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Force Modification Factors for Timber Structures with Nailed Shear Walls

C.K.A. Stieda[1], E. Karacabeyli[2] and M. Yasumura[3]

ABSTRACT

The National Building Code of Canada provides force modification factors for the design of structures subject to earthquakes. Timber structures utilizing nailed shearwalls designed according to the requirements of CSA Standard 086.1, Engineering Design in Wood, are assigned a force modification factor of R=3. To verify this force modification factor a series of cyclic tests has been performed by the Wood Engineering Department of Forintek Canada Corp in Vancouver. Analysis has been carried out at the Building Research Institute of the Ministry of Construction in Tsukuba, Japan and at Forintek Canada Corp. of a number of typical wood frame wall systems using a Single Degree of Freedom (SDOF) model for one storey and a Multi Degree of Freedom (MDOF) model for two, three and four-storey buildings. The experimental data have been used to establish models of skeleton curves and hysteresis loops for a number of different wall constructions using plywood and gypsum wall sheathing. Using a number of earthquake records the load-deformation behaviour of these wall systems has been calculated. With storey deformation as a criterion, the force modification factor of R=3 for shear walls results in acceptable designs for the plywood panels investigated in this study under Canadian design conditions.

INTRODUCTION

A procedure for calculating live loads due to earthquakes is provided in the National Building Code of Canada (NBCC, Associate Committee on the National Building Code 1990a). This procedure requires the determination of the minimum lateral seismic force V at the base of the structure, the so-called base shear. This base shear is a function of a number of factors, including the response of different structural systems and different materials to the dynamic loading produced by an earthquake. The procedure requires the calculation of an equivalent lateral seismic force which represents the elastic response of the structure. To allow for the different inelastic behaviour of various structural systems and materials, the NBCC then provides for a modification of the elastic response by a force modification factor R.

For the seismic design of timber structures designed and detailed according to the provisions of CAN/CSA 086.1M (CSA, 1989) the NBCC recognizes three different structural systems, each with

[1]Timber Engineering Codes and Standards Consultant, Vancouver, B.C. Canada
[2]Wood Engineering Scientist, Forintek Canada Corp., Vancouver, B.C. Canada
[3]Senior Research Scientist, Building Research Institute, Tsukuba, Japan

their own force modification factor R. Timber frame buildings with a lateral load resisting system provided by nailed shear panels sheathed with plywood, waferboard or strandboard have been assigned a factor R equal to 3. This factor is based on limited test data. To confirm the validity of these force modification factors, research is currently being carried out cooperatively by an international team of researchers from various institutions. As part of this research, twenty-one types of shear wall have been tested at Forintek to investigate the effects of sheathing materials, sheathing orientation, blocking between studs at panel edges, taping of gypsum wall sheathing (GWB), and nail spacing on the lateral load resistance of shear walls (Karacabeyli et al 1994). Preliminary results of the testing and analysis of three of those wall types are given here.

METHODOLOGY

A number of sheathed walls 2.4 m in height and 4.8 m long were subjected to in-plane monotonic or cyclic lateral displacements along one edge of the wall while the other edge was being held in a fixed position. The lateral resistance and the ultimate strength of these walls were determined. Models were developed for the load-deformation behaviour of these shear walls.

A series of building designs were drawn up using the NBCC and a force modification factor of 3. The dynamic behaviour of these buildings was studied analytically using a multi degree of freedom lumped-mass earthquake response analysis program developed by Yamanouchi (1990). The computer program includes a non-linear model with pinching. Seismic records used for this analysis were provided by the Building Center of Japan.

MATERIALS AND SPECIMEN ASSEMBLY

For the current investigation a 2.4 m high by 4.8 m long wall was chosen. This allowed staggering of the 1.2 m x 2.4 m panels when panels were positioned horizontally, and testing of specimens positioned either vertically or horizontally. The 4.8 m wall length is long enough to allow the investigation of the effect of openings in future studies. A test frame (Figure 1) for evaluating the behaviour of a 2.4 m x 4.8 m framed wall section under monotonic and cyclic loads was designed and built.

Of 21 wall types tested to date, three have been selected for presentation here. The framing of all walls consisted of vertical S-DRY (surfaced-dry) No. 1 and 2 SPF lumber, 38 mm x 89 mm in size, spaced at 400 mm, with a 4.8 m long double plate of S-DRY 1650f-1.5E SPF at the top and a single plate at the bottom. Two of these wall types (No. 14 and 16) used horizontally placed 9.5 mm CSP construction sheathing and continuous blocking. For wall type 14 the plywood was nailed around the perimeter with 3 x 65 mm common nails spaced at 150 mm. For wall type 16 the spacing was reduced to 64 mm. Nail spacing along studs not supporting a plywood edge was 300 mm. The third wall type reported on here is wall type 11. This wall type used 12.7 mm gypsum wall sheathing (GWB) screwed to the framing with 2.7 x 32 mm GWB screws spaced around the perimeter at 200 mm. Blocking along the long edges of the GWB was staggered such that the edges of the sheathing at half the wall height were supported in every other space between studs, that is, for a total length of 1.2 m along each 2.4 m long panel edge. For each wall type, two wall specimens (one for static and one for cyclic testing) were built in the laboratory by a building contractor following usual on-site construction practices. Layout of sheathing on the wall is shown in Figure 1.

TEST APPARATUS AND PROCEDURE

Cyclic and monotonic shear tests were conducted on 2.4 m x 4.8 m wood frame stud walls suspended from a steel frame bolted to a concrete strong wall (Figure 1). No standard tests are available for cyclic load testing of shear walls. Therefore the following test apparatus and procedures were used in this study. The double plate (top) of the wall was bolted to the steel framework; the single plate (bottom) of the wall was bolted to a built-up steel channel and plate assembly. A steel I-beam was mounted on vertical load cells attached to a concrete strong floor immediately below the plate channel assembly, and roller cages were used between this I-beam and the bottom plate channel assembly. Steel guides clamped to the floor provided lateral support for the bottom plate assembly. A servo-controlled hydraulic actuator, equipped with swivels at each end and with a load cell, was mounted horizontally on a bracket bolted to the floor. This actuator was used to impose a horizontal displacement to the bottom plate-steel channel assembly in the longitudinal direction of the wall.

A displacement transducer was attached to the far end of the bottom plate of the wall, opposite the end where the hydraulic actuator was located, to measure the horizontal displacement of the wall with respect to the concrete strong floor in the direction of the imposed displacement. The data from this transducer were used to determine yield displacements (δy).

Monotonic (static) and reverse cyclic displacements were applied to the bottom plate through a load spreader. For each type of construction, one wall was tested to failure in a static test using a stroke rate of 0.127 mm/s. A nominal yield point (δy) was defined as the horizontal displacement at a resistance equal to half of the maximum load (P_{max}) obtained during the monotonic loading test. Reverse cyclic wall tests consisting of a three-displacement-cycle sets or groups at 0.5 Hz (Figure 2) were then conducted to various maximum displacement levels. Maximum displacement for the first set of three cycles was 50% of the nominal yield point (δy). Maximum displacement for the last set ranged to as high as 600% δy.

TEST RESULTS AND INTERPRETATION OF TEST DATA

A summary of the test results is given in Table 1. The horizontal displacements at 50%, 80% and 100% P_{max} define three points on the load-displacement curve for the monotonic test specimen of a given wall type, and for the first cycle of a specific cycle group of the matching cyclic test specimen. Figures 4 and 7 illustrate the difference between the monotonic and the cyclic load-displacement curves.

Lateral resistance-displacement hysteresis loops from a typical reverse cyclic load test are shown in Figure 3 for wall type 16. The initial part of an imposed displacement is always shown as positive in the first quadrant. After the imposed displacement has returned to zero the reverse loading cycle is shown as negative loads and displacements in the third quadrant. The curve labelled 2 is the first cycle from the second set (or cycle group) which had a nominal maximum displacement equal to δy, the displacement at the nominal yield point defined for the static test. Actual average peak load from the first and third quadrant of Figure 3 for the first cycle was 8.6 kN/m at an average displacement of 10 mm. The curve labelled 3 is the first cycle in set 10. The maximum imposed displacement for this set was three times the nominal yield displacement. As in all sets the peak loads for the second and the third cycles were substantially lower than that for the first cycle. It appears that there were no appreciable reductions in load carrying capacity after the second cycle. The lateral resistance for the third cycle therefore was taken as the stabilized lateral resistance. The curve labelled 4 was produced by an imposed nominal maximum displacement of 6 times the nominal yield displacement. Actual displacement for the first and second cycle of this set was about 50 mm. Because the wall started to deteriorate substantially at this displacement, the control system of the hydraulic cylinder overshot. Consequently, in the third cycle of this set, a maximum displacement of about 70 mm was recorded.

The upper envelope curves obtained from increasing stabilized cycle groups are shown in Figure 4 for two walls. The maximum static shear resistance of the GWB walls (Wall type 11) with its 200 mm screw spacing was found to be about 40% of that for the plywood walls (Wall type 14) having a 150 mm nail spacing. For the maximum loads obtained in stabilized cycles, this ratio was approximately 30%. However, these maximum loads in GWB walls occurred at a lower displacement level of about 10 mm than that for plywood walls, which had a displacement of about 30 mm at maximum load. Furthermore, strength degradation in the GWB walls relative to the monotonic test started when the horizontal displacement reached the 4 mm level whereas the stabilized curve for the plywood wall followed the monotonic test curve until a 30 mm displacement had been reached. These findings suggests that one cannot superimpose the load carrying capacities of GWB walls and plywood walls without considering differences in strength and stiffness degradation under cyclic loading. Another observation that can also be seen in Figure 4 is that the maximum lateral resistance in the stabilized cycle group (3rd cycle) occurs in an earlier cycle group than the one for the first cycle because the maximum resistance of the first cycle groups generally constitutes the ultimate load in cyclic load testing. After this load has been achieved significant strength degradation occurs and consequently that cycle group does not give stabilized cycles. Only the previous increasing cycle group shows a stabilized cycle, which generally provides the maximum stabilized load.

The stiffness of the wall is influenced by 1) nail-slip between the sheathing and wood frame members; 2) shear deformation of the sheathing material; 3) Panel-to-panel contact and crushing of panel corners; 4) Displacement or crushing between the end studs and the plates; 5) Relative displacement between the steel plates and wood plates at the top and bottom of the wall (found to be minimal in this test); and 6) deformation of the overall testing frame (found to be minimal). Cyclic stiffnesses for the increasing stabilized cycle groups are shown in Figure 5. The cyclic stiffness here is defined as the ratio of the mean of the absolute values of the peak resistances for the cyclic and the reverse cycle loading to the mean of the absolute values of the corresponding displacements. Note that the cyclic stiffness decreases with increasing multiples of displacements at the yield point. Actual displacements at the yield point differ for the different wall types and are shown in Table 1 as well as in the legend in Figure 5.

Reducing the nail spacing from 150 mm for wall type 14 to 64 mm for wall type 16 increased the static and quasi-static shear resistance by about 60 - 80% (Table 1).

MODELS FOR SHEAR WALLS

The basic model for the dynamic behaviour of shear walls is an expansion of the model described by Yasumura (1991) and is illustrated in Figure 6. Lateral strength and corresponding displacements for points P_1, P_2 and P_3 are shown 10 models in Table 2. All models specify a maximum lateral strength of P_2. The first three models assume continuous yielding at constant load. The remaining models stipulate decreasing strength with increasing displacements. Figure 7 shows how model 16-b was fitted to the test data for wall type 16. For clarity only sets 2, 12 and 14 of the three cycle sets are shown. Figure 7 also shows the static load - deformation curve obtained for the first specimen of wall type 16. Loads for models 33 to 36 using 12.5 mm plywood were calculated by proportioning the experimentally obtained forces according to the strength values for these wall types given in CAN3/CSA-O86.1-M (CSA, 1989). Loads and displacements for models 11-b and 33 to 36 were used to calculate parameters for a building that used both gypsum wall sheathing and plywood.

BUILDING DESIGN

Using the provisions of CAN3/CSA-O86.1-M nine apartment buildings were designed for Vancouver, B.C. The first 8 designs were rectangular in plan and had external dimensions of 27m x 45 m. Typical apartment size was 9 m x 12 m. It was assumed that the building had a central corridor running in the longitudinal direction of the building. Framing was assumed to be a SPF stud grade. Calculations were only carried out for the longitudinal direction of the building, i.e. for the 45 m direction.

In Design No. 1, 2, 3 and 4 in Table 3, the total length of structural shear walls at each floor level was assumed to be the minimum length allowed for a nail spacing of 64 mm around the perimeter of individual panels. Total length of structural shear walls calculated by using the provisions of CAN3/CSA-O86.1-M are shown in Table 3. In Design No. 5, 6, 7 and 8, it was assumed that both exterior walls and the corridor walls were sheathed in plywood on one side and that these walls would carry the shear loads at each floor level. Allowing for window and door openings the available length of structural shear wall at each floor level was assumed as 130 m. Total mass, base shear and design details for the sheathing are also shown in Table 3. For design No. 5, a one-storey building, the nail spacing required around the perimeter was increased to 150 mm.

Design No. 9 represents a preliminary design for a four-storey apartment building prepared as a design example by the Wood Frame Committee of the Structural Engineers Association of British Columbia. This building was included in the analysis to study the effect of the use of gypsum sheathing on the resistance of buildings to dynamic forces. A peak acceleration of 2.25 m/s^2 was used. The total length of structural plywood walls on each floor in the longitudinal direction of this building was 170 m. The building had been designed to use 12.5 mm CSP. The spacings of nails around the perimeter of individual plywood panels ranged from 64 mm for the bottom storey to 150 mm for the top storey (See Table 3). All non-structural walls were sheathed with 12.7 mm GWB at a spacing of screws of 200 mm. Total length of GWB sheathed walls for each storey in the longitudinal direction of the building was 1184 m. In order to be able to calculate parameters P_1 and P_2 for a common model that could represent the combined effect of the plywood and the gypsum sheathing models had to be selected that provided for the same displacement for plywood and GWB at points P_1, P_2 and P_3. The parameters for these models are those in Table 2 shown for model types 11-b and 33 to 36. The peak lateral resistances P_1 and P_2 were then multiplied with the appropriate wall length for the plywood and the GWB and the products were added as required for each storey.

ANALYSIS

All designs were analyzed using acceleration records in the NS direction for El Centro and the EW direction for Taft and Hachinohe. Accelerations were scaled to maximum accelerations of 2.0, 3.0

and 4.0 m/s^2. The results for the El Centro and the Taft records were similar and therefore only results for Taft EW are given in this paper. The mass at each storey for each building was calculated and used in the dynamic analysis. Damping of 5% was assumed. Only the total mass for each building, and not the individual mass for each storey, is given in Table 3. Based on the experimental data a load-displacement model for each wall type had been developed. The wall type used for each building is shown in Table 3. Parameters for each wall type are shown in Table 2. Based on the length of structural wall available an initial shear stiffness for each storey was calculated and used in the analysis. Only the shear stiffness for the bottom storey is shown in Table 3.

RESULTS

As an overview of the dynamic analysis the natural period of the first mode and the maximum lateral displacement of the roof relative to the base of the building is given in Table 4. As can be expected the lateral displacement increases with the magnitude of the maximum acceleration used in the analysis. It is evident that the Hachinohe acceleration record will result in greater lateral displacements than the Taft record. For the Taft record the lateral displacements are greater for the buildings with minimum strength walls. This is also the case for the Hachinohe acceleration record except for the four storey building. The lateral displacements of the four storey building of design No. 9, which took into account the stiffness provided by the gypsum sheathing (No. 9.2), are less than those of the buildings for which only the stiffness of the plywood was considered (No. 9.1).

The dynamic analysis provided maximum shear forces and displacements for each storey as well as the maximum lateral deflection of the roof relative to the base of the building. The maximum lateral storey displacement of design No. 4, the 4-storey building with minimum structural wall length, are given in Fig. 8 for the two earthquakes with peak accelerations of 2.0, 3.0 and 4.0 m/s^2. Lateral storey displacements generally are greater for the third and the fourth storey than for the first and the second storey. Displacements are also greater for the Hachinohe earthquake than for the Taft acceleration record.

The model parameters used for this analysis are given in Table 2 and assume that the walls could yield indefinitely. The experimental evidence however indicated that beyond a lateral displacement of 30 mm the plywood wall started to deteriorate loosing its lateral resistance rapidly (see Fig. 3). Therefore designs giving lateral displacements greater than 30 mm, when using model No. 16, are not acceptable. Fig. 8 therefore indicates that for peak accelerations equal to or greater than 3.0 m/s^2 the design analyzed here could result in failure of the building.

Fig. 9 shows the maximum lateral displacement of the bottom storey of four different buildings ranging from 1 to 4 storeys in height. These buildings were designed to have structural walls of equal strength and stiffness for all storeys except the top storey, which had a reduced strength and stiffness. Again the model assumed an infinite capacity of the structural walls to yield. For this design there appears to exist a drastic difference in the maximum lateral displacement due to the type of earthquake that is used for the analysis. For the Hachinohe acceleration record in the EW direction the maximum lateral displacements of the bottom storey of the three and four storey high buildings are excessive for even the peak acceleration of 2.0 m/s^2. On the other hand for the Taft EW record storey displacements of more than 30 mm are only occurring for three and four storey buildings that are subject to accelerations greater than 3 m/s^2.

The maximum lateral forces generated in the first storey of these four buildings is shown in Fig. 10. It is evident from this figure, that both the Taft as well as the Hachinohe record in the EW direction, scaled to a maximum acceleration of 3.0 m/s^2, will result in a yielding of the shear walls for all the buildings, except the one-storey high building.

Since the NBCC does not provide design data for the structural utilization of GWB design No. 9 was first analyzed by considering the stiffness and strength of the structural plywood walls only. The results of this analysis are given in Table 5 as Case 1. The building was then reanalysed using the stiffness for model type 11-b. The results of this analysis are given in Table 5 as Case 2. The lateral shear forces at each floor level are given in Table 5. It should be noted that for Case 1 these forces are greater than P_1 but less than P_2 indicating that only a small amount of yielding would take place. This is confirmed by the storey deflections which do not exceed 20 mm. If the combined effect of plywood and gypsum wallboard are considered the increase in stiffness will result in smaller lateral displacements. As a result of the greater stiffness the combined lateral forces being resisted by the plywood and the gypsum sheathing also are greater than for Case 1. However for all four floors these forces do not exceed the value of P_1 for that floor, i.e. the response of the building remains within the elastic range of the wall stiffnesses. The total lateral shear forces have been proportioned according to the relative initial stiffness of the plywood and the gypsum sheathing for each storey. The resulting unit shear forces are shown in the last two columns of Table 5. It is evident that the inclusion of the gypsum sheathing will result in some reduction of the forces acting on the plywood.

DISCUSSION

The peak horizontal ground acceleration at a probability of annual exceedance of 0.0021 suggested for Vancouver in Table J-2 of the revised supplement to the National Building Code of Canada is 0.23 times gravity, or 2.25 m/s^2 (NBCC, 1990b). Maximum lateral forces and displacements calculated here for a peak acceleration of 2.0 m/s^2 therefore appear to be representative of the behaviour of timber-framed buildings up to four storeys in height designed according to the provisions of CSA O86.1 for plywood-sheathed shear walls using a force reduction factor of $R = 3$. It appears that such buildings when subject to an acceleration of the type recorded at Taft in the EW direction would be able to withstand such an earthquake. On the other hand for earthquakes that could be represented by a Hachinohe type record some four-storey buildings at least would deflect excessively and suffer substantial damage. Guidance on the type of earthquake that could be expected for a given location therefore should be sought.

CONCLUSION

Under Canadian design conditions, the force modification factor of $R=3$ currently suggested in the National Building Code of Canada for nailed shear walls results in acceptable designs when storey deformation is used as a criterion.

RECOMMENDATIONS

Guidance is required by seismologists on the specific acceleration records that should be used for dynamic analyses at locations across Canada.

Agreement should be sought by the international seismic research community on an acceptable standard definition of the yield point for timber structures.

Model parameters for plywood thicknesses other than 9.5 mm and other sheathing types should be developed.

Design procedures to account for the stiffness and strength of gypsum wall sheathing should be developed.

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Table 1. Test Results, Monotonic and First Cycle Maximum Lateral Resistances Displacements at Various Load Levels

Wall Type	Test Type	Cycle Group No.	Max. Lateral Resistance (P_{max})	Horizontal Displacement at		
				100% P_{max}	80% P_{max}	50% P_{max}
			kN/m	mm	mm	mm
11	Monotonic	-	3.6	15	7	4
	Cyclic	12	3.0	13	10	7
	Reverse Cyclic	12	3.1	13	10	8
14	Monotonic	-	8.7	54	24	9
	Cyclic	12	9.2	36	28	23
	Reverse Cyclic	12	8.6	36	28	23
16	Monotonic	-	15.4	48	24	11
	Cyclic	12	15.7	36	30	23
	Reverse Cyclic	10	13.8	30	26	20

Table 2. Model Parameters for Walls Walls Sheathed on One Side

Model Type	Strength			Displacement			Fastener Spacing	Wall Description
	P_1 kN/m	P_2 kN/m	P_3 kN/m	y_1 mm	y_2 mm	y_3 mm	mm	
11-a	1.72	2.32	2.32	3	6	13	200[3]	12.7 mm GWB-taped
14-a	4.69	7.00	7.00	9	23	28	150[1]	9.5 mm CSP
16-a	8.58	12.07	12.07	10	20	30	64[2]	9.5 mm CSP
11-b	2.32	1.39	0	10	30	59	200	12.7 mm GWB
14-b	4.9	7.3	0	10	30	59	150	9.5 mm CSP
16-b	8.6	13.8	0	10	30	59	64	9.5 mm CSP
33	5.8	8.7	0	10	30	59	150	12.5 mm CSP
34	7.0	11.2	0	10	30	59	100	12.5 mm CSP
35	9.3	14.9	0	10	30	59	75	12.5 mm CSP
36	10.4	16.7	0	10	30	59	64	12.5 mm CSP

[1]Fasteners: 3 x 65 mm common nails
[2]Fasteners: 3 x 65 mm common nails
[3]Fasteners: 2.7 x 32 mm GWB screws

Table 3. Design Parameters

Design No.	Model Type	No. of Storeys	Total Mass	Base Shear	Length of Structural Shear Wall Storey Number				Shear Stiffness, Bottom Storey	Sheathing		Nail Spacing
					1	2	3	4		Type	Thickness	
			t	kN	m	m	m	m	kN/mm		mm	mm
1	16-a	1	152	179	20.6				17.1	CSP	9.5	64
2	16-a	2	397	467	53.8	29.8			44.7	CSP	9.5	64
3	16-a	3	642	755	87.0	57.3	27.7		72.3	CSP	9.5	64
4	16-a	4	887	1043	120.2	106.1	77.8	35.2	99.9	CSP	9.5	64
5	14-a	1	152	179	130.0				69.3	CSP	9.5	150
6	16-a	2	397	467	130.0	130.0			108.0	CSP	9.5	64
7	16-a	3	642	755	130.0	130.0	130.0		108.0	CSP	9.5	64
8	16-a	4	887	1043	130.0	130.0	130.0	130.0	108.0	CSP	9.5	64-150[1]
9	33 -36	4	1252	1472	170.0	170.0	170.0	170.0	436	CSP	12.5	64-150[2]
	& 11-b				1184.4	1184.4	1184.4	1184.4		GWB	12.7	200

[1]Nail spacing around perimeter of individual 9.5 mm thick panels for storeys 1 to 3 equal to 64 mm, for top storey 150 mm
[2]Nail spacing around perimeter of individual 12.5 mm thick panels for storeys 1 to 4 equal to 64.5, 75, 100 and 150 mm respectively.

Table 4. Natural Periods and Maximum Lateral Roof Displacements Relative to Base of Building

Design No.	Natural Period, s First Mode	Maximum Lateral Displacement of Roof, mm Seismic Record					
		Taft EW			Hachinohe EW		
		2.0 m/s	3.0 m/s	4.0 m/s	2.0 m/s	3.0 m/s	4.0 m/s
1	0.592	28	50	74	37	72	105
2	0.696	42	65	95	112	236	354
3	0.851	54	81	100	136	292	359
4	0.906	62	82	103	71	104	169
5	0.294	9	15	25	11	17	31
6	0.437	24	35	53	27	49	88
7	0.612	33	56	76	55	148	241
8	0.796	45	70	92	116	226	351
9.1	0.816	62	72	100	132	-	-
9.2	0.485	32	50	55	34	73	-

Table 5. Model Parameters and Results of Dynamic Analysis for Four Storey Apartment Building (Design No. 9), Seismic Record: Taft EW, Scaled to 2.25 m/s^2

	P_1	P_2	Storey Deflection	Lateral Shear Force	Lateral Unit Forces	
					CSP	GWB
	kN	kN	mm	kN	kN/m	kN/m
Case 1: 12.5 mm CSP L = 170 m						
1. Storey	1765	2841	16	2056	12.1	-
2. Storey	1576	2535	17	1899	11.2	-
3. Storey	1182	1902	20	1539	9.1	-
4. Storey	984	1471	12	1023	6.0	-
Case 2: 12.5 mm CSP + 12.7 mm GWB CSP: L = 170 m GWB: L = 1184 m						
1. Storey	4513	4487	15	4511	7.0	2.8
2. Storey	4324	4181	10	4323	7.6	2.6
3. Storey	3930	3548	8	3047	6.5	1.6
4. Storey	3732	3117	5	1679	3.9	0.9

$P_3 = 0$; $y_1 = 10$ mm; $y_2 = 30$ mm; $y_3 = 59$ mm

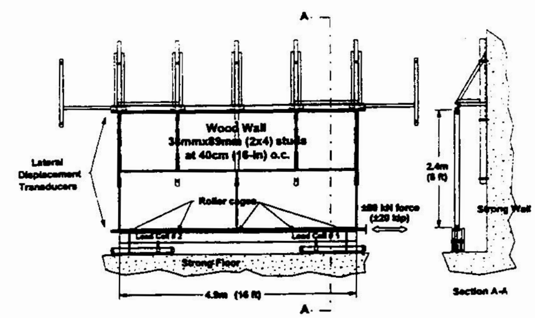

Figure 1. Test Set-up for Shear Wall (Horizontal Sheathing) Test

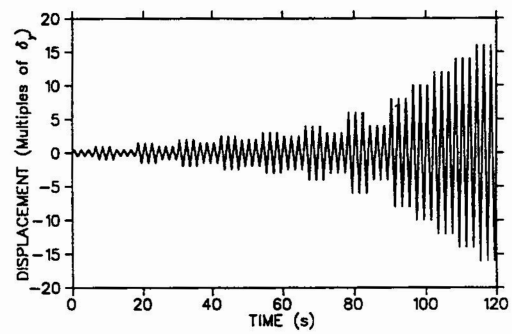

Figure 2. Quasi-Static Reverse Cyclic Displacement Schedule

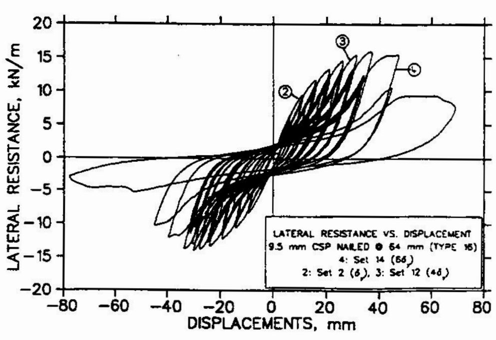

Figure 3. Test Results, Wall Type 16

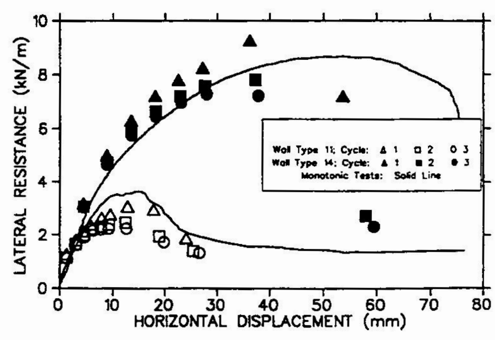

Figure 4. Upper Envelope Curves obtained from Increasing Stabilized (3rd) Cycle Groups

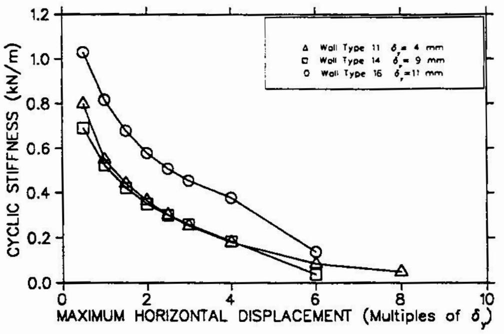

Figure 5. Cyclic Stiffness obtained from Increasing Stabilized (3rd) Cycle Groups

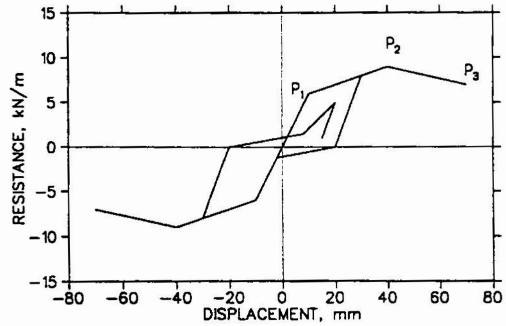

Figure 6. Hysteresis Model

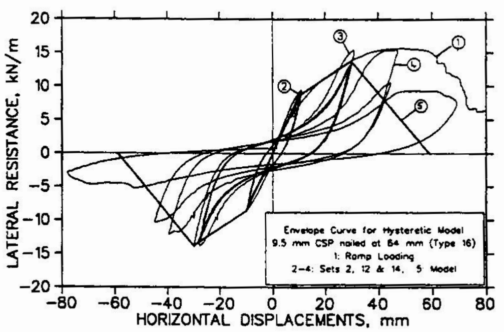

Figure 7. Hysteretic Model Used in the Analysis for Wall Type 16

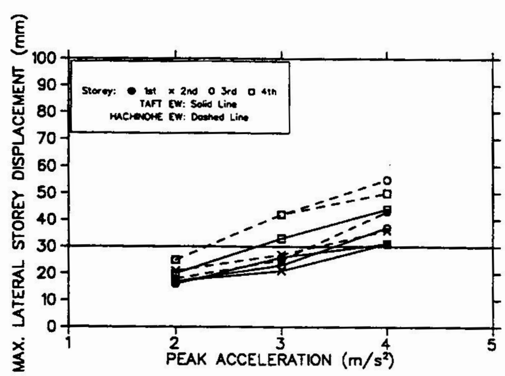

Figure 8. Maximum Lateral Storey Displacements, Four Storey Building - Structural Walls of MINIMUM Strength and Stiffness

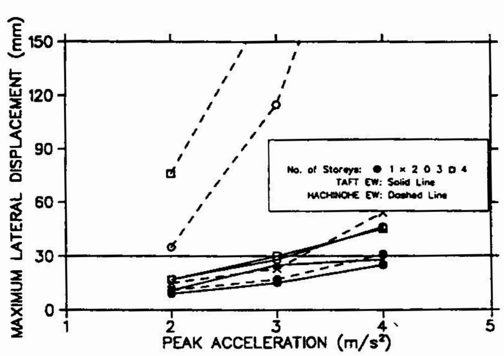

Figure 9. Maximum Lateral Displacements for First Storey of 1 to 4 Storey Buildings, Structural Walls of EQUAL Strength and Stiffness

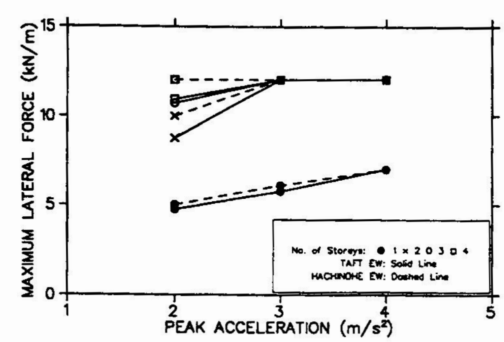

Figure 10. Maximum Unit Lateral Forces for First Storey of 1 to 4 Storey Buildings, Structural Walls of EQUAL Strength and Stiffness