wall #							
Wall Line:	X2	X2.1	X2.2	X2.3	X2.4	Stud Size:	2x6	TD Offset:	7.5 in
Level	Height	Wall Length	Wall Length	Wall Length	Wall Length	Shear Demand Per Unit Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i	L_i	L_i	L_i	v_i			
	ft	ft	ft	ft	ft	plf			Count
6	9	11.75	24	18	30.5	632	SW3	0.75" ⌀	2
5	9	11.75	24	18	30.5	1467	(2)-SW2	1" ⌀	2
4	9	11.75	24	18	30.5	2136	(2)-SW2-H	1.5 ⌀	4
3	9	11.75	24	18	30.5	2637	Mid+Std	1.5" ⌀ H	8
2	9	11.75	24	18	30.5	2971	Mid+Std	1.5" ⌀ H	10
1	9	11.75	24	18	30.5	3138	Mid+Std	1.75" ⌀ H	12

Party Walls (Y-Direction):

		Wall #						
Wall Line:	Y1	Y1.1	Y1.2	Y1.3	Stud Size:	2x6	TD Offset:	7.5 in

Level	Height	Wall Length	Wall Length	Wall Length	Shear Demand Per Unit Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i	L_i	L_i	v_i			
	ft	ft	ft	ft	plf			Count
6	9	13	16	22	437	SW4	0.75" ⌀	2
5	9	13	16	22	985	SW2-H	1" ⌀	2
4	9	13	16	22	1424	(2)-SW2	1.25" ⌀	4
3	9	13	16	22	1754	(2)-SW2	1.5 ⌀	6
2	9	13	16	22	1973	(2)-SW2-H	1.75" ⌀	6
1	9	13	16	22	2083	(2)-SW2-H	2" ⌀	8

		Wall #				
Wall Line:	Y2	Y2.1	Stud Size:	2x6	TD Offset:	7.5 in
Level	Height	Wall Length	Shear Demand Per Unit Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i	v_i			
	ft	ft	plf			Count
6	9	22	528	SW4	0.75" ⌀	2
5	9	22	1221	(2)-SW2	0.75" ⌀	2
4	9	22	1776	(2)-SW2	1.25" ⌀	4
3	9	22	2191	(2)-SW2-H	1.5 ⌀	6
2	9	22	2468	Mid+Std	1.75" ⌀	8
1	9	22	2607	Mid+Std	2" ⌀	10

C5. Deflection Equations

Four term Deflection Equation:

The deflection equation of the wood shear walls:

$$\Delta_{T,i} = \Delta_{s,i} + \Delta_{n,i} + \Delta_{b,accum,i} + \Delta_{a,i}$$

where,

$\Delta_{s,i}$ is deflection due to panel shear,

$\Delta_{n,i}$ is deflection due to nail slippage,

$\Delta_{b,accum,i}$ is deflection due to wall bending and rotation of stacked wall panels,

$\Delta_{a,i}$ is deflection due to tie-down slip.

Apparent Shear-Through-Thickness Rigidity:

The deflection due to the plywood shear:

$$\Delta_{s,i} = \frac{v_i * h_i}{B_{v,i}}$$

The deflection due to nail slippage:

$$\Delta_{n,i} = 0.0025h_i * e_{n,i}$$

$$\mathrm{e}_{n,i} = \left(\frac{0.013(v_i/n_p) * s_i}{{d_{f,i}}^2}\right)^2$$

where,

h_i is sheathing height at level i,

$e_{n,i}$ is nail slip at level i,

n_p is the number of shear planes per shear wall at level i,

v_i/n_p is shear force per wall sheathing.

The relation of plywood deflection varies linearly with applied force, i.e. the wall stiffness is independent of applied load. This is also true for wall bending and tie-down slip discussed in the next section. However, nail slippage is not a linear relation between deflection and applied force, making wall stiffness dependent on applied force.

It is desirable to approximate wall stiffness as linear for quick convergence of iterative analysis. A method presented in SDPWS2015 is used to combine panel shear and nail slippage into a single linear term based on stiffness at maximum shear capacity. This simplifies the four-term deflection equation into a three-term equation.

Combined panel shear and nail slippage deflection:

$$\Delta_{sn,i} = \Delta_{s,i} + \Delta_{n,i}$$

$$\Delta_{sn,i} = \frac{v_i * h_i}{B_{v,i}} + 0.0025h_i * e_{n,i}$$

Writing combined term as,

$$\Delta_{sn,i} = \frac{v_i * h_i}{B_{a,i}}$$

Where $B_{a,i}$ is the apparent shear through thickness rigidity of the shear wall.

$$\frac{v_i * h_i}{B_{a,i}} = \frac{v_i * h_i}{B_{v,i}} + 0.0025h_i * e_{n,i}$$

$$B_{a,i} = \frac{v_i}{\frac{v_i}{B_{v,i}} + 0.0025e_{n,i}}$$

Calculating value of B_a at $v = v_r$,

$$B_a = \frac{v_r}{\frac{v_r/n_p}{B_v} + 0.0025e_{n,r}}$$

Where $e_{n,r}$ is the nail slip at level i at max shear wall capacity v_r,

$$e_{n,r} = \left(\frac{0.013v_r/n_p) * s}{{d_f}^2}\right)^2$$

The above equations are used to calculate the values of $e_{n,r}$ *and* B_a for typical shear wall assemblies considered in this analysis. These are given in the table below:

Shear Wall ID	Nail Dia	Panel Edge Nail Spacing	No. of Shear Planes	Shear Capacity	Shear-Through-Thickness Rigidity of Sheathing	Nail Slip	**Apparent Shear Through Thickness Rigidity of wall**
	d_f	s	n_p	v_r	B_v	$e_{n,r}$	$\boldsymbol{B_a}$
	mm	mm		kN/m	N/mm		**N/mm**
SW4	3.33	100	1	8.3	11000	0.95	**2,659**
SW3	3.33	75	1	10.6	11000	0.87	**3,381**
SW2	3.33	50	1	13.7	11000	0.64	**4,794**

SW2-H	3.66	50	1	16.8	12000	0.66	**5,488**
(2)-SW2	3.33	50	2	27.4	11000	0.64	**9,588**
(2)-SW2-H	3.66	50	2	33.6	12000	0.66	**10,976**
MidPly	3.66	50	2	33.7	12000	0.67	**10,958**
Mid+Std	3.66	50	3	50.5	12000	0.67	**16,446**

The B_a for Mid+Std wall is sum of B_a of MidPly sheathing and SW2-H sheathing.

Consolidated Three-term Deflection Equation:

The three term deflection equation can then be written as:

$$\Delta_{T,i} = \Delta_{sn,i} + \Delta_{b,accum,i} + \Delta_{a,i}$$

Rational time-period:

The deflection of the shear wall is calculated at the rational time-period of the building (Rayleigh Period) :

$$T_{a,rational} = 2\pi \sqrt{\frac{\sum W_i \Delta^2_{T,accum,i}}{g \sum F_i \Delta_{T,accum,i}}}$$

where,

$\Delta_{T,accum,i}$ is the cumulative deflection of shear wall line on level i

F_i is storey force per diaphragm trib associated with shear wall line on level i,

g is acceleration due to gravity,

W_i is seismic weight of the diaphragm trib associated with shear wall line on level i.

However, since the time-period itself is a function of building deflections, it makes the process of obtaining deflections iterative, beginning with code-based time-period and iterating until it converges at the rational time-period. In this example, the first iteration of calculation is shown in detail (for code-based time-period). Results from subsequent iterations are summarized afterwards.

C6. Deflection: Short Direction

Deflection for wall line Y2 is more straightforward since all shear walls have the same length and relative stiffness and support similar diaphragm tributary areas. This section focuses on the detailed deflection calculation for shear wall Y2.1.

For the first iteration,

$$T_a = T_{a,code}$$

The forces on wall Y2.1 for the first iteration, calculated in the previous sections, are summarized in the table below.

Level	Counteracting DL	Shear Demand	Shear Demand per unit length	Counteracting DL Moment	Overturning Moment	
i	$P_{DL,L/R}$	V_i	v_i	M_{DL}	M_i	M_{i+1}
	kN	kN	kN/m	kNm	kNm	kNm
6	10.4	52.1	7.8	67.32	75.5	0
5	27.7	120.4	17.9	118.17	287.5	75.5
4	45.3	175	26.1	118.17	649.3	287.5
3	62.9	215.9	32.2	118.17	1123.5	649.3
2	80.5	243.2	36.3	118.17	1672.6	1123.5
1	98.1	256.9	38.3	118.17	2259.1	1672.6

B6.1. Combined Deflection due to panel shear and nail slippage:

As discussed above, the combined deflection term $\Delta_{sn,i}$:

$$\Delta_{sn,i} = \frac{v_i * h_i}{B_{a,i}}$$

The floor joists + floor sheathing depth are considered as 250mm (10”) for all levels. Hence sheathing height,

$$h_i = H_i - 250mm$$

For wall Y2.1, this gives us the following deflections:

Level	SW Y2.1	Level Height	Sheathing Height	Wall Length	Apparent Shear Through Thickness Rigidity	Shear Demand per unit length	**Combined Plywood Shear and Nail Slip Deflection**
i		H_i	h_i	L_i	$B_{a,i}$	v_i	$\boldsymbol{\Delta_{sn,i}}$
		m	m	m	kN/m	kN/m	**mm**
6	SW4	2.74	2.49	6.71	2716	7.8	**7.1**
5	(2)-SW2	2.74	2.49	6.71	9952	17.9	**4.5**
4	(2)-SW2	2.74	2.49	6.71	9952	26.1	**6.5**
3	(2)-SW2-H	2.74	2.49	6.71	10975	32.2	**7.3**
2	Mid+Std	2.74	2.49	6.71	16446	36.3	**5.5**
1	Mid+Std	2.74	2.49	6.71	16446	38.3	**5.8**

B6.2. Deflection due to wall bending:

The EI_i for shear wall is calculated as follows:

$$E_c = 9500\ MPa \text{ (SPF end post)}$$

$$E_t = 200000\ MPa \text{ (Steel tie-down)}$$

$$n = \frac{E_t}{E_c} = 21.05$$

Transformed tie-down area and moment of inertia:

$$A_{t,tr,i} = A_{t,i} * n$$

$$y_{tr,i} = \frac{A_{c,i} * l_i}{A_{t,tr,i} + A_{c,i}}$$

$$I_{tr,i} = A_{t,tr,i} * {y_{tr,i}}^2 + A_{c,i} * (l_i - y_{tr,i})^2$$

$$EI_{wall,i} = E_c * I_{tr,i}$$

where,

E_c is young's modulus of end post

E_t is young's modulus of tie-down

$A_{t,i}$ is net tie down, on level i

$A_{c,i}$ is net end post area on level i.

$A_{t,tr,i}$ is transformed tie-down area on level i.

$y_{tr,i}$ is depth of the wall neutral axis on level i.

$I_{tr,i}$ is transformed wall second moment of area on level i.

For wall Y2.1, EI_i for all levels are given below:

Level	Length	Tie-down	End Post	Net Tie-down Area	Net End Post Area	Lever Arm	Transformed Tie-down Area	Neutral Axis Depth	Second Moment of Area	**Bending Stiffness**
i	L_i			$A_{t,i}$	$A_{c,i}$	l_i	$A_{t,tr,i}$	$y_{tr,i}$	$I_{tr,i}$	$\boldsymbol{EI_i}$
	m		Count	mm2	mm2	m	mm2	m	m4	**kNm2**
6	6.71	0.75" ⌀	2	215	10645	6.32	4536	4.43	1.30E-01	**1.20E+06**
5	6.71	0.75" ⌀	2	215	10645	6.32	4536	4.43	1.30E-01	**1.20E+06**
4	6.71	1.25" ⌀	4	625	21290	6.32	13161	3.91	3.30E-01	**3.10E+06**
3	6.71	1.5 ⌀	6	910	31935	6.32	19151	3.95	4.80E-01	**4.50E+06**

2	6.71	1.75" ⌀	8	1226	42581	6.32	25806	3.94	6.40E-01	**6.10E+06**
1	6.71	2" ⌀	10	1613	53226	6.32	33956	3.86	8.30E-01	**7.90E+06**

The deflection due to wall bending for a single level:

$$\Delta_{b,i} = \frac{V_i H_i^{\ 3}}{3EI_i} + \frac{M_{i+1} H_i^{\ 2}}{2EI_i}$$

For level 6,

$$M_{i+1} = M_7 = 0$$

The wall rotations $\theta_{b,i}$ are:

$$\theta_{b,i} = \frac{V_i H_i^{\ 2}}{2EI_i} + \frac{M_{i+1} H_i}{EI_i}$$

For level 0,

$$\theta_{b,0} = 0$$

Total bending deflection of multi-storey stacked shear wall is:

$$\Delta_{b,accum,i} = \Delta_{b,i} + H_i \sum_{0}^{i-1} \theta_{b,j}$$

Level	Level Height	Bending Stiffness	Shear Demand	Overturning moment	Bending Deflection	Wall Rotation	Cumulative Rotation	**Cumulative Bending Deflection**
i	H_i	EI_i	V_i	M_{i+1}	$\Delta_{b,i}$	$\theta_{b,i}$	$\sum_{0}^{i-1} \theta_{b,j}$	$\boldsymbol{\Delta_{b,accum,i}}$
	m	kNm2	kN	kNm	mm	rads	rads	**mm**
6	2.74	1.20E+06	52.1	0	0.3	1.60E-04	2.90E-03	**8.4**
5	2.74	1.20E+06	120.4	75.5	0.9	5.50E-04	2.40E-03	**7.5**
4	2.74	3.10E+06	175	287.5	0.7	4.70E-04	1.90E-03	**6.0**
3	2.74	4.50E+06	215.9	649.3	0.9	5.70E-04	1.40E-03	**4.6**
2	2.74	6.10E+06	243.2	1123.5	1.0	6.50E-04	7.10E-04	**2.9**
1	2.74	7.90E+06	256.9	1672.6	1.0	7.10E-04	0.00E+00	**1.0**

B6.3. Deflection due to tie-down slippage:

The deflection due to anchor slippage:

$$\Delta_{a,i} = H_i \sum_{0}^{i-1} \frac{d_{a,j}}{l_j}$$

where,

$d_{a,i}$ is tie-down slip on level i,

l_i is the moment arm of the shear wall at level i.

Assumed values of tie-down slip, typical for products available in the market, are given below,

Max tie-down slip (including ratcheting device) = 1.3mm

Max bearing compression = 1mm

Therefore, total tie-down slip,

$$d_{a,max} = 2.3mm$$

$$d_{a,i} = d_{a,max} * T_i/T_r$$

For level 0,

$$d_{a,i} = d_{a,0} = 0$$

For shear wall Y2.1,

Level	Level Height	Tie-down	Tie-Down Capacity	Lever Arm	Overturning Moment	Tie-Down Demand	Tie-Down Slip	Wall Rotation	Cumulative Rotation	**Deflection due to Tie-Down Slip**
i	H_i	Tie-down	T_r	l_i	M_i	T_i	$d_{a,i}$	$\frac{d_{a,i}}{l_i}$	$\sum_{0}^{i-1} \frac{d_{a,j}}{l_j}$	$\boldsymbol{\Delta_{a,i}}$
	m		kN	m	kNm	kN	mm	rads	rads	**mm**
6	2.74	0.75" ⌀	63.5	6.32	75.5	11.9	0.4	6.80E-05	1.30E-03	**3.9**
5	2.74	0.75" ⌀	63.5	6.32	287.5	45.5	1.6	2.60E-04	1.00E-03	**4.4**
4	2.74	1.25" ⌀	181.1	6.32	649.3	102.7	1.3	2.10E-04	8.00E-04	**3.5**
3	2.74	1.5 ⌀	262	6.32	1123.5	177.6	1.6	2.50E-04	5.50E-04	**3.1**
2	2.74	1.75" ⌀	355	6.32	1672.6	264.5	1.7	2.70E-04	2.80E-04	**2.5**
1	2.74	2" ⌀	465.8	6.32	2259.1	357.2	1.8	2.80E-04	0.00E+00	**1.8**

B6.4. Total Deflection from first iteration:

In summary, the total elastic deflection for wall Y2.1:

Level	Combined Plywood shear and Nail Slip Deflection	Cumulative Bending Deflection	Deflection due to Tie-Down Slip	**Total Deflection**
i	$\Delta_{sn,i}$	$\Delta_{b,accum,i}$	$\Delta_{a,i}$	$\boldsymbol{\Delta_{T,i}}$
	mm	mm	mm	**mm**
6	7.1	8.4	3.9	**19.4**
5	4.5	7.5	4.4	**16.4**
4	6.5	6	3.5	**16**
3	7.3	4.6	3.1	**15**
2	5.5	2.9	2.5	**10.9**
1	5.8	1	1.8	**8.6**

Therefore, the rational time-period of the building:

Level	Level Height	Total Deflection	Total Cumulative Deflection	Storey Shear for Wall Line	Storey Forces for Wall Line		
i	H_i	$\Delta_{T,i}$	$\Delta_{T,Cumm,i}$	V_i	F_i	$W_i\Delta^2_{T,accum,i}$	$F_i\Delta_{T,accum,i}$
	m	mm	mm	kN	kN	N-m	N-m
6	2.74	19.4	86.2	729.1	729.1	8.40E+03	6.29E+04
5	2.74	16.4	66.9	1684.9	955.8	7.94E+03	6.39E+04
4	2.74	16	50.5	2449.6	764.6	4.52E+03	3.86E+04
3	2.74	15	34.4	3023	573.5	2.10E+03	1.97E+04
2	2.74	10.9	19.5	3405.3	382.3	6.72E+02	7.44E+03
1	2.74	8.6	8.6	3596.5	191.2	1.31E+02	1.64E+03
Σ						2.38E+04	1.94E+05

$$T_{a,rational} = 2\pi\sqrt{\frac{\sum W_i \Delta^2_{T,accum,i}}{g\sum F_i \Delta_{T,accum,i}}} = \mathbf{0.7s}$$

B6.5. Subsequent iterations and final deflections:

Since,

$$T_{a,code} \neq T_{a,rational}$$

For the next iteration, taking,

$$T_a = T_{a,rational}$$

Taking the updated shear wall forces for Y2.1 for new T_a and repeating the above process for updated deflections and time-period.

Level	$P_{DL,L/R}$	V_i	v_i	M_{DL}	M_i	M_{i+1}
i	kN	kN	kN/m	kNm	kNm	kNm
6	10.4	43.4	6.5	67.32	51.8	0.0
5	27.7	100.4	15.0	118.17	209.0	51.8
4	45.3	145.9	21.8	118.17	491.2	209.0
3	62.9	180.1	26.9	118.17	867.1	491.2
2	80.5	202.9	30.3	118.17	1305.4	867.1
1	98.1	214.3	32.0	118.17	1775.0	1305.4

Skipping ahead to the final results, the time-period converges at the following value:

$$T_a = T_{a,rational} = 0.69s$$

And final total deflections are as follows:

Level	Level Height	Combined Plywood shear and Nail Slip Deflection	Cumulative Bending Deflection	Deflection due to Tie-Down Slip	Total Deflection		**Drift Ratio**
i	H_i	$\Delta_{sn,i}$	$\Delta_{b,accum,i}$	$\Delta_{a,i}$	$\Delta_{T,i}$	$\Delta_{T,i} * R_d R_o$	$\boldsymbol{\Delta_{T,i} * R_d R_o / H_i}$
	m	mm	mm	mm	mm	mm	**%age**
6	2.74	6	6.7	3	15.7	79.9	**2.9%**
5	2.74	3.8	6	3.4	13.1	66.9	**2.4%**

4	2.74	5.5	4.8	2.7	13	66.4	**2.4%**
3	2.74	6.2	3.7	2.4	12.2	62.4	**2.3%**
2	2.74	4.6	2.3	2	8.9	45.4	**1.7%**
1	2.74	4.9	0.8	1.4	7.1	36.2	**1.3%**

Since the drift ratio exceeds 2.5%, the level 6 wall sheathing is revised to reduce deflection. The strength check is not repeated, as the original design already satisfied capacity requirements at code time-period and the design is upsized.

Revised design:

	Base Design			Revised Design		
Level	Shearwall ID	Tie-Down	End Post	Shearwall ID	Tie-Down	End Post
			Count			Count
6	SW4	0.75" ⌀	2	**SW2**	0.75" ⌀	2
5	(2)-SW2	0.75" ⌀	2	(2)-SW2	0.75" ⌀	2
4	(2)-SW2	1.25" ⌀	4	(2)-SW2	1.25" ⌀	4
3	(2)-SW2-H	1.5 ⌀	6	(2)-SW2-H	1.5 ⌀	6
2	Mid+Std	1.75" ⌀	8	Mid+Std	1.75" ⌀	8
1	Mid+Std	2" ⌀	10	Mid+Std	2" ⌀	10

The rational time-period for this revised design is:

$$T_a = T_{a,rational} = 0.68s$$

The total deflections for the shear wall for the revised design:

Level	Level Height	Combined Plywood shear and Nail Slip Deflection	Cumulative Bending Deflection	Deflection due to Tie-Down Slip	Total Deflection		**Drift Ratio**
i	H_i	$\Delta_{sn,i}$	$\Delta_{b,accum,i}$	$\Delta_{a,i}$	$\Delta_{T,i}$	$\Delta_{T,i} * R_d R_o$	$\boldsymbol{\Delta_{T,i} * R_d R_o / H_i}$
	m	mm	mm	mm	mm	mm	**%age**
6	2.74	3.0	6.8	3.0	12.8	65.2	**2.4%**
5	2.74	3.5	6.0	3.4	12.9	66.0	**2.4%**
4	2.74	5.0	4.9	2.8	12.7	64.6	**2.4%**
3	2.74	6.2	3.7	2.5	12.4	63.1	**2.3%**

2	2.74	4.7	2.3	2.0	9.0	45.9	**1.7%**
1	2.74	4.9	0.8	1.4	7.2	36.6	**1.3%**

Final drift ratios with the revised sheathing are within acceptable limits (≤2.5%).

C7. Deflection: Long Direction

Corridor Wall lines (X direction walls) are connected by continuous elements (double spliced top plates or drag struts) at every level which re-distributes load among the walls at all levels, such that these must deflect by the same value at every level, despite their differing lengths and stiffnesses. The deflection of the wall lines is obtained by back calculating wall stiffnesses from trial deflection values and re-distributing forces among the shear walls based on these stiffnesses to obtain same deflection on all walls.

SCHEMATIC OF CORRIDOR WALL LINE

For N shear walls in a corridor wall line, the force attributed to a single shear wall is:

$$V_i = k_i / k_{T,i} * V_{T,i}$$

where,

$$k_{T,i} = \sum_{1}^{N} k_{n,i}$$

$$V_{T,i} = \sum_{1}^{N} V_{n,i}$$

and k_i is calculated iteratively from the trial deflections of the walls,

$$k_{i,trial} = V_{i,trial}/\Delta_i$$

The first set of trial forces to calculate the trial stiffnesses are already calculated in previous sections. To re-iterate here:

$$V_i = \mathrm{L}_i/\,\mathrm{L}_{T,i} * V_{T,i}$$

That is, the relative stiffness is proportional to the length of the wall. This assumption is true for squat shear walls, where the primary mode of deflection is due to shear forces. But for six-storey stacked shear walls, the bending deflections become significant, especially at the upper levels, that actual wall stiffness is proportional to a higher exponent of wall length. This makes the short walls more sensitive to bending deflections. This value of wall stiffness also varies for different levels for the same shear wall, with lower levels acting with more stiffness (due to deflecting primarily in shear only) than the upper levels (due to deflecting in both bending and shear), with continuous elements redistributing the forces between longer and shorter walls.

B7.1. Deflection of Wall line X1

Corridor wall line X1 consists of shear walls X1.1, X1.2 and X1.3 as given in the X direction key plan. Wall X1.2 repeats 5 times while walls X1.1 and X1.2 do not repeat. This can be written as:

$$(n_{X1.1}, n_{X1.2}, n_{X1.3}) = (1 \quad 5 \quad 1)$$

And,

$$(\mathrm{L}_{X1.1}, \mathrm{L}_{X1.2}, \mathrm{L}_{X1.3}) = (27.5 \quad 18 \quad 30)\, ft$$

The initial relative wall stiffness for these shear wall types at level i, can be given by:

$$({k^R}_{X1.1,i}, {k^R}_{X1.2,i}, {k^R}_{X1.3,i}) = (\mathrm{L}_{X1.1,i}, \mathrm{L}_{X1.2,i}, \mathrm{L}_{X1.3,i})/(\mathrm{L}_{X1.1,i} + \mathrm{L}_{X1.2,i} + \mathrm{L}_{X1.3,i})$$
$$= \begin{pmatrix} 0.36 & 0.24 & 0.40 \\ 0.36 & 0.24 & 0.40 \\ 0.36 & 0.24 & 0.40 \\ 0.36 & 0.24 & 0.40 \\ 0.36 & 0.24 & 0.40 \\ 0.36 & 0.24 & 0.40 \end{pmatrix}$$

Total corridor wall forces at $T_a = T_{a,code}$,

$$V_{T,i} = \begin{pmatrix} 100 \\ 230 \\ 334 \\ 412 \\ 464 \\ 490 \end{pmatrix} kip$$

Therefore,

$$(V_{X1.1,i}, V_{X1.2,i}, V_{X1.3,i}) = (k^R{}_{X1.1,i}/n_{X1.1}, k^R{}_{X1.2,i}/n_{X1.2}, k^R{}_{X1.3,i}/n_{X1.3}) * V_{T,i}$$

$$= \begin{pmatrix} 18.56 & 12.15 & 20.59 \\ 42.74 & 27.98 & 47.41 \\ 62.08 & 40.64 & 68.86 \\ 76.59 & 50.13 & 84.95 \\ 86.26 & 56.46 & 95.67 \\ 91.10 & 59.63 & 101.04 \end{pmatrix} kip$$

Performing deflection analysis individually on these walls for the given forces,

$$(\Delta_{X1.1,i}, \Delta_{X1.2,i}, \Delta_{X1.3,i}) = \begin{pmatrix} 19.7 & 27.5 & 18.3 \\ 17.7 & 24.7 & 16.5 \\ 17.6 & 23.4 & 16.6 \\ 14.6 & 19.2 & 13.8 \\ 12.8 & 15.7 & 12.3 \\ 10.2 & 11.1 & 10.0 \end{pmatrix} mm$$

As expected, with the initial force distribution, the deflections for shortest wall (second column) are larger than the longer walls (first and third column). Also, this effect is higher for upper levels than lower levels.

Recalculating the relative wall stiffnesses,

$$(k_{X1.1,i}, k_{X1.2,i}, k_{X1.3,i}) = (V_{X1.1,i}, V_{X1.2,i}, V_{X1.3,i})/(\Delta_{X1.1,i}, \Delta_{X1.2,i}, \Delta_{X1.3,i}) = \begin{pmatrix} 0.38 & 0.18 & 0.45 \\ 0.38 & 0.18 & 0.45 \\ 0.37 & 0.18 & 0.44 \\ 0.37 & 0.19 & 0.44 \\ 0.37 & 0.20 & 0.43 \\ 0.37 & 0.22 & 0.41 \end{pmatrix}$$

Compared to the initial stiffness distribution, longer walls are relatively stiffer and attract more forces, especially on upper levels. The shorter wall (X1.2 here) is, relatively speaking, stiffer on lower levels (behaving as squat shear wall) than on upper levels.

$$(V_{X1.1,i}, V_{X1.2,i}, V_{X1.2,i}) = (k_{X1.1,i}/n_{X1.1}, k_{X1.2,i}/n_{X1.2}, k_{X1.3,i}/n_{X1.3}) * V_{T,i}$$

$$= \begin{pmatrix} 22.04 & 10.31 & 26.31 \\ 50.70 & 23.77 & 60.49 \\ 72.04 & 35.44 & 84.87 \\ 88.20 & 44.10 & 103.52 \\ 96.30 & 51.33 & 111.33 \\ 95.73 & 57.31 & 108.01 \end{pmatrix} kip$$

Calculating deflections with these forces and repeating the process, the deflections in the shear walls are:

$$(\Delta_{X1.1,i}, \Delta_{X1.2,i}, \Delta_{X1.3,i}) = \begin{pmatrix} 23.3 & 23.6 & 23.3 \\ 21.0 & 21.3 & 20.9 \\ 20.5 & 20.4 & 20.5 \\ 16.9 & 16.9 & 16.9 \\ 14.3 & 14.2 & 14.4 \\ 10.7 & 10.6 & 10.7 \end{pmatrix} mm$$

We can see, as expected for all the shear walls in the corridor wall line, the wall deflections have converged to same values on all levels. Updating the wall stiffness and force distribution among the shear walls:

$$(k^R{}_{X1.1,i}, k^R{}_{X1.2,i}, k^R{}_{X1.3,i}) = \begin{pmatrix} 0.38 & 0.17 & 0.46 \\ 0.38 & 0.18 & 0.45 \\ 0.38 & 0.18 & 0.44 \\ 0.37 & 0.20 & 0.43 \\ 0.37 & 0.21 & 0.42 \\ 0.36 & 0.23 & 0.41 \end{pmatrix}$$

$$(V_{X1.1,i}, V_{X1.2,i}, V_{X1.3,i}) = \begin{pmatrix} 22.34 & 10.17 & 26.74 \\ 51.48 & 23.39 & 61.62 \\ 71.80 & 35.61 & 84.27 \\ 87.72 & 44.40 & 102.47 \\ 94.19 & 52.43 & 107.91 \\ 92.74 & 58.81 & 103.51 \end{pmatrix} kip$$

From this point onwards, the deflection calculation follows the same procedure given in the previous section for Y direction walls. Deflection of any shear wall X1.1, X1.2 or X1.3 can be taken as representative of the deflection of the entire corridor line, since these all are going to be same. Providing the results for shear wall X1.1 below,

$$T_a = T_{a,rational} = 0.78s$$

Level	Level Height	Combined Plywood shear and Nail Slip Deflection	Cumulative Bending Deflection	Deflection due to Tie-Down Slip	Total Deflection		**Drift Ratio**
i	H_i	$\Delta_{sn,i}$	$\Delta_{b,accum,i}$	$\Delta_{a,i}$	$\Delta_{T,i}$	$\Delta_{T,i} * R_d R_o$	$\boldsymbol{\Delta_{T,i} * R_d R_o / H_i}$
	m	mm	mm	mm	mm	mm	**%age**
6	2.74	6.5	8.0	3.2	17.7	90.1	**3.3%**
5	2.74	5.2	7.3	3.4	15.9	80.9	**2.9%**
4	2.74	6.7	6.1	2.8	15.5	79.2	**2.9%**
3	2.74	5.4	4.9	2.5	12.8	65.5	**2.4%**
2	2.74	5.9	3.3	1.8	10.9	55.7	**2.0%**
1	2.74	5.8	1.1	1.3	8.2	41.8	**1.5%**

Since drift ratio > 2.5%, revising the design,

	Base Design			Revised Design		
Level	Shearwall ID	Tie-Down	End Post	Shearwall ID	Tie-Down	End Post
			Count			Count
6	SW3	0.75" ⌀	2	**SW2-H**	0.75" ⌀	4
5	(2)-SW2	1" ⌀	2	**(2)-SW2-H**	**1.25" ⌀**	**4**
4	(2)-SW2-H	1.5 ⌀	4	**Mid+Std**	**1.75" ⌀**	**8**
3	Mid+Std	1.5" ⌀ H	8	Mid+Std	**1.75" ⌀**	8
2	Mid+Std	1.5" ⌀ H	10	Mid+Std	**2" ⌀ H**	**14**
1	Mid+Std	1.75" ⌀ H	12	Mid+Std	**2" ⌀ H**	**14**

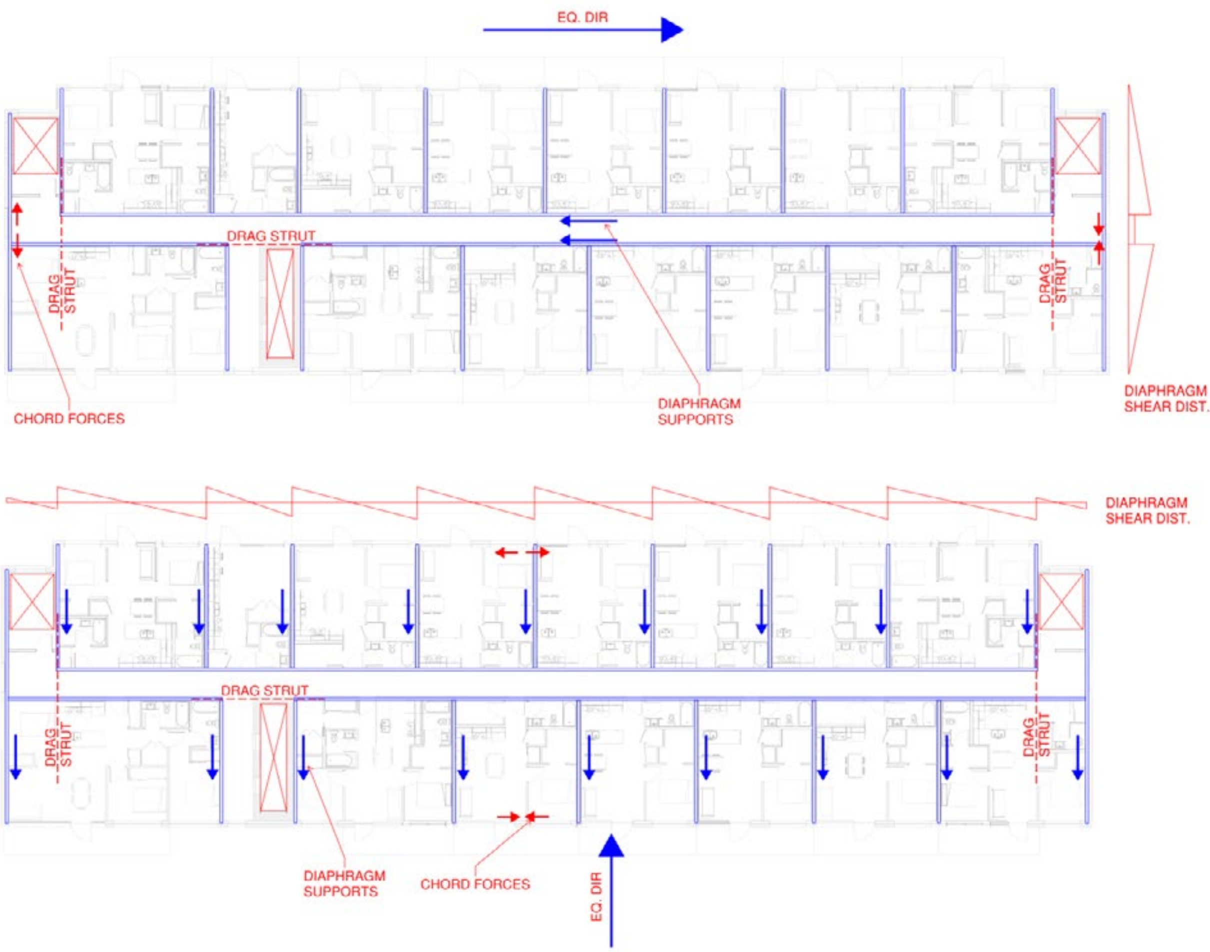
For the revised design,

$$T_a = T_{a,rational} = 0.70s$$

Level	Level Height	Combined Plywood shear and Nail Slip Deflection	Cumulative Bending Deflection	Deflection due to Tie-Down Slip	Total Deflection		**Drift Ratio**
i	H_i	$\Delta_{sn,i}$	$\Delta_{b,accum,i}$	$\Delta_{a,i}$	$\Delta_{T,i}$	$\Delta_{T,i} * R_d R_o$	$\boldsymbol{\Delta_{T,i} * R_d R_o / H_i}$
	m	mm	mm	mm	mm	mm	**%age**
6	2.74	4.5	6.3	2.8	13.6	69.3	**2.5%**
5	2.74	5.1	5.7	2.6	13.4	68.5	**2.5%**
4	2.74	4.9	5.0	2.3	12.2	62.0	**2.3%**
3	2.74	5.7	3.9	2.4	12.0	61.3	**2.2%**
2	2.74	6.3	2.6	1.2	10.0	51.1	**1.9%**
1	2.74	6.2	1.0	1.1	8.3	42.1	**1.5%**

C8. Diaphragm

The design of diaphragm is beyond the scope of this example. In the schematic diaphragm below, diaphragm assumptions and expected behavior are shown:



C9. Design Summary

The final design summary for the building is given below:

Corridor Walls (X-Direction):

		Wall #					
Wall Line:	X1	X1.1	X1.2	X1.3	Stud Size: 2x6	TD Offset:	7.5 in
Level	Height	Wall Length	Wall Length	Wall Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i	L_i	L_i			
	ft	ft	ft	ft			Count
6	9	27.5	18	30.5	SW2-H	0.75" ⌀	4
5	9	27.5	18	30.5	(2)-SW2-H	1.25" ⌀	4
4	9	27.5	18	30.5	Mid+Std	1.75" ⌀	8

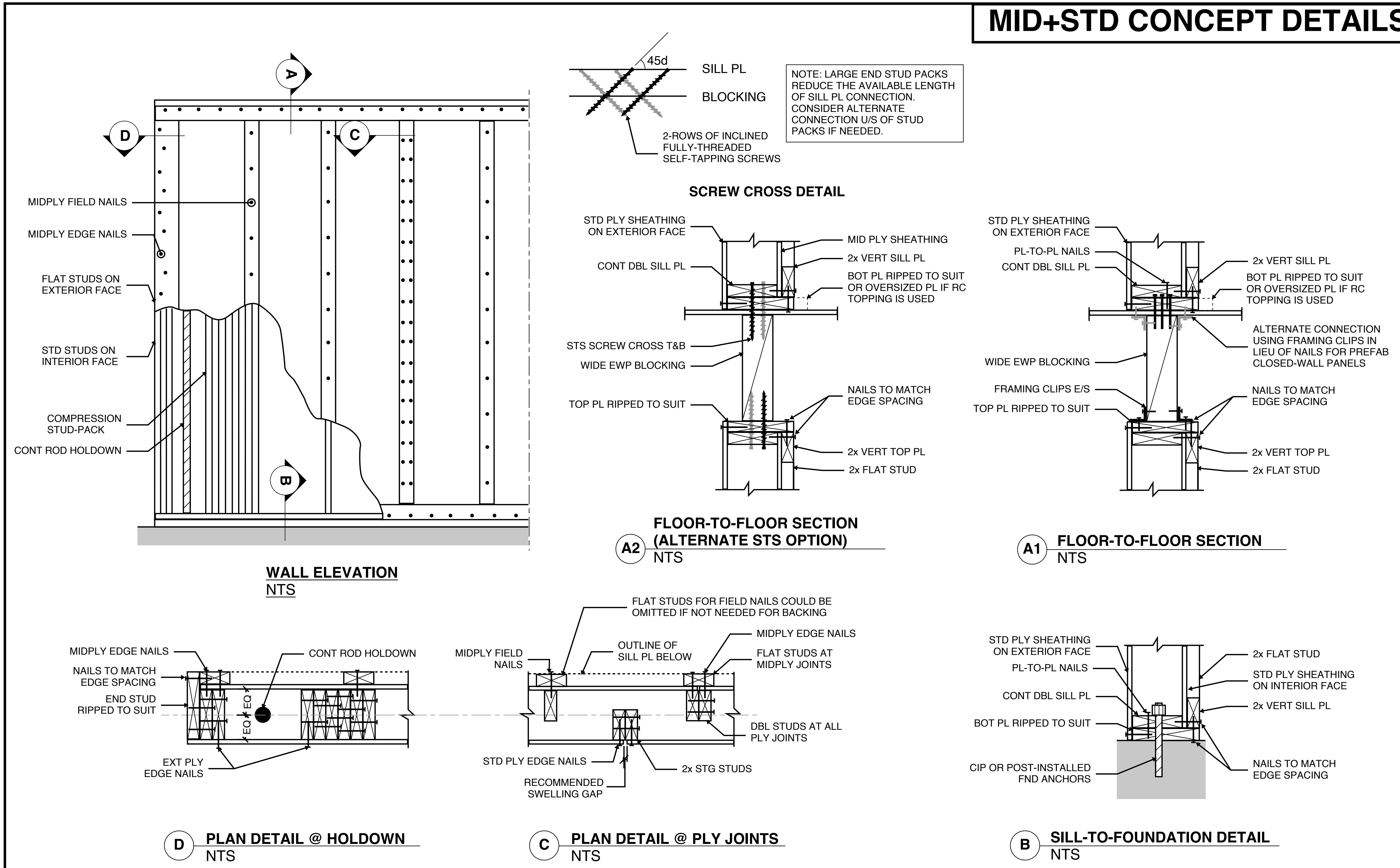
3	9	27.5	18	30.5	Mid+Std	1.75" ⌀	8
2	9	27.5	18	30.5	Mid+Std	2" ⌀ H	14
1	9	27.5	18	30.5	Mid+Std	2" ⌀ H	14

		Wall #						
Wall Line:	X2	X2.1	X2.2	X2.3	X2.4	Stud Size: 2x6	TD Offset:	7.5 in
Level	Height	Wall Length	Wall Length	Wall Length	Wall Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i	L_i	L_i	L_i			
	ft	ft	ft	ft	ft			Count
6	9	11.75	24	18	30.5	SW2-H	1" ⌀	4
5	9	11.75	24	18	30.5	(2)-SW2-H	1.5 ⌀	6
4	9	11.75	24	18	30.5	Mid+Std	1.75" ⌀	8
3	9	11.75	24	18	30.5	Mid+Std	1.75" ⌀	8
2	9	11.75	24	18	30.5	Mid+Std	2" ⌀ H	14
1	9	11.75	24	18	30.5	Mid+Std	2" ⌀ H	14

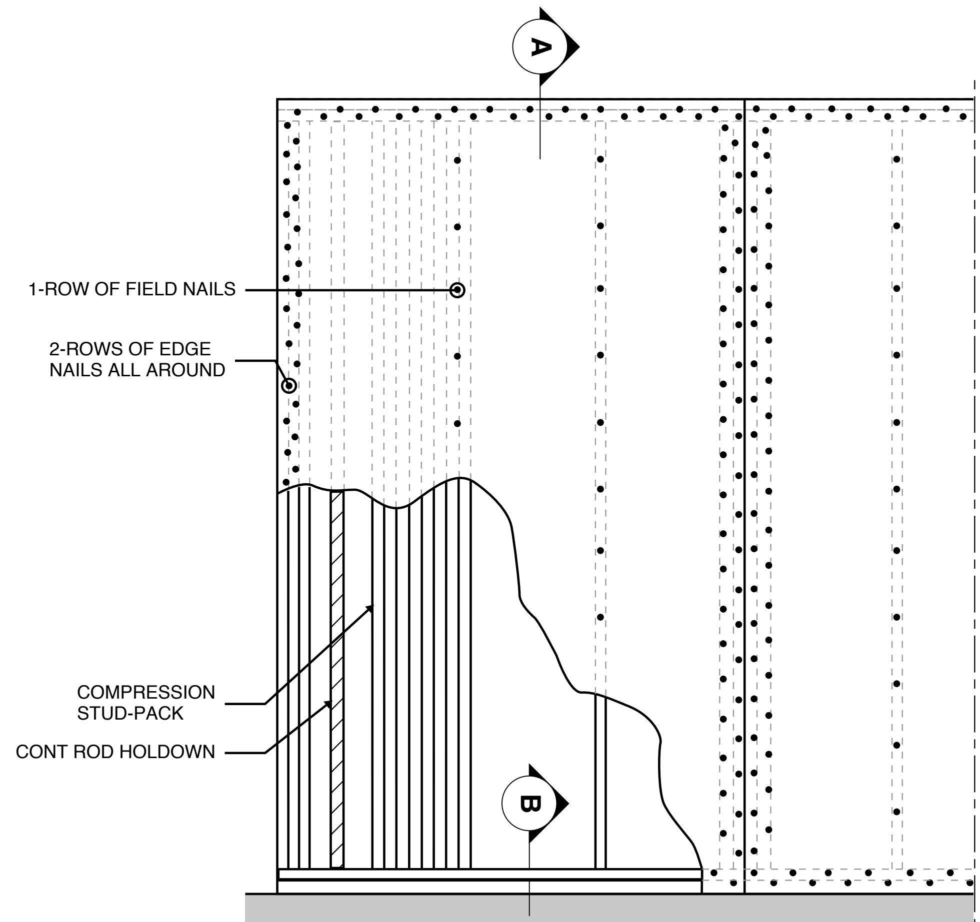
Party Walls (Y-Direction):

		Wall #					
Wall Line:	Y1	Y1.1	Y1.2	Y1.3	Stud Size: 2x6	TD Offset:	7.5 in
Level	Height	Wall Length	Wall Length	Wall Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i	L_i	L_i			
	ft	ft	ft	ft			Count
6	9	13	16	22	SW2-H	0.75" ⌀	2
5	9	13	16	22	(2)-SW2	1" ⌀	2
4	9	13	16	22	(2)-SW2	1.25" ⌀	4
3	9	13	16	22	(2)-SW2	1.5 ⌀	6
2	9	13	16	22	(2)-SW2-H	1.75" ⌀	6
1	9	13	16	22	(2)-SW2-H	2" ⌀	8

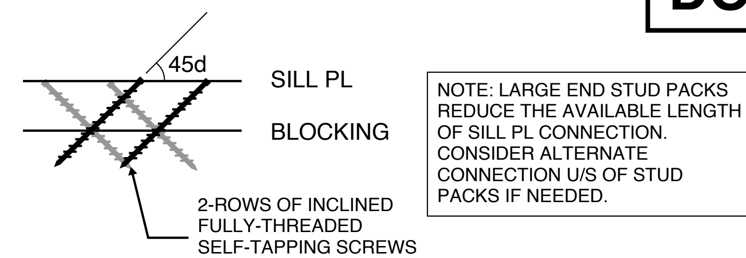
		Wall #			
Wall Line:	Y2	Y2.1	Stud Size: 2x6	TD Offset:	7.5 in
Level	Height	Wall Length	Shearwall ID	Tie-Down	End Post
i	H_i	L_i			
	ft	ft			Count
6	9	22	SW2	0.75" ⌀	2
5	9	22	(2)-SW2	0.75" ⌀	2
4	9	22	(2)-SW2	1.25" ⌀	4
3	9	22	(2)-SW2-H	1.5 ⌀	6
2	9	22	Mid+Std	1.75" ⌀	8
1	9	22	Mid+Std	2" ⌀	10


MID+STD CONCEPT DETAILS
A
D
C
B
MIDPLY FIELD NAILS
MIDPLY EDGE NAILS
FLAT STUDS ON EXTERIOR FACE
STD STUDS ON INTERIOR FACE
COMPRESSION STUD-PACK
CONT ROD HOLDOWN
WALL ELEVATION
NTS
45d
SILL PL
BLOCKING
2-ROWS OF INCLINED FULLY-THREADED SELF-TAPPING SCREWS
NOTE: LARGE END STUD PACKS REDUCE THE AVAILABLE LENGTH OF SILL PL CONNECTION. CONSIDER ALTERNATE CONNECTION U/S OF STUD PACKS IF NEEDED.
SCREW CROSS DETAIL
STD PLY SHEATHING ON EXTERIOR FACE
CONT DBL SILL PL
STS SCREW CROSS T&B
WIDE EWP BLOCKING
TOP PL RIPPED TO SUIT
MID PLY SHEATHING
2x VERT SILL PL
BOT PL RIPPED TO SUIT OR OVERSIZED PL IF RC TOPPING IS USED
NAILS TO MATCH EDGE SPACING
2x VERT TOP PL
2x FLAT STUD
A2
FLOOR-TO-FLOOR SECTION (ALTERNATE STS OPTION)
NTS
STD PLY SHEATHING ON EXTERIOR FACE
PL-TO-PL NAILS
CONT DBL SILL PL
WIDE EWP BLOCKING
FRAMING CLIPS E/S
TOP PL RIPPED TO SUIT
2x VERT SILL PL
BOT PL RIPPED TO SUIT OR OVERSIZED PL IF RC TOPPING IS USED
ALTERNATE CONNECTION USING FRAMING CLIPS IN LIEU OF NAILS FOR PREFAB CLOSED-WALL PANELS
NAILS TO MATCH EDGE SPACING
2x VERT TOP PL
2x FLAT STUD
A1
FLOOR-TO-FLOOR SECTION
NTS
MIDPLY EDGE NAILS
NAILS TO MATCH EDGE SPACING
END STUD RIPPED TO SUIT
EXT PLY EDGE NAILS
CONT ROD HOLDOWN
EQ
EQ
D
PLAN DETAIL @ HOLDOWN
NTS
FLAT STUDS FOR FIELD NAILS COULD BE OMITTED IF NOT NEEDED FOR BACKING
MIDPLY FIELD NAILS
OUTLINE OF SILL PL BELOW
MIDPLY EDGE NAILS
FLAT STUDS AT MIDPLY JOINTS
DBL STUDS AT ALL PLY JOINTS
STD PLY EDGE NAILS
RECOMMENDED SWELLING GAP
2x STG STUDS
C
PLAN DETAIL @ PLY JOINTS
NTS
STD PLY SHEATHING ON EXTERIOR FACE
PL-TO-PL NAILS
CONT DBL SILL PL
BOT PL RIPPED TO SUIT
CIP OR POST-INSTALLED FND ANCHORS
2x FLAT STUD
STD PLY SHEATHING ON INTERIOR FACE
2x VERT SILL PL
NAILS TO MATCH EDGE SPACING
B
SILL-TO-FOUNDATION DETAIL
NTS

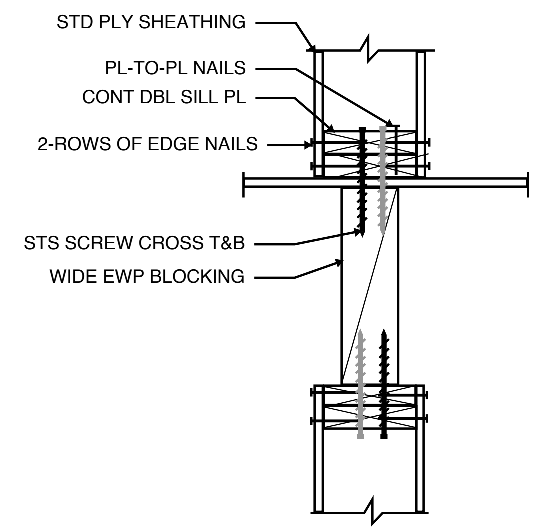
DOUBLE NAIL CONCEPT DETAILS



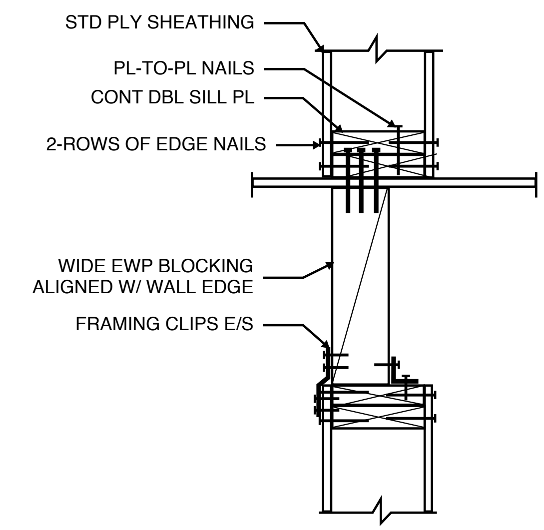
WALL ELEVATION
NTS



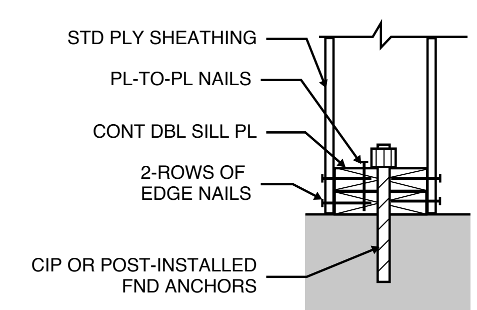
SCREW CROSS DETAIL



A2 **FLOOR-TO-FLOOR SECTION (ALTERNATE STS OPTION)**
NTS



A1 **FLOOR-TO-FLOOR SECTION**
NTS



B **SILL-TO-FOUNDATION DETAIL**
NTS