Chapter 4

Results and Discussion

4.1 Introduction

Results from the parametric study outlined in the previous chapter are presented. These results include the displacement, maximum monotonic loading, and dynamic base shear of the wall, along with the force distribution throughout the wall and joist supporting the wall. These results are utilized to determine the effect that placing a shear wall on-top-of a joist, thereby simulating a flexible foundation, will have on the performance of the wall that is subjected to monotonic or dynamic loading. Ultimately, the results will be utilized as a basis for recommendations for change in the design methodology of timber shear walls.

4.2 Monotonic Loading

Results from the parametric study for the cases when the wall model was subjected to monotonic loading are presented in Tables 4.1 through 4.3. A few typical load-displacement curves are presented in Figure 4.1, with the remainder of the load-displacement curves included in Appendix A. Results indicate a shear wall on an elastic foundation subjected to monotonic loading will exhibit the following characteristics:

Table 4.1: Results from the Parametric Study for Walls Subjected to Monotonic Loading with 12.7 mm (0.5 in) Anchorage Connection

Deflection Criteria for Joist	Displacement[1]		Racking Displacement	Maximum Resistance
	Lateral	Vertical		
	mm (in)	mm (in)	mm (in)	kN (kips)
L/180	55.7 (2.19)	-52.1 (-2.05)	21.0 (0.83)	28.8 (6.48)
L/240	47.2 (1.86)	-40.7 (-1.60)	20.1 (0.79)	35.2 (7.92)
L/300	40.9 (1.61)	-32.2 (-1.27)	19.4 (0.76)	28.8 (6.48)
L/330	39.5 (1.56)	-30.2 (-1.19)	19.4 (0.76)	28.8 (6.48)
L/360	38.0 (1.50)	-28.2 (-1.11)	19.2 (0.76)	28.8 (6.48)
L/420	35.1 (1.38)	-24.2 (-0.95)	19.0 (0.75)	38.4 (8.64)
L/480	33.0 (1.30)	-21.4 (-0.84)	18.8 (0.74)	38.4 (8.64)
L/510	32.1 (1.26)	-20.1 (-0.79)	18.7 (0.74)	35.2 (7.92)
L/540	31.4 (1.24)	-19.2 (-0.76)	18.6 (0.73)	35.2 (7.92)
L/600	30.3 (1.19)	-17.6 (-0.69)	18.6 (0.73)	32.0 (7.20)
L/720	28.4 (1.12)	-14.9 (-0.59)	18.4 (0.73)	32.0 (7.20)
RIGID	15.6 (0.62)	-0.5 (-0.02)	15.3 (0.60)	54.5 (12.24)

[1] - Displacements are taken at a load of 28.8 kN (6.48 kips).

Table 4.2: Results from the Parametric Study for Walls Subjected to Monotonic Loading with 25.4 mm (1 in) Anchorage Connection

Deflection Criteria for Joist	Displacement[1]		Racking Displacement	Maximum Resistance
	Lateral	Vertical		
	mm (in)	mm (in)	mm (in)	kN (kips)
L/180	55.4 (2.18)	-51.8 (-2.04)	20.9 (0.82)	28.8 (6.48)
L/240	47.0 (1.85)	-40.5 (-1.59)	20.0 (0.79)	32.0 (7.20)
L/300	40.8 (1.61)	-32.2 (-1.27)	19.4 (0.76)	35.2 (7.92)
L/330	39.2 (1.54)	-30.0 (-1.18)	19.2 (0.76)	28.8 (6.48)
L/360	37.7 (1.48)	-28.0 (-1.10)	19.1 (0.75)	28.8 (6.48)
L/420	34.8 (1.37)	-24.0 (-0.94)	18.8 (0.74)	38.4 (8.64)
L/480	32.7 (1.29)	-21.1 (-0.83)	18.6 (0.73)	38.4 (8.64)
L/510	31.8 (1.25)	-19.8 (-0.78)	18.5 (0.73)	38.4 (8.64)
L/540	31.1 (1.22)	-19.0 (-0.75)	18.5 (0.73)	38.4 (8.64)
L/600	30.0 (1.18)	-17.4 (-0.68)	18.4 (0.72)	35.2 (7.92)
L/720	28.0 (1.10)	-14.7 (-0.58)	18.2 (0.72)	41.6 (9.36)
RIGID	15.6 (0.61)	-0.4 (-0.02)	15.3 (0.60)	54.5 (12.24)

[1] - Displacements are taken at a load of 28.8 kN (6.48 kips).

Table 4.3: Results from the Parametric Study for Walls Subjected to Monotonic Loading with Rigid Anchorage Connection

Deflection Criteria for Joist	Displacement[1]		Racking Displacement	Maximum Resistance
	Lateral	Vertical		
	mm (in)	mm (in)	mm (in)	kN (kips)
L/180	55.3 (2.18)	-51.7 (-2.04)	20.8 (0.82)	28.8 (6.48)
L/240	46.9 (1.85)	-40.4 (-1.59)	20.0 (0.79)	32.0 (7.20)
L/300	41.2 (1.62)	-32.4 (-1.28)	19.5 (0.77)	35.2 (7.92)
L/330	39.1 (1.54)	-29.9 (-1.18)	19.2 (0.76)	28.8 (6.48)
L/360	37.6 (1.48)	-27.9 (-1.10)	19.0 (0.75)	28.8 (6.48)
L/420	34.7 (1.37)	-23.9 (-0.94)	18.8 (0.74)	38.4 (8.64)
L/480	32.6 (1.28)	-21.0 (-0.83)	18.6 (0.73)	38.4 (8.64)
L/510	31.7 (1.25)	-19.8 (-0.78)	18.5 (0.73)	38.4 (8.64)
L/540	31.0 (1.22)	-18.9 (-0.74)	18.4 (0.72)	38.4 (8.64)
L/600	29.8 (1.18)	-17.3 (-0.68)	18.3 (0.72)	35.2 (7.92)
L/720	27.9 (1.10)	-14.6 (-0.57)	18.2 (0.72)	41.6 (9.36)
RIGID	15.6 (0.61)	-0.4 (-0.02)	15.3 (0.60)	54.5 (12.24)

[1] - Displacements are taken at a load of 28.8 kN (6.48 kips).

1.) Lateral displacement increases as joist stiffness decreases

2.) The performance of the joist could be the controlling factor when determining the allowable resistance of the shear wall system

3.) The percentage of racking displacement increases as the joist stiffness increases

4.) Anchorage size has a minimal effect on the response of a wall as long as slip does not occur between the hold-down connector and the stud.

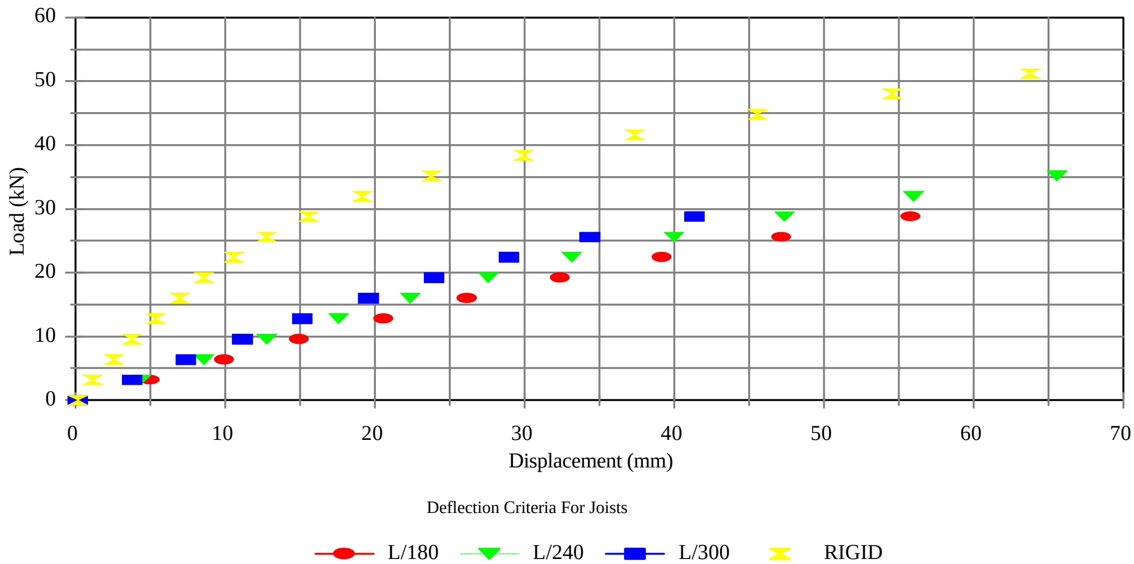

Figure 4.1: Typical Load Versus Lateral Displacement Curve

4.2.1 Displacement

Displacements listed in Tables 4.1 through 4.3 were recorded at the top corner of the end of the wall model which a location where maximum lateral displacement of the wall would occur. This position corresponds to the worst case scenario for displacements. Lateral displacements are compared when a load of 28.8 kN (6.48 kips) was applied for all of the wall models, although some of the models resisted larger loads and thus experienced larger maximum displacements. This was done because 28.8 kN (6.48 kips) was the lowest maximum loading recorded for some of the foundation conditions and also the load at which the deflection at the center of the joist were becoming excessive. Results of the parametric study indicate that the lateral displacement at the top of a shear wall increases as the joist stiffness decreases, as is shown in Figure 4.2. This occurs because as stiffness of the joist decreases so does the stiffness of the system, therefore the system is more susceptible to deformation. Lateral displacement of the wall model resting on a rigid foundation is significantly lower than that of a wall resting on the stiffest joist (i.e., joist with deflection criterion of L/720), indicating that the rigid wall allows very little lateral movement within a given shear wall.

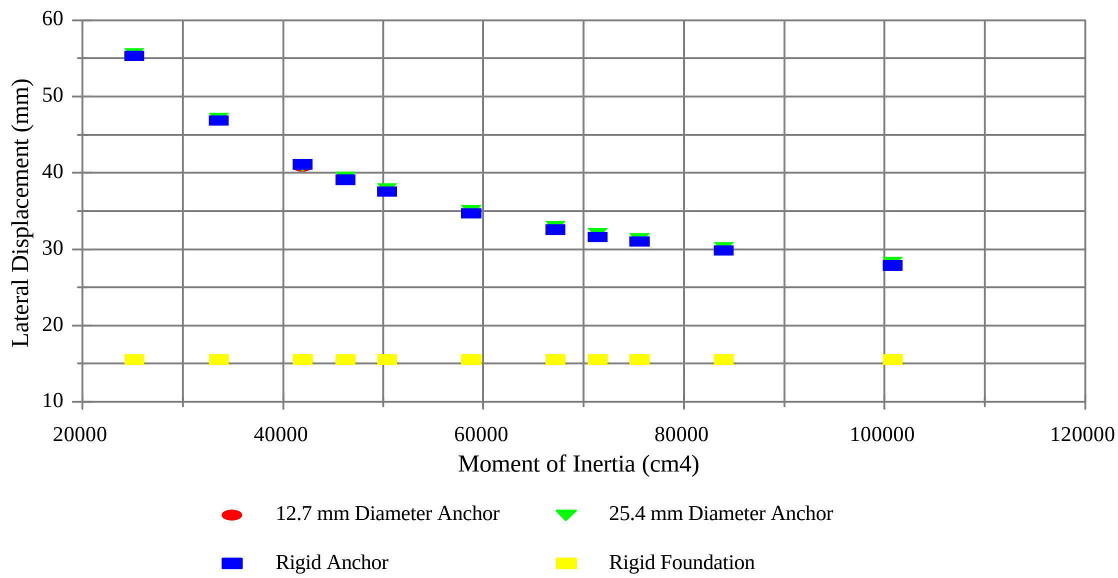

Figure 4.2: Lateral Displacement Versus Joist Stiffness

Therefore, the walls on a non-rigid foundation would be more susceptible to cracking and architectural damage than a wall on a rigid foundation. The size of the hold-down had only a minute effect on the total lateral displacement.

Total lateral displacement of a wall, however, is a combination of wall rotation and shear deformation of the wall model. Shear deformation was calculated by subtracting the combination of the height to length ratio (8/12 or 2/3) of the wall multiplied by the vertical displacement from the total lateral displacement. Racking displacement in a wall is important for determining the overall performance of the wall because it is an indication of how much deformation occurs to the wall as opposed to the foundation. The racking displacement increased with a decrease in joist stiffness (see Figure 4.3), although the range of racking displacements within the flexible foundation models was 18.2-21 mm (0.7-0.8 in). The increase in racking displacement as the joist stiffness decreases occurs

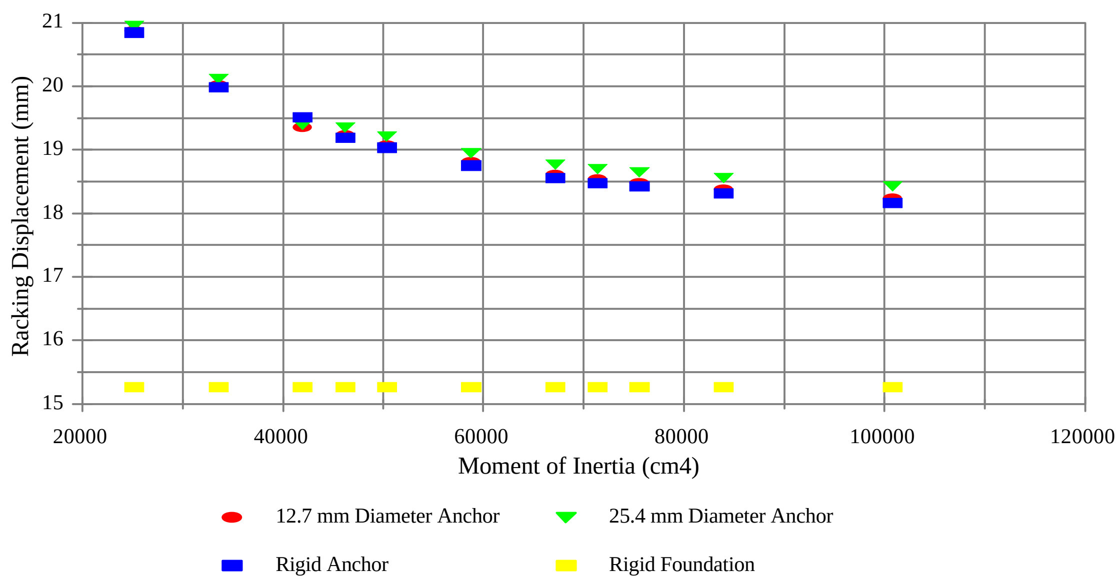

Figure 4.3: Racking Displacement Versus Joist Stiffness

because the lower stiffness makes it easier to rack the wall. However, the percentage of racking displacement versus the joist stiffness shows a different trend than the amount of racking displacement. The percentage of racking displacement is the ratio of the racking displacement versus the total lateral displacement of a wall. The percentage of racking displacement is an indicator of how much, percentage wise, a wall has rotated and how much it has deformed due to rotation. The percentage of racking displacement (Figure 4.4) for the flexible foundation models ranges from approximately 38% for the walls with a joist stiffness of 25182 cm^4 (605 in^4) up to approximately 65% with an joist stiffness of 100786 cm^4 (2421 in^4). Rigid foundation models show a 100% racking displacement and no rotation. The increase in percentage of shear deformation as joist stiffness increases indicates that more racking of the wall, as opposed to rigid body rotation, occurs.

The effect of anchorage conditions on the lateral displacement of the walls was also examined in the parametric study (see Tables 4.1-4.3). The effect of anchorage conditions was relatively minimal overall, but lateral displacement at the top of the wall was found to increase slightly as the anchorage connection stiffness decreased. The trend was constant for all eleven flexible foundation models as well as the rigid foundation model. The trend was expected because the stiffer a connection between two members (i.e. the shear wall and the foundation), the lower the displacement between the two members, and therefore, the lower the displacement in the system. Furthermore, lateral displacement of the wall was found to increase as the combination of joist stiffness and anchorage stiffness decreased. Thus the wall resting on the joist with stiffness of 25182 cm^4 (605 in^4), and connected to the joist with a 12.7 mm (0.5 in) diameter anchor bolt, displayed the greatest lateral displacement, while the wall resting on the rigid foundation and connected to the foundation with a

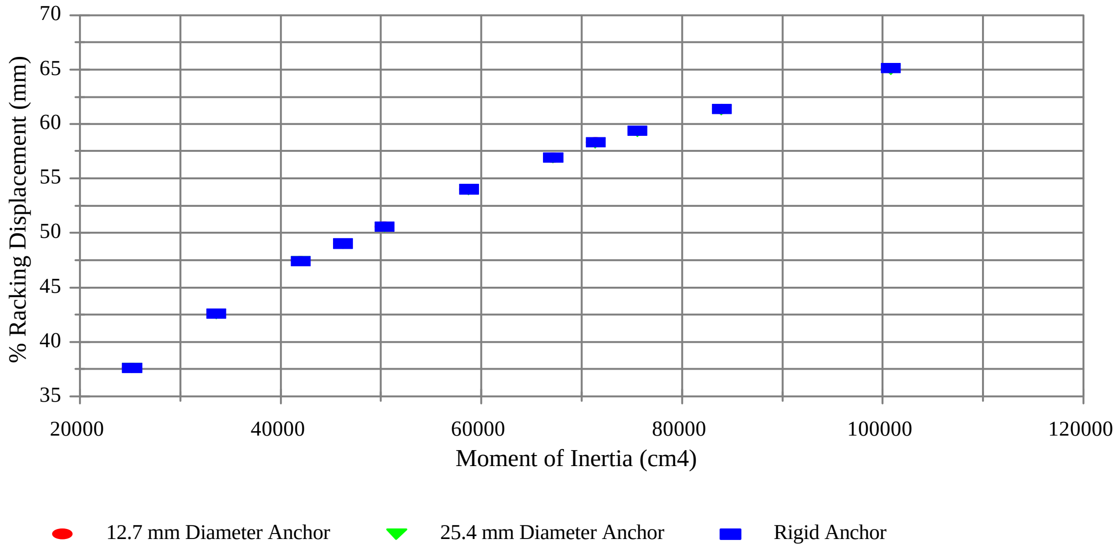

Figure 4.4: Percent Racking displacement Versus Joist Stiffness

rigid anchorage connection showed the least lateral displacement. Anchorage connection stiffness had only a minimal effect on the displacement of the wall.

Vertical displacements for each of the three anchorage conditions versus joist stiffness are shown in Figure 4.5. Vertical displacement increases as the joist stiffness decreases. This is to be expected because in theory, a resisting force-couple is formed in a shear wall with the forces at the chords. Given that walls are subjected to the same loads, the vertical force at the chord located at the center of the wall would be identical for each foundation condition, and therefore the displacement would increase as the stiffness of the joist decreases. Displacement is similar to the displacement calculated using the equation for deflection of a beam at its center point subjected to a concentrated load at its center point (see Figure 4.5). The parametric study did reveal an area of concern. The joist stiffness was calculated using a set of deflection criteria as defined earlier but upon analyzing the walls, it was found that all of the wall models deflected more than the deflection criteria allowed (Figure 4.5) before failure of the wall model occurred. Allowable deflections were from 68 to 78 percent of the

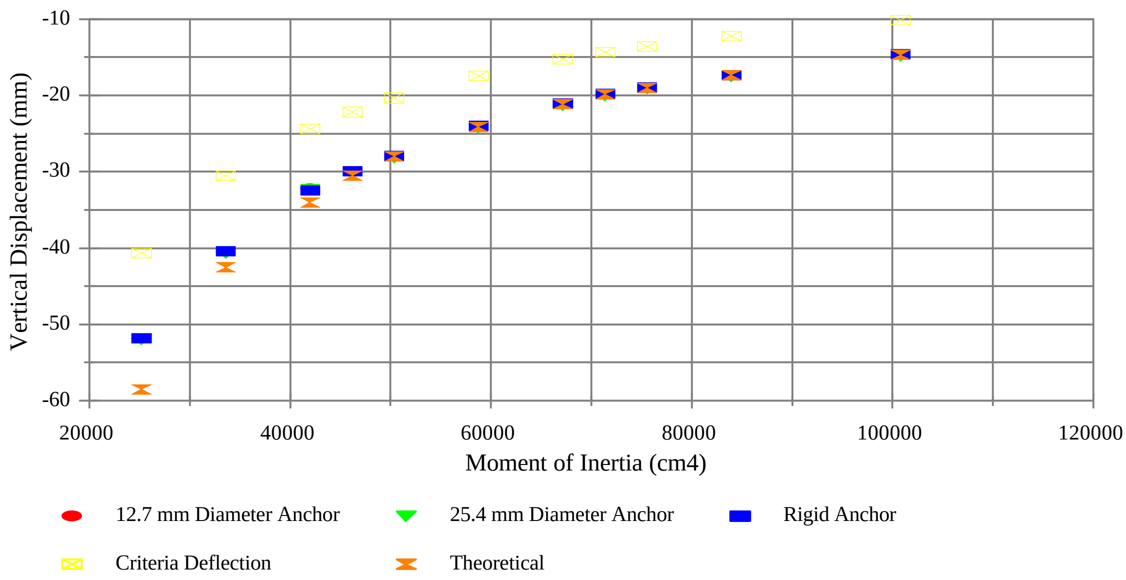

Figure 4.5: Vertical Displacement Versus Moment of Inertia

deflections calculated using WALSEIZ1. The deflections from the analysis were closer for the lower joist stiffness than for higher joist stiffness. Although the deflections do exceed the maximum allowable deflections according to the code, WALSEIZ1 is designed to test the walls until failure and deflection criteria for serviceability do not to reflect the maximum deflection a wall can resist before failing. This indicates that the joist supporting the wall, as opposed to the wall itself, may be the critical factor in the design of the system.

4.2.2 Maximum Loading

Maximum loading was calculated by taking the uniformly distributed load increment of 72.91 N/m (5 lb/ft), multiplying it by the wall length of 3.66 m (12 ft), and then multiplying that value by the number of load increments. Results of the parametric study do not show a trend between maximum loading, lateral displacements, and joist stiffness. However, as stated earlier, the model's joist deflection exceeded the deflection criteria for all of the joist sizes at 28.8 kN (6480 lbs). Therefore, the joist stiffness could be the controlling factor in the design of shear walls.

4.2.3 Internal Forces

Distribution of axial forces will be examined for the top plate, studs, bottom plate, and joist supporting the wall. The majority of axial and shear load in the framing and sheathing are created by the connectors. As stated previously, when a shear wall is subjected to a lateral load, the framing members will distort as a parallelogram while the sheathing panels will maintain a rectangular shape and rotate. This displacement pattern results in the sheathing connectors being displaced, thus transferring axial forces into the studs and plates. The top plate experiences a constant distributed axial load from the applied force and a constant, distributed axial load from the sheathing connectors. Framing connectors provide resistance to the axial force and when analyzing this system the resistance can be thought of as a reaction. The loading results in a maximum load near the ends of the top plate, with the loads generally decreasing toward the center of the top plate. At each point where the stud and top plate are joined, there is a "jump" in the force diagram, similar to having a concentrated load at that point. Force distribution along a typical top plate is shown in Figure 4.6.

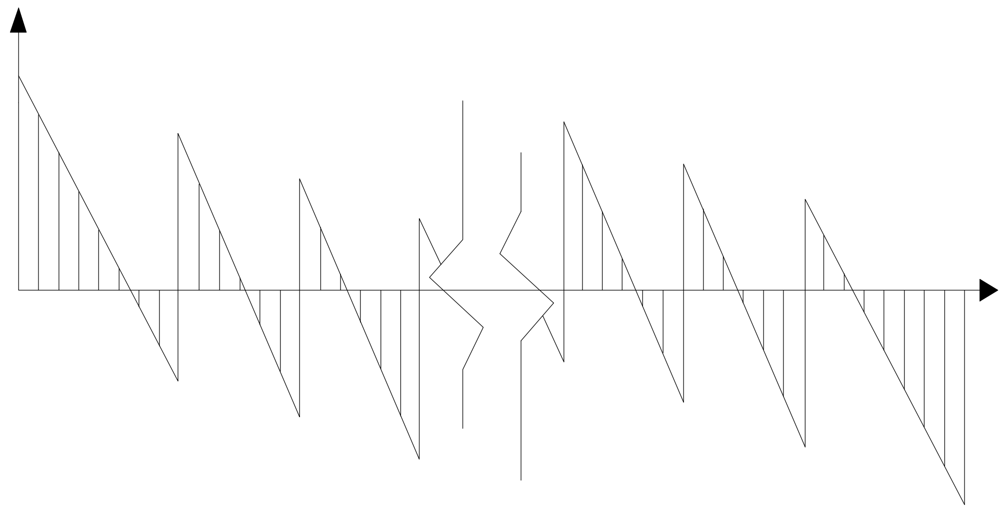

Figure 4.6: Axial Force Distribution Along a Typical Top Plate

Studs/chords are subjected to a constant, distributed axial load from the connectors, and possibly a concentrated axial load transferred from the top plate. Studs and chords, when in compression, experience large axial loads at the base of the wall and small axial loads at the top of the wall (Figure 4.7). Studs in tension experience a relatively low, uniform loading throughout the member. The hold-downs provide a resistance to tensile loading at the chords, while bearing between the studs (and/or chords) and the bottom plate provides a resistance to compressive loading. This occurs because the loads are distributed from the source, in this case a distributed load along the top of the wall, and are transferred down to the foundation, adding up as they progress downwards. The studs are more heavily loaded in compression or tension at the ends while the studs experience lower internal forces as they approach the middle of the wall.

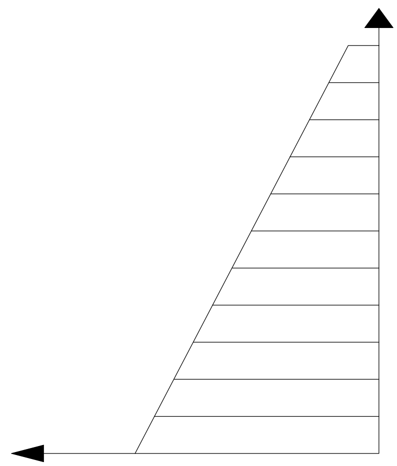

Figure 4.7: Axial Force Distribution Along a Typical Stud

Forces applied to the bottom plate are similar to those of the top plate with the exception of the applied axial load. Forces in the bottom plate exhibit similar behavior as the top plate in that the forces are the greatest at the ends with the maximum force occurring over the end of the wall that is at the end of the joist (Figure 4.8).

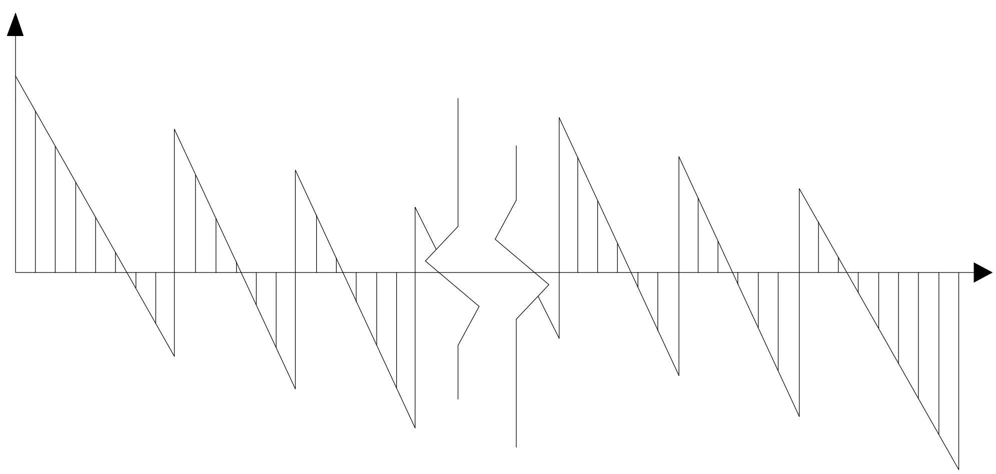

Figure 4.8: Axial Force Distribution Along a Typical Bottom Plate

A typical joist from the shear wall models receives load from the anchor bolts, connecting the joist to the bottom plate, and experiences the highest axial forces in the half of the joist after the wall ends and this force is approximately the same at every node along the twelve feet length. The half of the joist with the wall resting on it experiences the greatest axial forces at the ends of the wall. The forces decrease as the joist approaches the center of the wall and increase as the joist approaches the opposite end of the wall. Figure 4.9 shows the distribution of forces across a typical joist.

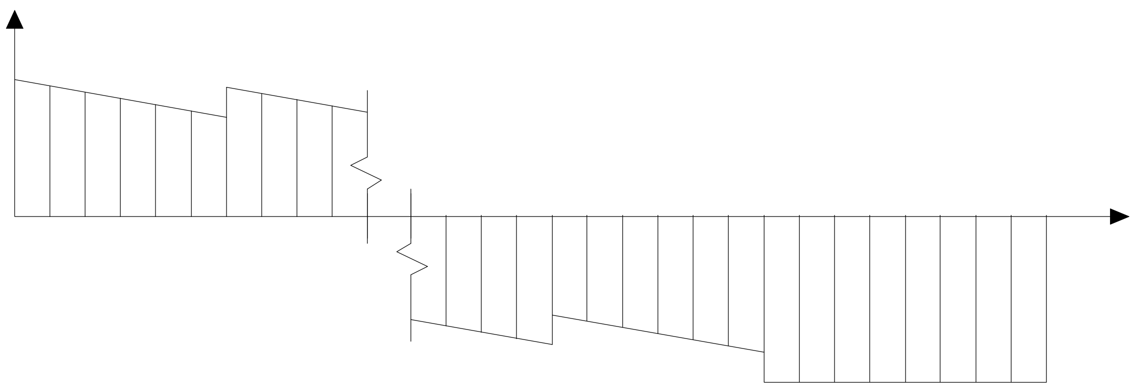

Figure 4.9: Distribution of Axial Force Along the Full Length of a Typical Joist

4.3 Dynamic Loading

Results from the parametric study for the cases when the wall was subjected to seismic loading are presented in Tables 4.4 through 4.6. A few typical time displacement curves are presented in Figure 4.10, with the remainder of the time displacement curves included in Appendix B. Results indicate shear walls on an elastic foundation subjected to dynamic loading will have the following performance:

1.) Maximum base shear and racking displacement occur when a wall is on a rigid foundation

2.) Displacements generally increase as stiffness of the foundation decreases

3.) A localized peak occurs for lateral displacement and base shear, which would indicate that a resonant situation is occurring

Table 4.4: Results from the Parametric Study for Walls Subjected to Dynamic Loading with 12.7 mm (0.5") Diameter Anchorage Connection

Deflection Criteria for Joist	Displacement		Racking Displacement	Maximum Base Shear
	Lateral	Vertical		
	mm (in)	mm (in)	mm (in)	kN (kips)
L/180	114.4 (4.50)	134.5 (5.29)	24.8 (0.98)	34.2 (7.70)
L/240	120.5 (4.74)	133.3 (5.25)	31.6 (1.24)	39.6 (8.91)
L/300	123.4 (4.86)	127.3 (5.01)	38.6 (1.52)	47.5 (10.70)
L/330	125.5 (4.94)	122.8 (4.83)	43.7 (1.72)	52.3 (11.77)
L/360	126.2 (4.97)	117.5 (4.63)	47.9 (1.88)	55.1 (12.40)
L/420	124.0 (4.88)	109.5 (4.31)	51.1 (2.01)	57.6 (12.95)
L/480	117.8 (4.64)	103.9 (4.09)	48.6 (1.91)	55.7 (12.52)
L/510	113.7 (4.48)	100.7 (3.97)	46.6 (1.83)	54.7 (12.32)
L/540	112.6 (4.43)	98.6 (3.88)	46.8 (1.84)	53.1 (11.95)
L/600	109.8 (4.32)	94.1 (3.7)	47.0 (1.85)	54.9 (12.36)
L/720	113.3 (4.46)	83.6 (3.29)	57.6 (2.27)	58.2 (13.11)
RIGID	78.3 (3.08)	2.1 (0.08)	76.9 (3.03)	64.2 (14.44)

Table 4.5: Results from the Parametric Study for Walls Subjected to Dynamic Loading with 25.4 mm (1") Diameter Anchorage Connection

Deflection Criteria for Joist	Displacement		Racking Displacement	Maximum Base Shear
	Lateral	Vertical		
	mm (in)	mm (in)	mm (in)	kN (kips)
L/180	113.6 (4.47)	133.5 (5.26)	24.6 (0.97)	34.7 (7.80)
L/240	118.8 (4.68)	129.6 (5.10)	32.4 (1.28)	40.0 (8.99)
L/300	121.8 (4.79)	123.7 (4.87)	39.3 (1.55)	47.7 (10.74)
L/330	124.1 (4.89)	119.2 (4.69)	44.6 (1.76)	52.4 (11.79)
L/360	124.8 (4.92)	114.2 (4.49)	48.8 (1.92)	55.8 (12.54)
L/420	122.1 (4.81)	106.8 (4.21)	50.8 (2.00)	57.1 (12.84)
L/480	115.6 (4.55)	102.1 (4.02)	47.6 (1.87)	56.5 (12.71)
L/510	112.2 (4.42)	99.5 (3.92)	45.9 (1.80)	54.9 (12.36)
L/540	110.3 (4.34)	97.5 (3.84)	42.3 (1.78)	53.5 (12.04)
L/600	107.7 (4.24)	93.4 (3.68)	45.4 (1.79)	53.5 (12.04)
L/720	111.1 (4.37)	82.7 (3.26)	56.0 (2.20)	57.1 (12.84)
RIGID	77.4 (3.05)	1.8 (0.07)	76.2 (3.00)	64.1 (14.42)

Table 4.6: Results from the Parametric Study for Walls Subjected to Dynamic Loading with Rigid Anchorage Connection

Deflection Criteria for Joist	Displacement		Racking Displacement	Maximum Base Shear
	Lateral	Vertical		
	mm (in)	mm (in)	mm (in)	kN (kips)
L/180	113.4 (4.46)	132.9 (5.23)	24.8 (0.98)	34.9 (7.84)
L/240	118.8 (4.68)	129.1 (5.08)	32.7 (1.29)	40.1 (9.03)
L/300	121.2 (4.77)	123.0 (4.84)	39.3 (1.55)	47.8 (10.76)
L/330	123.4 (4.86)	118.4 (4.66)	44.5 (1.75)	52.6 (11.83)
L/360	124.1 (4.89)	113.0 (4.45)	48.8 (1.92)	55.4 (12.46)
L/420	121.3 (4.78)	106.1 (4.18)	50.6 (1.99)	57.1 (12.85)
L/480	114.6 (4.51)	101.5 (4.00)	46.9 (1.85)	56.2 (12.65)
L/510	111.5 (4.39)	99.1 (3.90)	45.5 (1.79)	54.6 (12.28)
L/540	109.8 (4.32)	97.1 (3.82)	45.1 (1.77)	53.4 (12.01)
L/600	107.6 (4.24)	93.3 (3.67)	45.4 (1.79)	53.3 (11.99)
L/720	110.6 (4.35)	82.3 (3.24)	55.7 (2.19)	57.3 (12.90)
RIGID	77.1 (3.04)	1.7 (0.07)	76.0 (2.99)	64.4 (14.48)

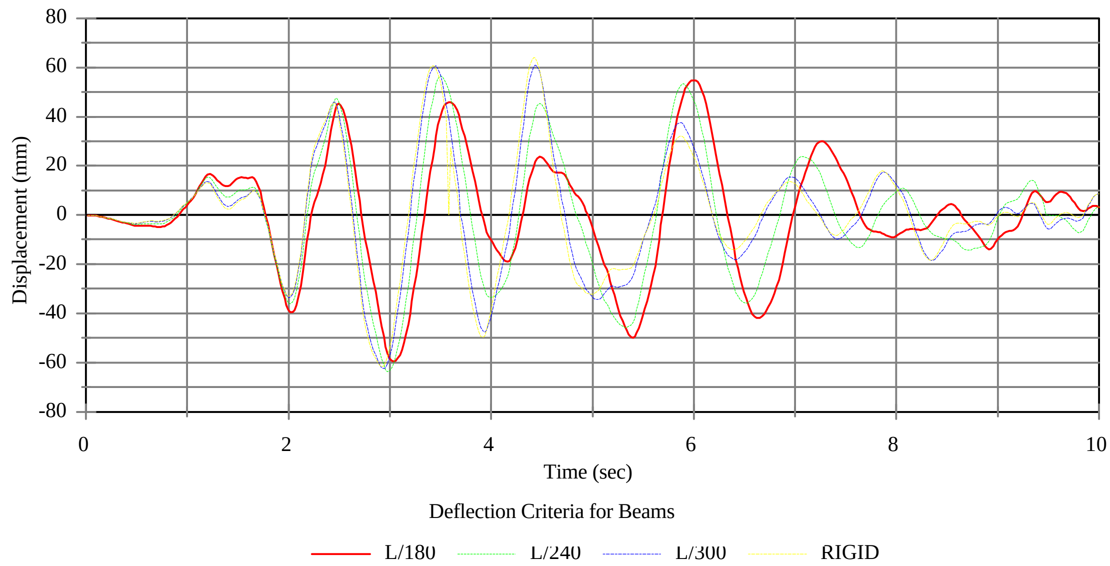

Figure 4.10: Typical Time Versus Lateral Displacement Curve for Wall Models Subjected to Dynamic Loading

4.3.1 Maximum Base Shear

A plot of the maximum base shear, shown in Figure 4.11, indicates that a localized peak shear force occurs when the joist has a deflection criterion of L/420, and the largest base shear occurs for the wall on a rigid foundation. Initially, the maximum base shear increased as the stiffness of the joist increased, which was anticipated given the assumption that the greater the stiffness of the system, the larger force it would attract. However, for joists with a deflection criterion greater than L/420, the base shear decreases until deflection criterion L/600 is reached, at which point the base shear increases as the joist stiffness is further increased (seemingly until the foundation becomes rigid). This pattern would indicate that resonance occurs for the joist with a deflection criterion of L/420 and that once the stiffness associated with a deflection criterion of L/600 is exceeded, the base shear would generally increase.

Hold-down size has very little effect on the magnitude of base shear. In models with the lower joist stiffness, the base shear increased slightly with an increase in anchorage stiffness, while in the models with moderate joist stiffness the effect of the anchorage connections did not follow a set pattern. The last two stiffest flexible foundation models showed a slight decrease in base shear with an increase in anchorage conditions (Tables 4.4-4.6).

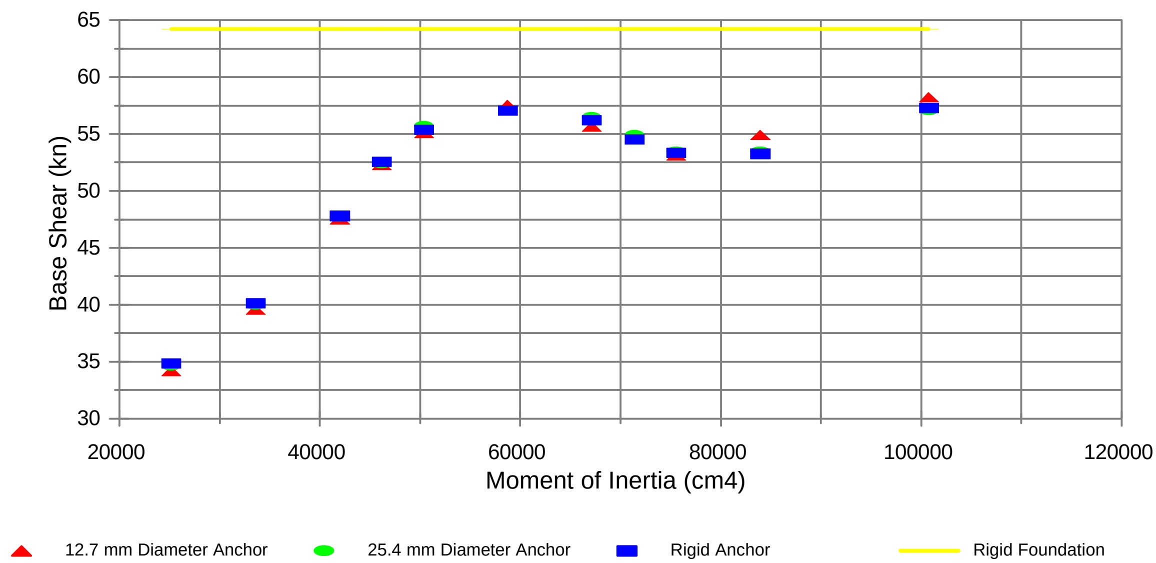

Figure 4.11: Maximum Base Shear Versus Moment of Inertia

4.3.2 Displacements

Results of the parametric study, shown in Figure 4.12, indicate the following: maximum displacement occurs for the wall model resting on a joist with a deflection criteria of L/360, lateral displacement for walls on a non-rigid foundation are greater than that for walls on a rigid foundation, and size of the hold-down has little to no effect on the displacement provided it does not yield or slip with respect to the stud. Lateral displacement of walls increases initially as joist size increases and then decreases, only to increase again and finally have a minimum value for the rigid foundation models. The lateral displacement for flexible foundation wall models increases based upon deflection criteria as follows: L/600 is the lowest and then L/540, L/720, L/510, L/180, L/480, L/240, L/300,

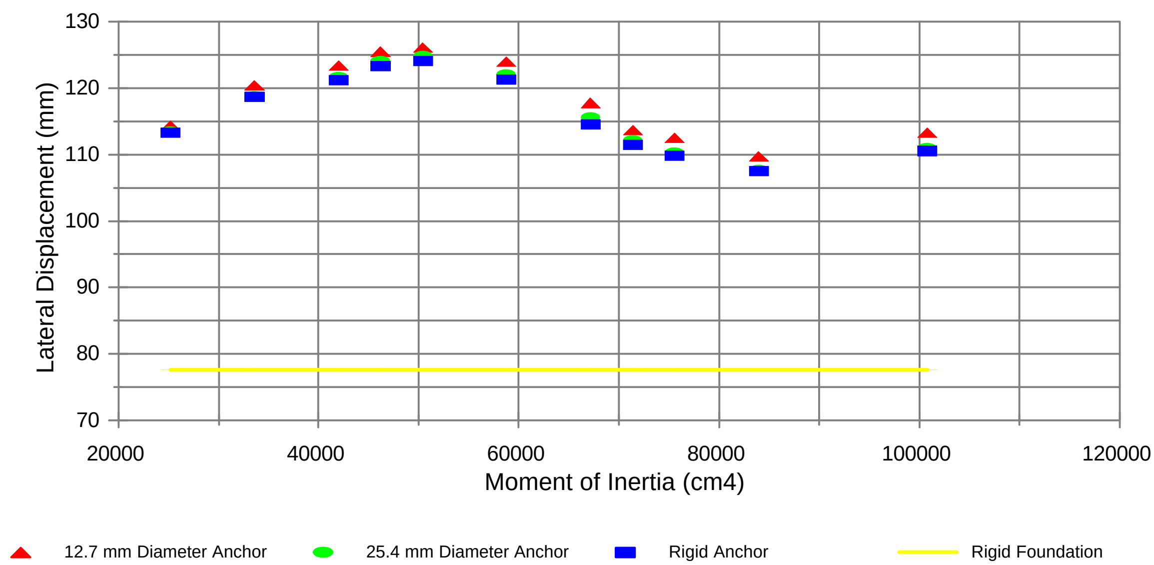

Figure 4.12: Maximum Lateral Displacement Versus Moment of Inertia

L/420, L/330, L/360. This pattern is the same for all three anchorage conditions. The rigid foundation model has the lowest lateral displacement. As with the monotonic loading models, the rigid foundation model has the greatest stiffness for the entire system and no potential for rotation, therefore, it has the lowest lateral displacement. Results for the flexible foundation models did not follow the assumed pattern. In theory, the stiffer joist system should deform less laterally, but one combination of stiffness results in a system frequency that is close to the forcing frequency, resulting in resonance. The maximum base shear, which also varies as a function of joist stiffness (see Figure 4.11), also affects the maximum displacement because, for a given system, larger forces result in greater displacement.

As with monotonic loading, lateral displacement of the wall occurs through two mechanisms: wall rotation and shear deformation. Racking displacement is calculated in the same fashion as it was for monotonic loading. The maximum racking displacement shown in Figure 4.13, has a trend similar to that of maximum lateral displacements. Racking displacements show an increase with an initial increase in joist stiffness and then a decrease in racking displacement with continued increase in joist stiffness. Finally the racking displacements show an increase in racking displacement with a further increase in joist stiffness. Rigid foundation models display the greatest racking displacement. The percentage of racking displacement was also calculated, and in general showed a trend of increasing as joist stiffness increases (similar to the trend for monotonic loading) as shown in Figure 4.14. The percentage of racking displacement ranged from 22% for foundations with the lowest joist stiffness up to 50% for foundations with the greatest joist stiffness. The rigid foundation models experienced racking displacements of 100% which means that no rigid body displacement occurred for the walls. This seems to indicate that resonance is occurring for the wall supported by a joist of depth 429 mm (16.9 in). Although changing parameters (nail spacing, wall length, acceleration record, etc.) would alter the specific condition at which resonance would occur, it seems as if a pattern similar to the one shown in Figure 4.13 would occur. Although the wall on a rigid foundation has more shear deformation, and consequently more strain in the wall, the total displacement is greater for walls on a non-rigid foundation.

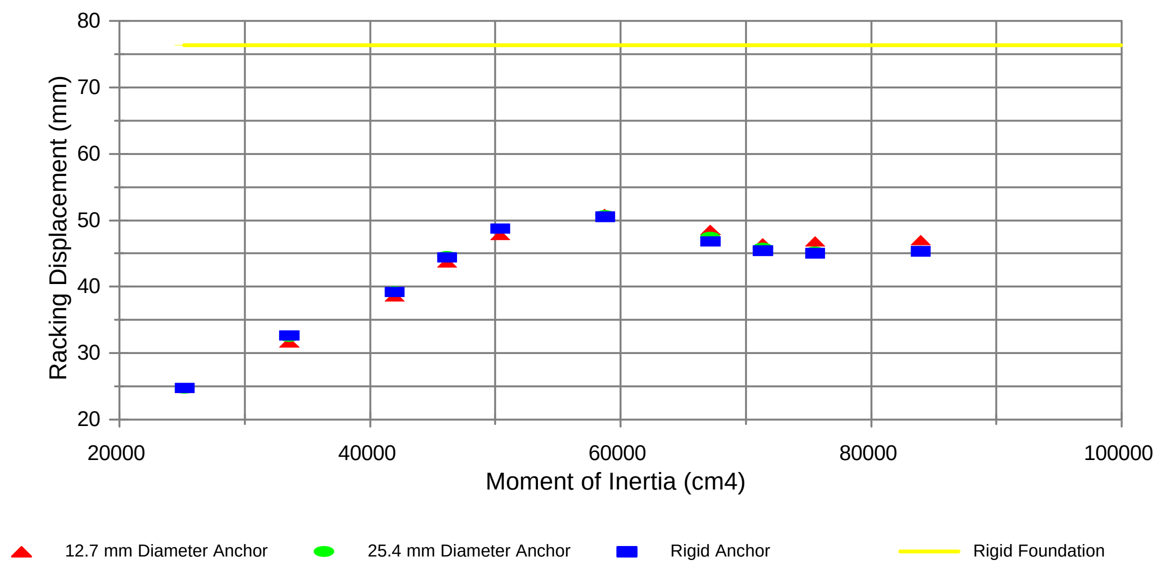

Figure 4.13: Maximum Racking displacement Versus Moment of Inertia

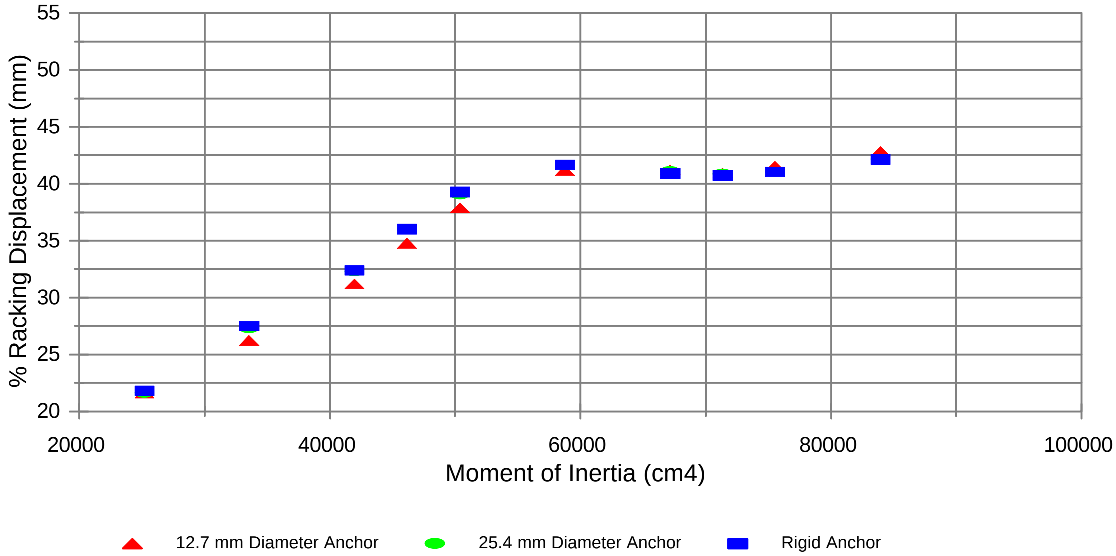

Figure 4.14: Percent Racking displacement Versus Moment of Inertia

Analysis of the vertical displacements showed that vertical displacement increased as joist stiffness and anchorage connection stiffness decreased (Figure 4.15). Deflections followed the trend that was expected with different foundation conditions. The more flexible a wall and joist system is the greater it will displace vertically. Results from the parametric study indicated that the maximum vertical displacements were greater than the deflection criteria used to define the joist stiffness. The ratios of the allowable displacements versus analytical displacements ranged from 0.2 for the models with higher joist stiffness to 0.75 for the models with lower joist stiffness. With the deflection criteria for the joists being exceeded before the walls failed, again indicates that for this system, the joist size would be a critical design consideration. Exceedence of the deflection limits is important for future acceptance of foundation systems.

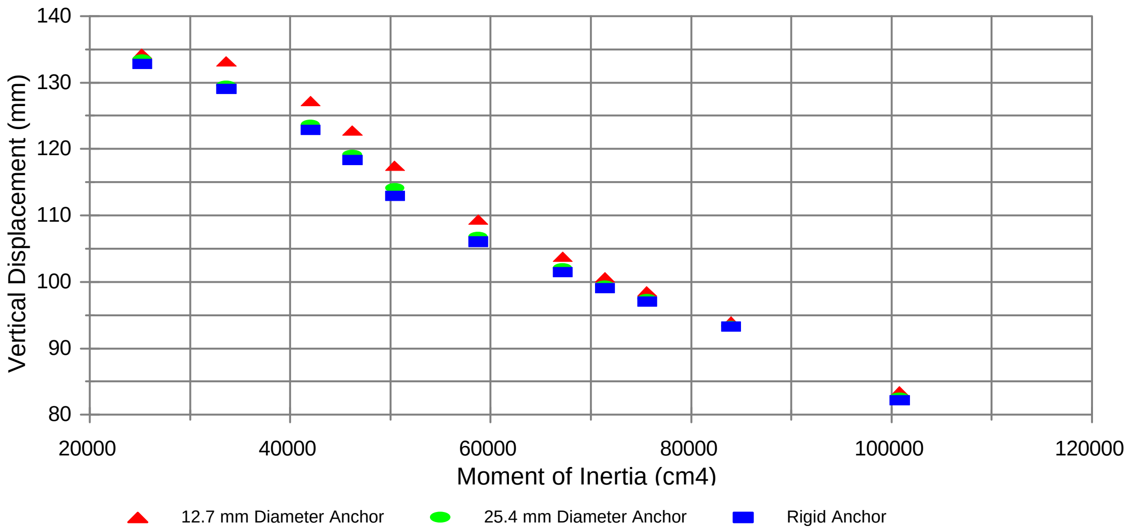

Figure 4.15: Maximum Vertical Displacement of a Wall Model Versus Joist Stiffness

4.4 Conclusion

Results from the parametric study outlined in Chapter 3 were presented. The parametric study revealed the following trends for shear walls on a non-rigid foundation subjected to monotonic loading: 1.) Displacements increase as joist stiffness decreases; 2.) displacements increase with a decrease in stiffness of anchorage connections (although this effect was relatively minor).

The parametric study revealed the following trends for shear walls subjected to dynamic loading: 1.) Vertical displacements increase as joist stiffness decreases. 2.) Lateral and racking displacements increase initially as joist stiffness increases, and then decreases, only to increase again and finally have a minimum value for the rigid foundation model; 3.) A similar trend was also evident for the maximum base shear. This would seem to indicate the presence of a resonance effect for the wall supported by a joist with a depth of 429 mm (16.9 in), for these particular wall configurations.

Chapter 5

Summary and Future Research

5.1 Introduction

Timber shear walls form part of the lateral force resisting systems in many low-rise structures in North America. Shear walls, which typically consist of a structural-use sheathing panel such as plywood or OSB attached to a light-frame consisting of dimension lumber with dowel-type fasteners such as nails or screws, are often supported by a relatively rigid foundation, such as concrete block, along its entire length. However, there are situations when a wall is supported by a relatively flexible foundation, such as a floor joist. Although a nonrigid foundation would have an effect on the response of a shear wall, there has been no research which focused on this phenomenon. Therefore, a study was performed to determine the effect of non-rigid foundations on the response of a timber shear walls.

5.2 Research Method

Research was performed by creating a numerical wall model, varying support stiffness at the base of the model, subjecting the model to monotonic and dynamic loading for each support condition, and utilizing the finite element program WALSEIZ1 to analyze the system. The program is a modification of the program WALSEIZ, which was developed by White and Dolan (1995). The program has elements corresponding to the following items associated with a shear wall: framing, sheathing, connectors between the sheathing and framing and between framing members, and bearing between adjacent sheathing panels. Framing elements are modeled using a 6 degree-of-freedom beam element while the sheathing element is modeled with a four node plane rectangular bilinear element. Connector and bearing elements are modeled using springs. WALSEIZ1 is capable of analyzing wall models with either a rigid or flexible foundation, subjected to monotonic or dynamic loading, calculating the displacement at the sheathing and framing nodes, forces and stresses in each element, stresses in the members, accelerations at the nodes for a dynamic loading condition, and maximum loading for a monotonic loading condition.

Shear wall models utilized in the study correspond to a 2.4 m (8 ft) high by 3.7 m (12 ft) long wall with Canadian Softwood Plywood attached to Spruce-Pine-Fir framing with 8d common nails. Studs were spaced 41 mm (16 in) on-center, and connectors attaching the sheathing to the framing were spaced 10 mm (4 in) along the perimeter of a sheathing panel and 15 mm (6 in) along the interior of a sheathing panel. The wall model was supported by a total of twelve different foundation conditions, one rigid foundation and eleven flexible foundations, along with three different types of anchorage connections at the ends of the wall for a total of thirty-six support conditions. Flexible foundations consisted of joists of varying depths and a constant width of 89 mm (3.5 in). The length of the joist was 7.2 m (24 ft) long and was supported at both ends as a simple beam. The wall model was parallel to the joist, with one chord at the edge of the joist and the other chord at the center of the joist. There were three hold-down connections used for the study: a 12.7 mm (0.5 in) diameter anchor bolt, a 25.4 mm (1 in) diameter anchor bolt, and a rigid hold-down tied to the sill plate. The wall model was subjected to both in-plane monotonic loading and the acceleration record for the S00E component from the 1940 El Centro Earthquake for each support condition.

5.3 Results and Discussion

Results from the parametric study indicated the following concerning shear walls supported by a nonrigid foundation subjected to monotonic loading: 1.) Lateral, vertical, and racking displacements increase as the joist stiffness decreases because of reduced stiffness of the system; 2.) Displacements also increased with a decrease in anchorage connection stiffness, although the effect was minimal; 3.) The percentage of racking displacement increased as the joist stiffness increased because the increase in stiffness of the system resulted in more relative deformation occurring from racking of the wall than from rotation of the wall; 4.) Maximum resistance of the system could be dictated by joist stiffness, not the wall strength alone, for walls supported by a joist; 5.) Results indicate that for a wall on a non-rigid foundation, maximum displacement exceeds that of a wall on a rigid foundation, while maximum resistance would be lower than that of a wall on a rigid foundation. Because both of these values are un-conservative for walls on a non-rigid foundation, these performance parameters should be further examined for walls on a non-rigid foundation.

Results from the parametric study indicated the following concerning shear walls subjected to dynamic loading: 1.) Vertical displacement increases with a decrease in joist stiffness, while the lateral displacement appeared to experience a resonant effect, with a localized peak lateral displacement occurring for the joist with a depth of 409 mm (16.1 in); 2.) This pattern was also evident with racking displacement and the maximum base shear; 3.) The percentage shear deformation increased as joist stiffness increased, except for a plateau around the joist size corresponding to the "resonant" configuration; 4.) Maximum displacement for walls on a non-rigid foundation exceed that of walls on a rigid foundation; 5.) Maximum base shear for walls on non-rigid foundations is lower than that for walls on rigid foundations. Given that maximum displacement is unconservative for a wall on a non-rigid foundation, these items should be further examined for shear walls on a non-rigid foundation.

5.4 Areas of Future Research

Although results of this study have shed some light on the effects of non-rigid foundations on the response of timber shear walls, there remain areas where research needs to be performed. These include:

1. Experimental tests need to be performed to further validate the program and to quantify the response of the system. Results from the present study were based on a numerical model that was validated using experimental data from wall tests conducted on a rigid foundation only. In order to truly validate the accuracy of WALSEIZ1 simulations of the response of a wall supported by floor joists, the program must be validated with data from experimental tests of walls resting on a flexible foundation. This would indicate whether or not the modeling assumptions are acceptable. Also, WALSEIZ1 is an idealized system in which it is assumed, among other things, that failure of the wall will occur because the connectors attaching the sheathing to the framing will fail. Experimental testing provides the opportunity

to observe how the system actually responds to loading so that any localized failures can be noted. Experiments on full size shear walls would provide answers to some questions concerning the effects of a flexible foundation on the response of shear walls.

2. Expand the parametric study to include changes in wall geometry (nail spacing, material properties, walls with openings, wall size) and excitation of the system. It is known that the response of a shear wall is dependent on the properties of the wall. Wall models included in the parametric study are identical models and can be altered and re-analyzed to investigate different parameters or configurations. Nail spacing is an important parameter that can be changed to determine if the optimum nail spacing for walls on a flexible foundation is the same as the optimum nail spacing for a wall on a rigid foundation. Material and geometric properties of the framing and sheathing should also be altered. Besides geometry and material properties, the effect of aspect ratio and openings needs to be considered. The presence of openings in walls may have a significant effect on walls resting on a flexible foundation as opposed to walls resting on a rigid foundation. However, until research is conducted, the effects of aspect ratio versus joist stiffness will not be known. Other items that could be investigated include the acceleration record used, the anchorages at the base of the wall (nailed versus bolted), and the effect of having a wall perpendicular to the joists.

3. Modify WALSEIZ so that a three-dimensional structure can be modeled and analyzed. Although most testing of shear walls focuses on the shear wall, the surrounding elements have

a significant effect on the response of the wall in a structure. These components are present in real life and could add stiffness to the shear wall. Some tests have been performed on complete structural systems but the systems were modeled as being on a rigid foundation and no consideration was given for the system being placed on a flexible foundation. Additional components will have an effect on the wall, but to what extent is unknown.

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