

## Chapter 3

### Modeling Method

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The main aim of this chapter is to provide a method required for the development of FE models for light frame wood buildings subjected to lateral loads. An analytical study of the resultant force on one of the tie downs used to support shear walls in light frame houses is presented. To simplify the problem, only two-dimensional finite element models of shear walls and lip elements are generated in SAP 2000. Also, the effect of the following parameters on the ratio of actual tie down force to its maximum value is observed:

1. B/H ratio (width/height of shear wall),
2. A, Spacing between shear walls,
3. d, Depth of lip element,
4. Tie down anchor stiffness, and
5. Stud wall stiffness

Figure 3.1 show a simple 3D light frame wooden house. The detailed view of one of the cross-sections, presenting the above parameters in a simple 3D light frame wooden house, is shown in Figure 3.2.

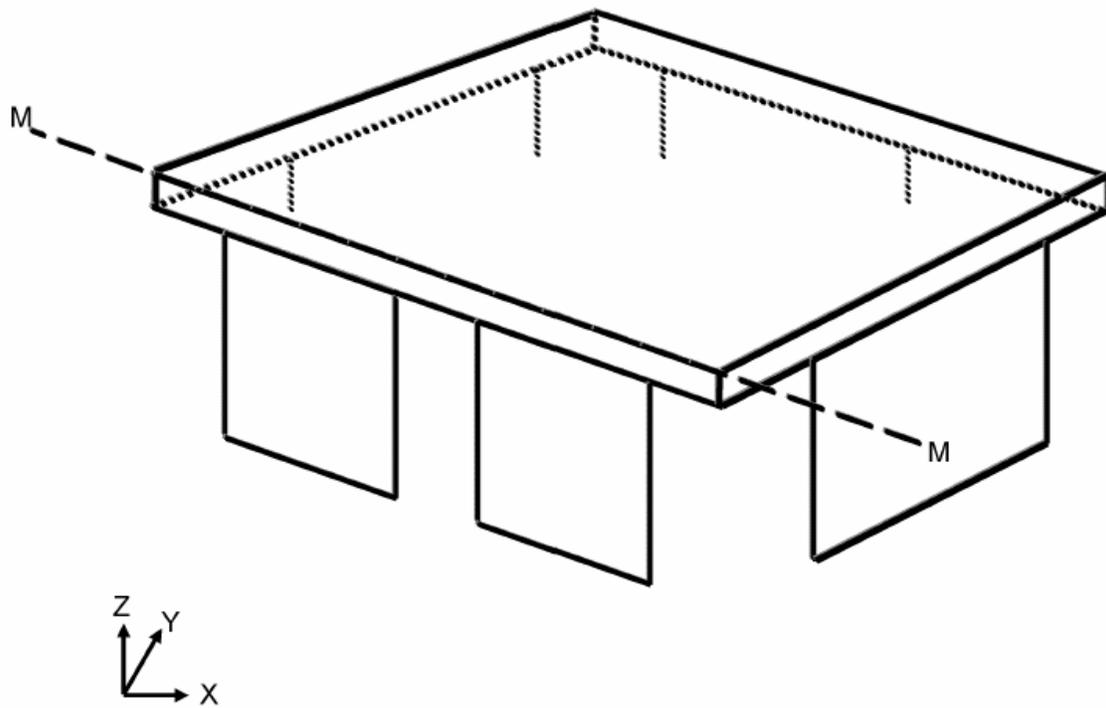


Figure 3.1: A simple 3D light frame wooden house

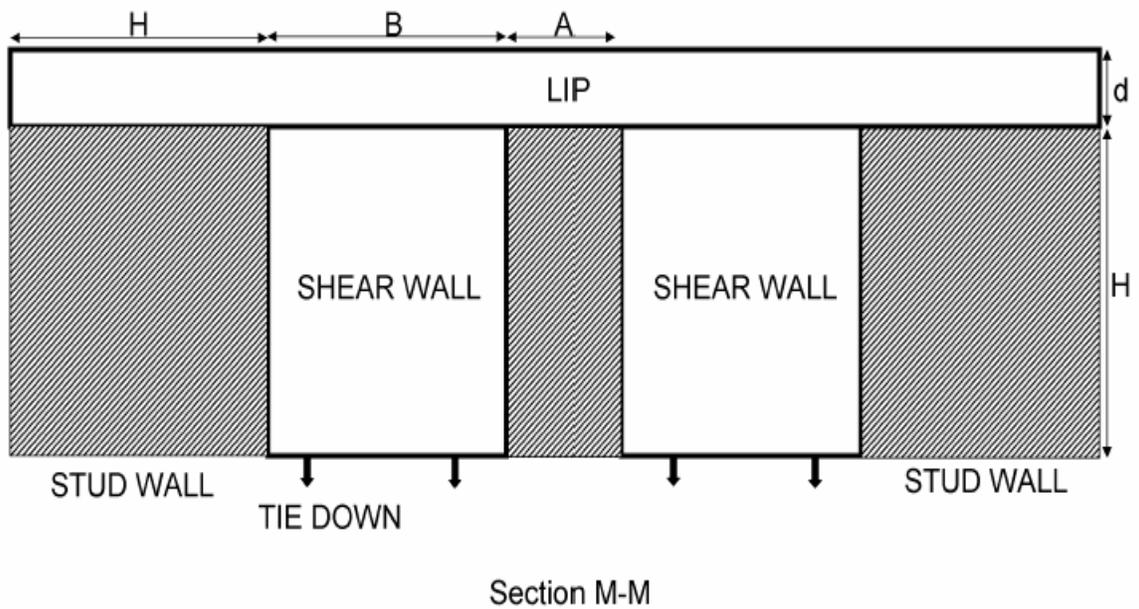


Figure 3.2: Showing detailed view of section M-M

### 3.1 Current Practices

A light frame house is constituted of several components which include diaphragms, walls, floors, and roof systems. All these components are interconnected by connections such as, nails, staples, bolts, and metal straps, forming a light frame house as a three dimensional highly indeterminate structural system. Very little is known about how lateral loads get distributed among these “structural”<sup>1</sup> and “non-structural”<sup>2</sup> components. Hence, simplifying assumptions are made in the analysis and design of such structures, resulting in either under-designed or over-designed elements.

The Residential Structural Design Guide (NAHBRC, 2000) provides the details for design of light frame wood houses, apartments and town houses in the United States. The guide presents the importance of lateral load distribution in the design process, but in the absence of any experimental and comparative evidences, it makes no attempt to identify which approach may be considered as the best. The three most common methods of lateral force distribution are as follows:

1. The tributary area method, which assumes that the diaphragms are flexible, compared to the shear walls.
2. The total shear method, which is based on designer judgment of the load distribution.
3. The relative stiffness method, which assumes that the horizontal diaphragms are rigid, compared to shear walls.

Thus, we see a clear need to supplement the current analysis practices for analyzing the light frame wood houses. The improved procedures (1) should be analytically correct and (2) should offer reasonable trade offs between uncertainties and neglect in the design process. Kasal et al, (2004) presented a comparative study of methods for lateral force distribution with a detailed three-dimensional analysis using a finite element model. An example test model of an L-shaped single-story wood frame house was developed and the results were compared. The FE model was generally very accurate with respect to the experimentally measured load distribution.

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<sup>1</sup> Structural component is defined as a component which supports variable and non-variable loads occurring in a structure for example shear walls and diaphragms.

<sup>2</sup> Non-structural component is defined as non-load bearing member in a structure for example stud walls, bolts, and nails attached to any non-structural component.

## 3.2 Modeling Requirements

To study the lateral force distribution on the vertical tie downs, various simplified finite element models are required to be generated for cross-sections similar to the one shown in Figure 3.2. The shear walls and lip<sup>3</sup> are modeled using quadrilateral shell elements. As we know that walls are based on ground which is stiff, hence in all the models a rigid element is used below the walls. Tie downs are to be modeled as some sort of flexible elements. As the connection between walls and lip is not continuous, the connection is made using some linear connector elements. To impose the displacement compatibility between the nodes connected by connector elements, two rigid beams are placed parallel to one another, between the walls and lip elements. In one set of models, the stud wall behavior is assumed as rigid in the vertical direction and hence roller supports are placed on the corresponding nodes. In the other set of models, the stud wall behavior is assumed as flexible in vertical direction and hence spring supports are placed on the corresponding nodes.

## 3.3 Analysis

The data adopted for the analytical models is fairly typical of U.S. construction. The results obtained for the tie down forces in the shear wall 2, as shown in Figures 3.3 and 3.4, is plotted as the percentage of the corresponding maximum tie down force for comparison. The two dimensional finite element models of the walls are generated in SAP 2000. Figures 3.3 and 3.4 model the stud wall by roller supports and spring supports, respectively. The vertical tie down of interest is encircled in these figures.

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<sup>3</sup> Lip is a filler beam between the wall and diaphragm

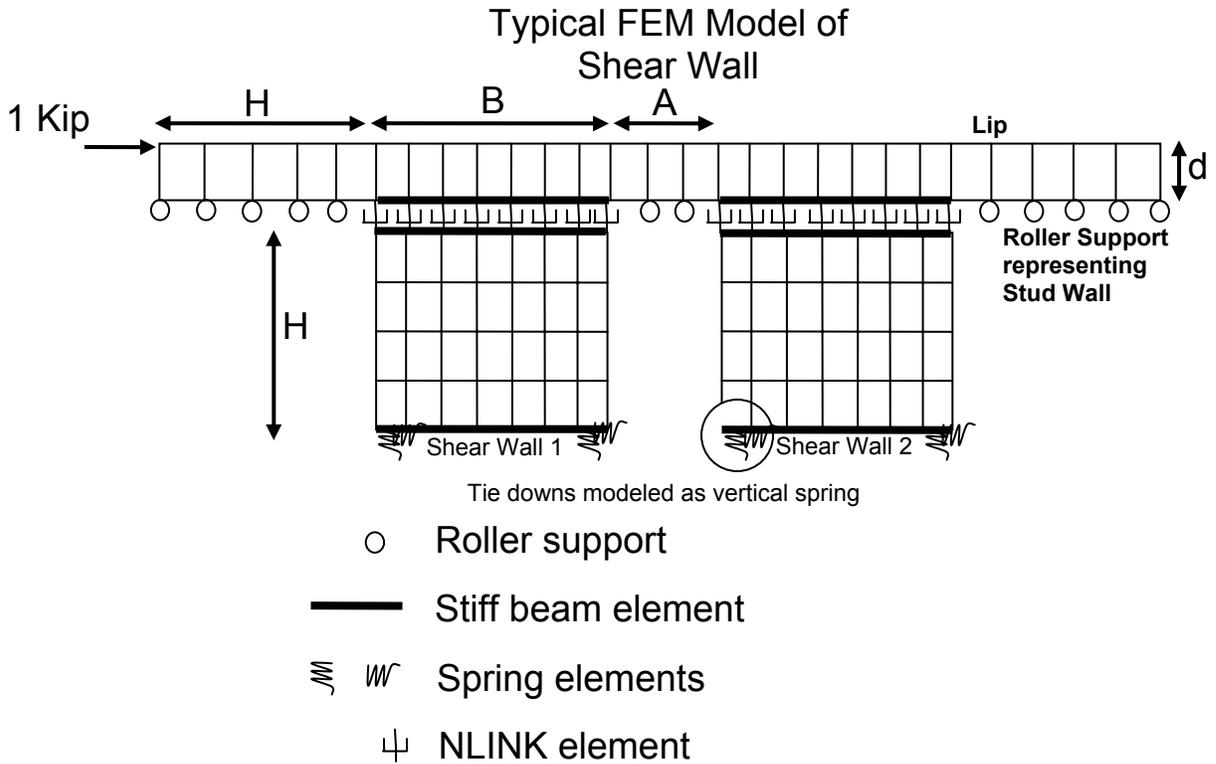


Figure 3.3: Typical FEM model generated in SAP 2000. The stud wall is modeled by roller supports.

One hundred and eight such models, thirty six as shown in Figure 3.3 and seventy two as shown in Figure 3.4, with the varying physical parameters as shown in Table 3.1, were generated using SAP 2000.

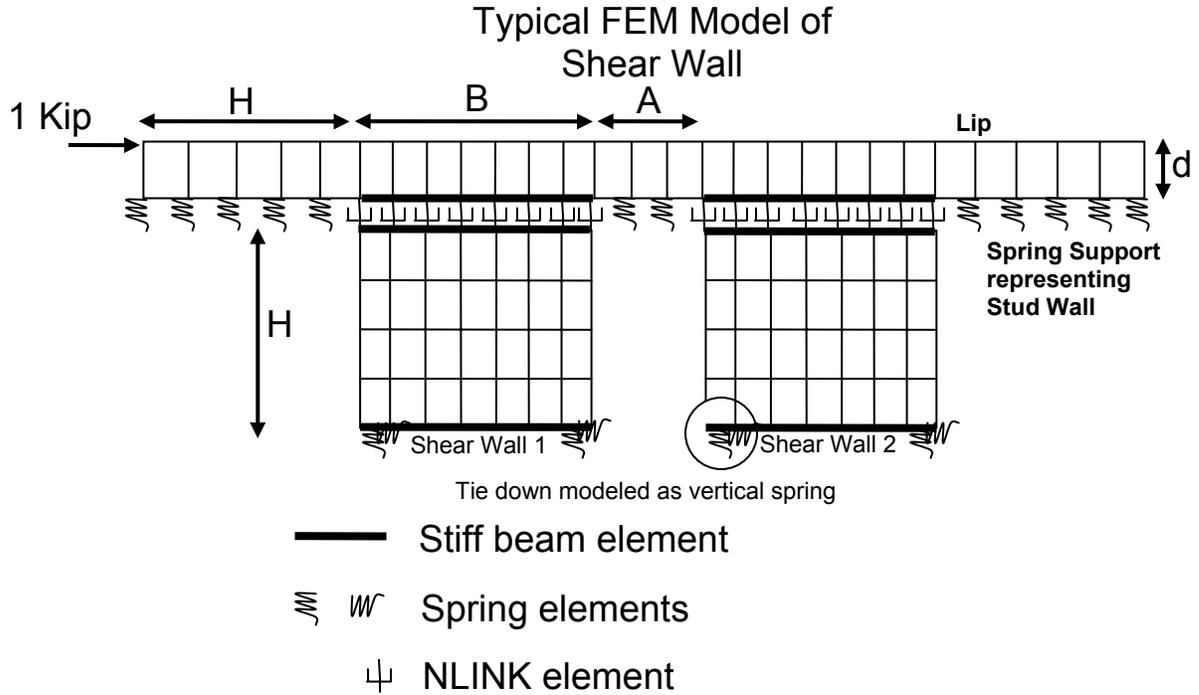


Figure 3.4: Typical FEM model generated in SAP 2000. The stud wall is modeled by spring supports.

Table 3.1: Typical parameters for the analytical models

<b>B/H ratio</b>	<b>d(in.)</b>	<b>A(in.)</b>	<b>Tie Down Anchor Stiffness (Kips/in.)</b>	<b>Stud Wall</b>
0.5	12	24	5000	Roller Support
1	24	48	50000	Spring Support (K=76 Kips/in.)
2	36	72	500000	Spring Support (K=19 Kips/in.)
-	-	108	-	

Figure 3.5 shows the free body diagram for the calculation of the maximum ideal tie down force for the tie down attached to the shear wall. It is assumed that half of the applied horizontal load goes into one shear wall. Also note that the maximum tie down force is the force without assuming any coupling provided by the lip element. The values calculated for maximum tie down forces are shown in Table 3.2.

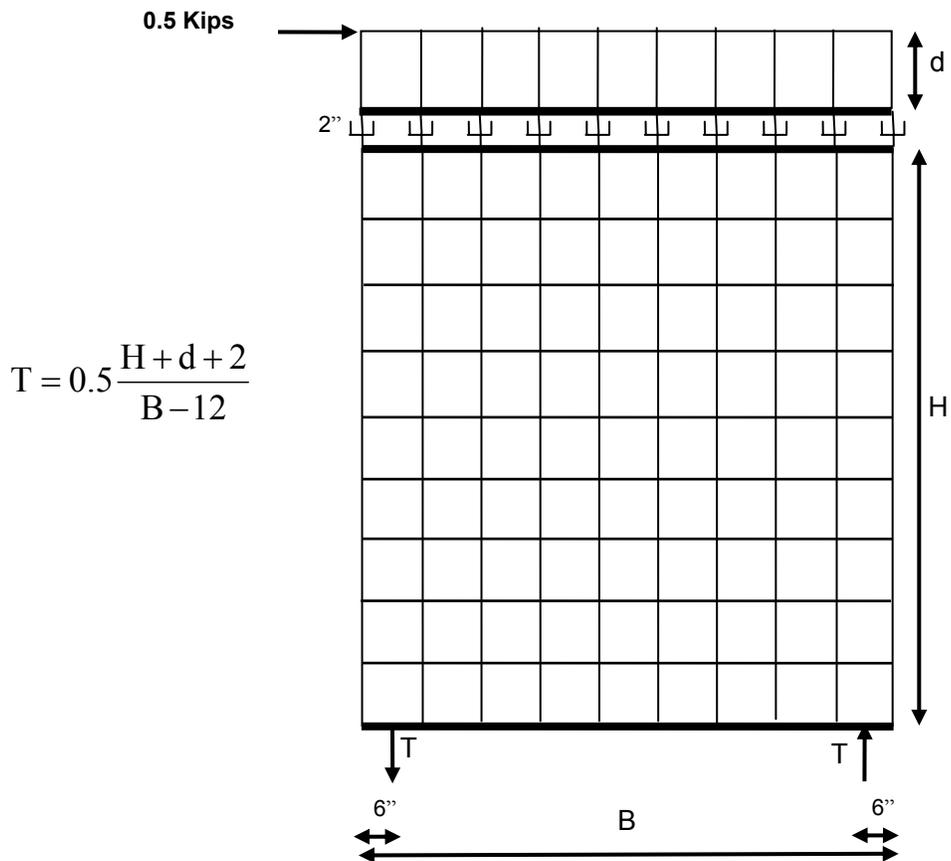


Figure 3.5: Free body diagram for maximum tie down force calculation (length units are in inches)

Table 3.2: The maximum tie down forces (kips)

	<b>B=0.5H</b>	<b>B=H</b>	<b>B=2H</b>
<b>d = 12"</b>	1.452	0.635	0.300
<b>d = 24"</b>	1.595	0.698	0.328
<b>d = 36"</b>	1.738	0.760	0.760

### 3.3.1 Geometry & SAP Model

Typical dimensions of shear walls used are 4.5 ft x 9 ft, 9 ft x 9 ft and 18 ft x 9 ft. The walls are modeled in SAP using shell elements with membrane thickness as 0.438 in. and modulus of elasticity as 1600 kips/in<sup>2</sup>. Each model has two shear walls located in the XZ plane. The shear walls are supported on a beam element of very high axial, flexural

and shear rigidity, which in turn is supported on two vertical and two horizontal spring elements. These spring elements represent the anchor tie downs and the lateral force resisting elements, respectively. Similar beam elements are used at the top edge of the wall which connects to another similar stiff beam element using SAP 2000 NLLINK elements. The beam elements supported on the NLLINK element are connected to the lip element. The lip is modeled with shell elements with a similar membrane thickness as the shear wall. The bottom edge of the lip element which does not lie over the shear wall is supported on the roller supports (Figure 3.3) or on the spring supports (Figure 3.4) at each node. These roller and spring supports in reality represent the stud wall. The overall configuration represents the actual lateral load resisting system which is developed in one plane for simplicity in analysis and modeling.

### **3.4 Results and Discussion**

A unit lateral force is applied on the left topmost node of each finite element model. For analysis, the stud wall is modeled in the following three different ways:

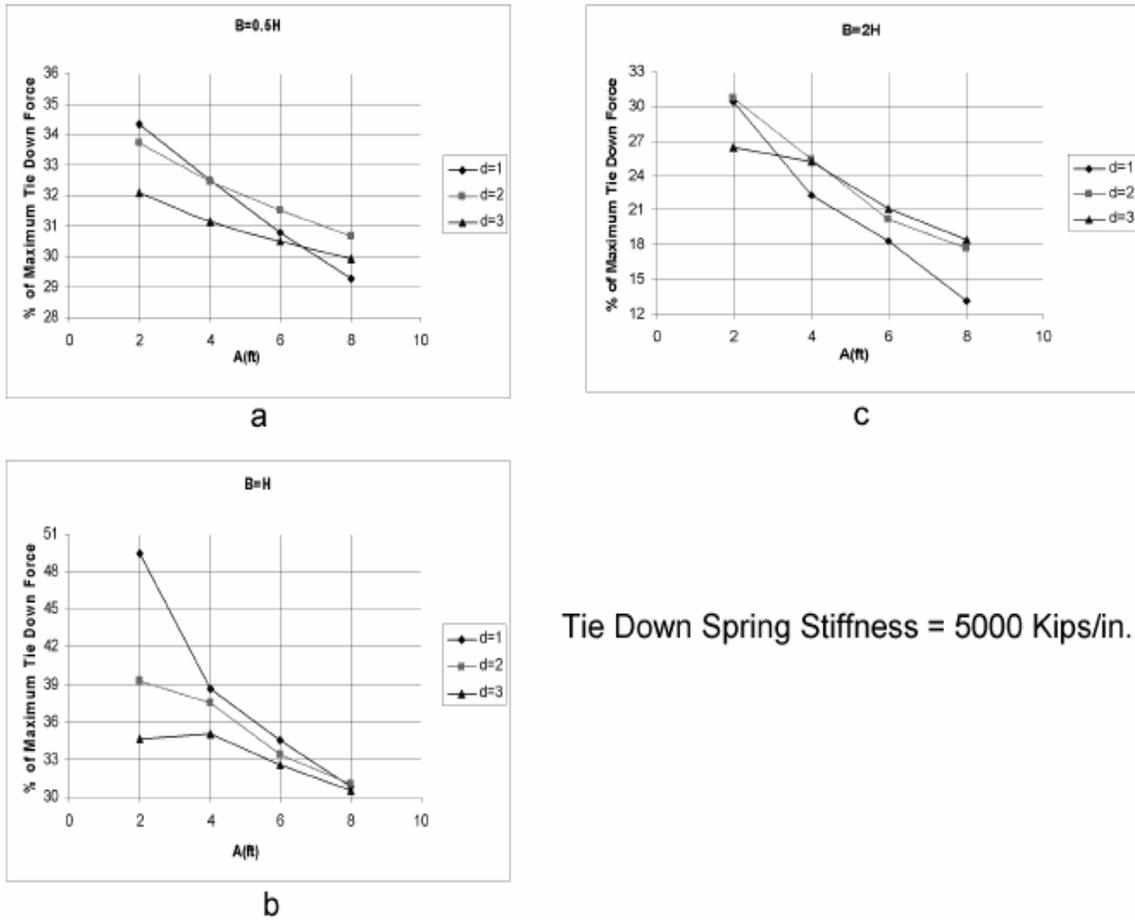
- 1.) As a roller support.
- 2.) As a spring attached at each unsupported bottom node with stiffness = 76 kips/in.
- 3.) As a spring attached at each unsupported bottom node with stiffness = 19 kips/in.

The spring stiffness in case 1 is calculated using the formula for the equivalent rectangular truss member of length  $H$  and width equal to the distance between any two adjacent nodes (12 in.). The thickness of the truss member is taken as 0.4375 in. (which is the same as the thickness of the shear wall and lip element). The modulus of elasticity is taken as 1600 kips/in<sup>2</sup>. The spring stiffness in case 3 is taken as one-fourth of the spring stiffness in case 2.

#### **Case 1:**

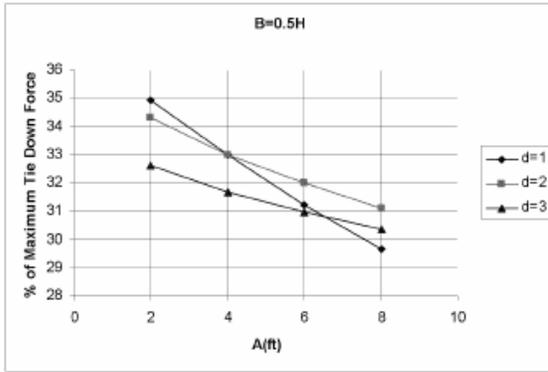
The plots for this case are shown in Figures 3.6, 3.7 and 3.8. The stud wall in this case is modeled as a roller support. As the stiffness of the tie down springs is increased from 5,000 kips/in. to 50,000 kips/in. and then to 500,000 kips/in. there is no significant change in the ratio of tie down forces. In all the plots, we note that there is a decrease in the ratio of actual tie down force as we increase the spacing between the shear walls.

This remains true for all the cases (i.e.,  $d = 1$  ft,  $d = 2$  ft and  $d = 3$  ft) as we move from  $B/H = 0.5$  to  $B/H = 2$ . The maximum percentages of tie down forces for all the cases are obtained for  $B/H = 1$  and this is evident from the Figure 3.6(b).

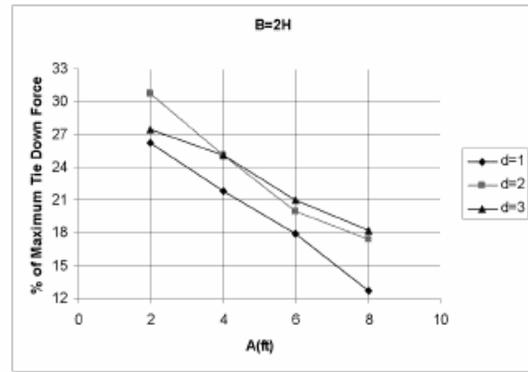


Tie Down Spring Stiffness = 5000 Kips/in.

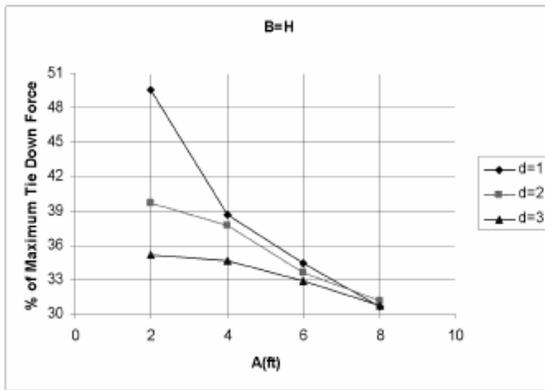
Figure 3.6: Stud wall modeled as roller support and tie down spring stiffness = 5,000 kips/in.



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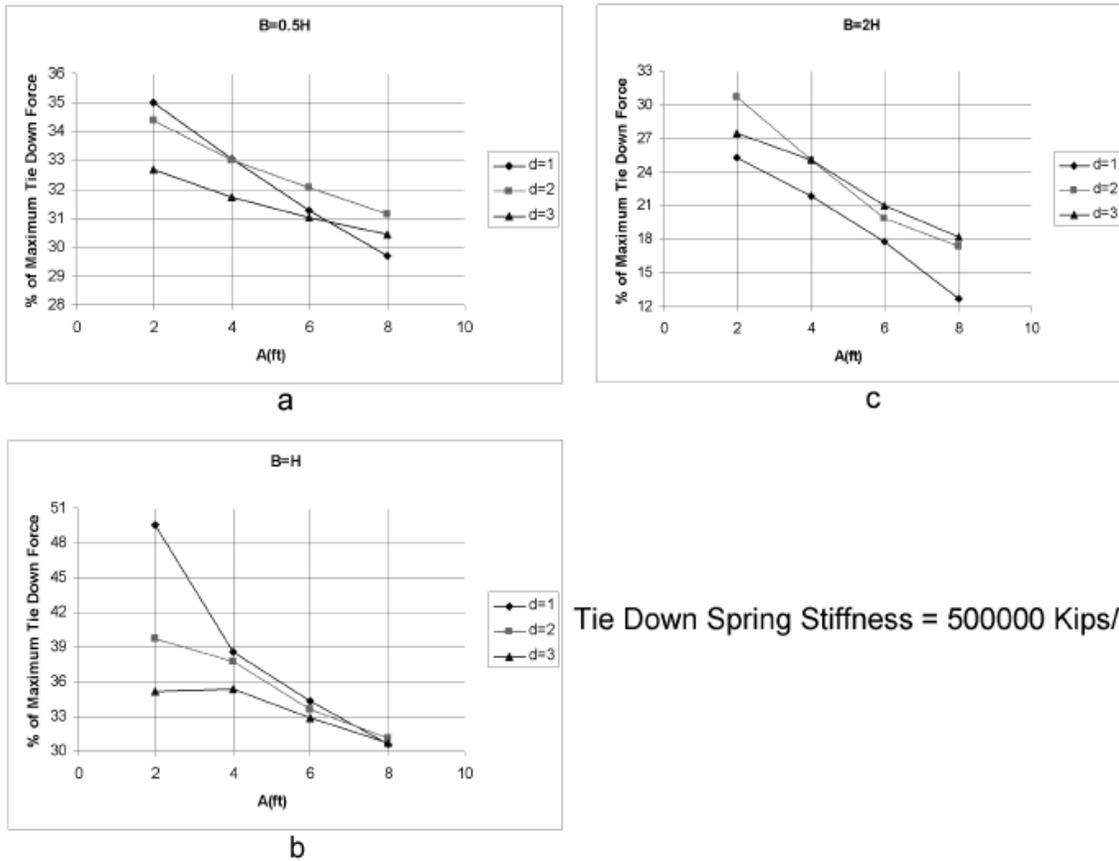
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Tie Down Spring Stiffness = 50000 Kips/in.

Figure 3.7: Stud wall modeled as roller support and tie down spring stiffness = 50,000 kips/in.



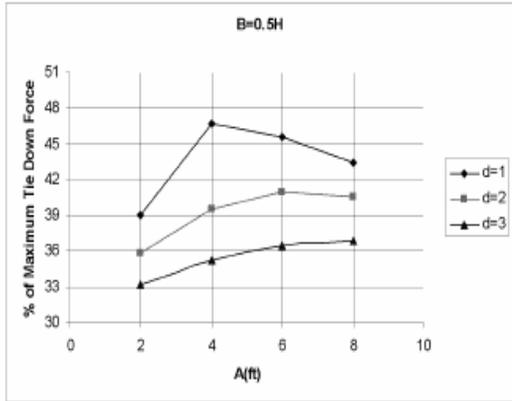
Tie Down Spring Stiffness = 500000 Kips/in.

Figure 3.8: Stud wall modeled as roller support and tie down spring stiffness = 500,000 Kips/in.

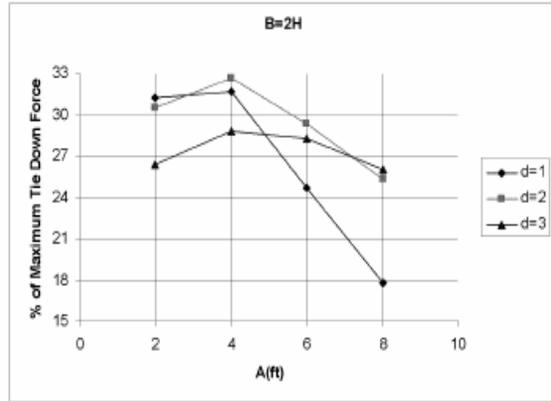
**Case 2:**

The plots for this case are shown in Figures 3.9, 3.10 and 3.11. The stud wall in this case is modeled as a spring support with stiffness equal to 76 kips/in. As we increase the stiffness of the tie down springs from 5,000 kips/in. to 50,000 kips/in. and then to 500,000 kips/in. there is no significant increase in the tie down force. The behavior of the tie down anchor force is very different to the one observed in the previous case.

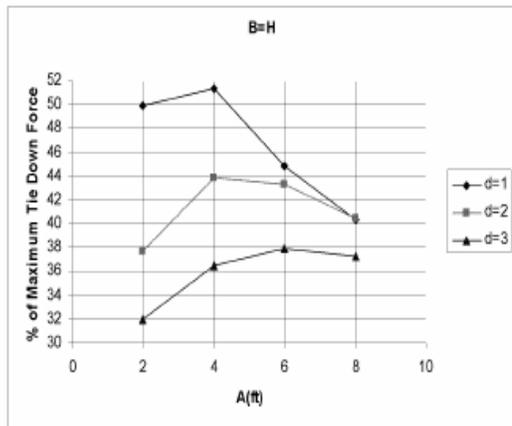
It is found that there is an initial increase and then a decrease in the ratio of actual tie down force as the spacing is increased. This is due to the upward reaction in the additional springs (stud wall area increased), followed by downward reaction in the further additional springs coming between the shear wall as the spacing (A) increases. Upward reaction followed by downward reactions causes the overturning moment going in the shear wall to increase and then decrease respectively. This remains true for all the cases (i.e., d = 1 ft, d = 2 ft and d = 3 ft) as we move from B/H = 0.5 to B/H = 2.



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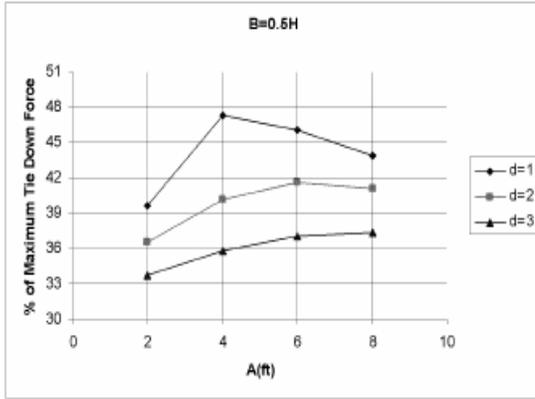
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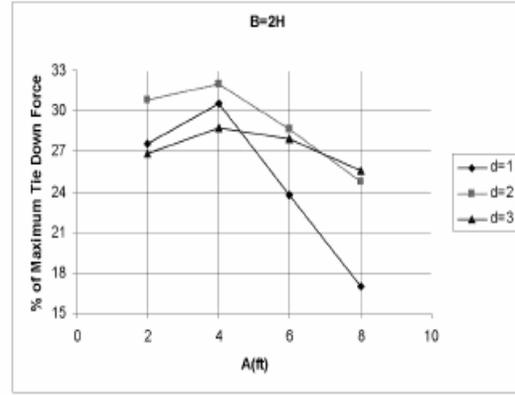
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Tie Down Spring Stiffness = 5000 Kips/in.

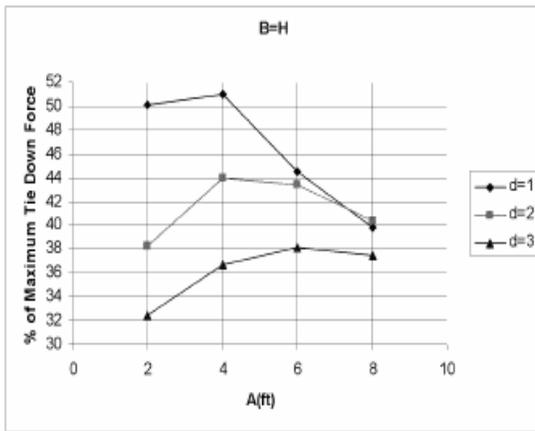
Figure 3.9: Stud wall modeled as spring support with stiffness = 76 kips/in. and tie down spring stiffness = 5,000 kips/in.



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Tie Down Spring Stiffness = 50000 Kips/in

Figure 3.10: Stud wall modeled as spring support with stiffness = 76 kips/in. and tie down spring stiffness = 50,000 Kips/in.

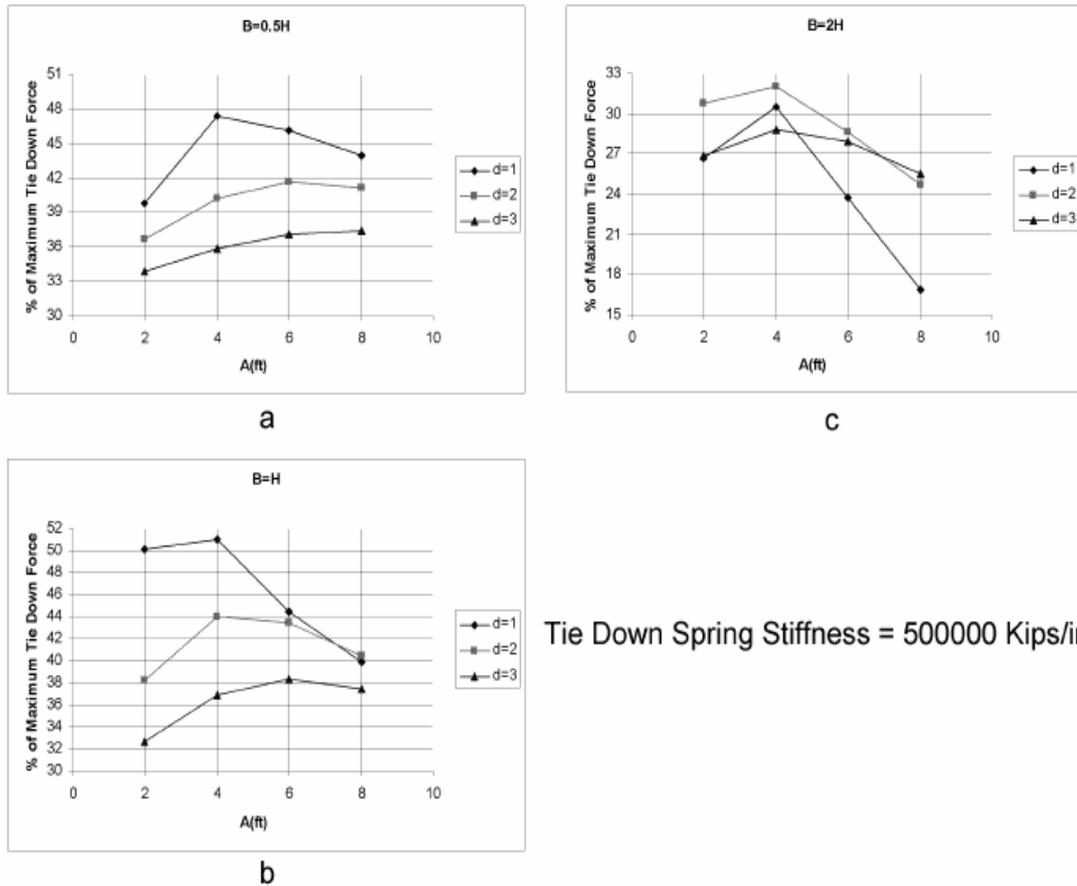
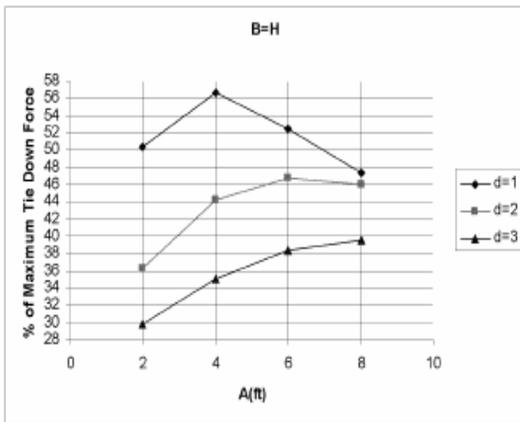
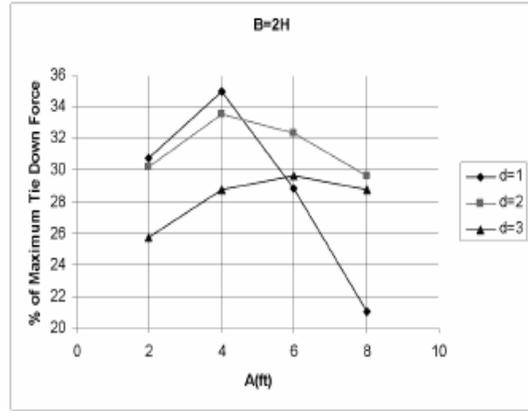
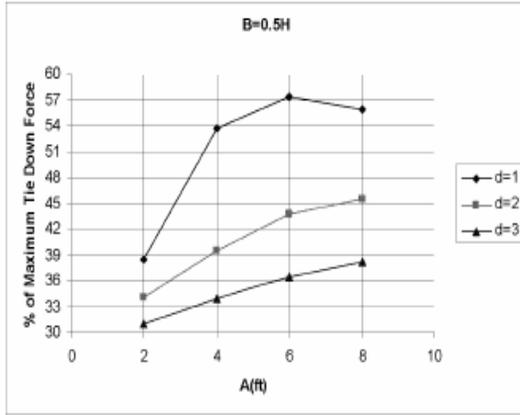


Figure 3.11: Stud wall modeled as spring support with stiffness = 76 kips/in. and tie down spring stiffness = 500,000 kips/in.

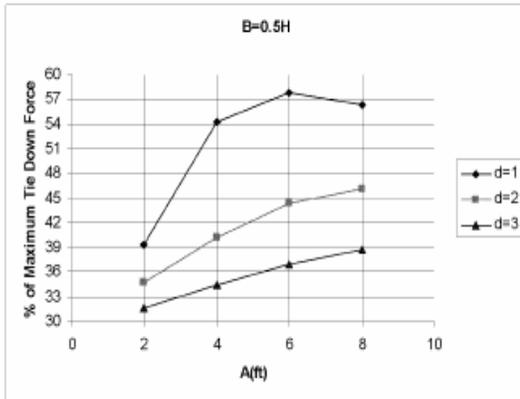
**Case 3:**

The plots for this case are shown in Figures 3.12, 3.13 and 3.14. The stud wall in this case is modeled as a spring support with stiffness equal to 19 kips/in. The behavior of the tie down forces remains similar to the plots in Case 2, except the percentage of actual tie down forces increases a bit further as the flexibility of the stud wall in the vertical direction has increased.

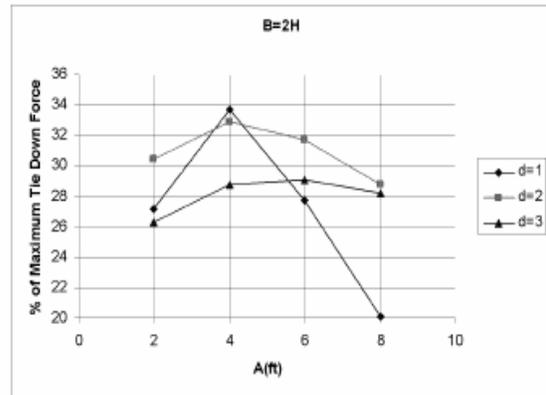


Tie Down Spring Stiffness = 5000 Kips/in.

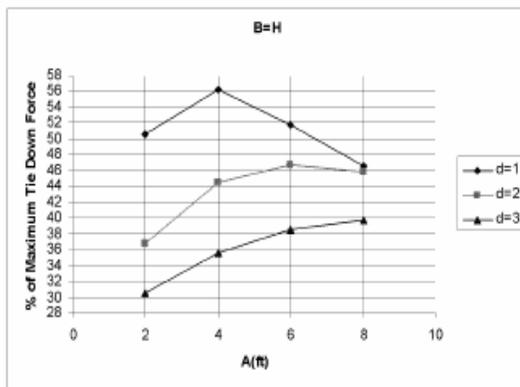
Figure 3.12: Stud wall modeled as spring support with stiffness = 19 kips/in. and tie down spring stiffness = 5,000 kips/in.



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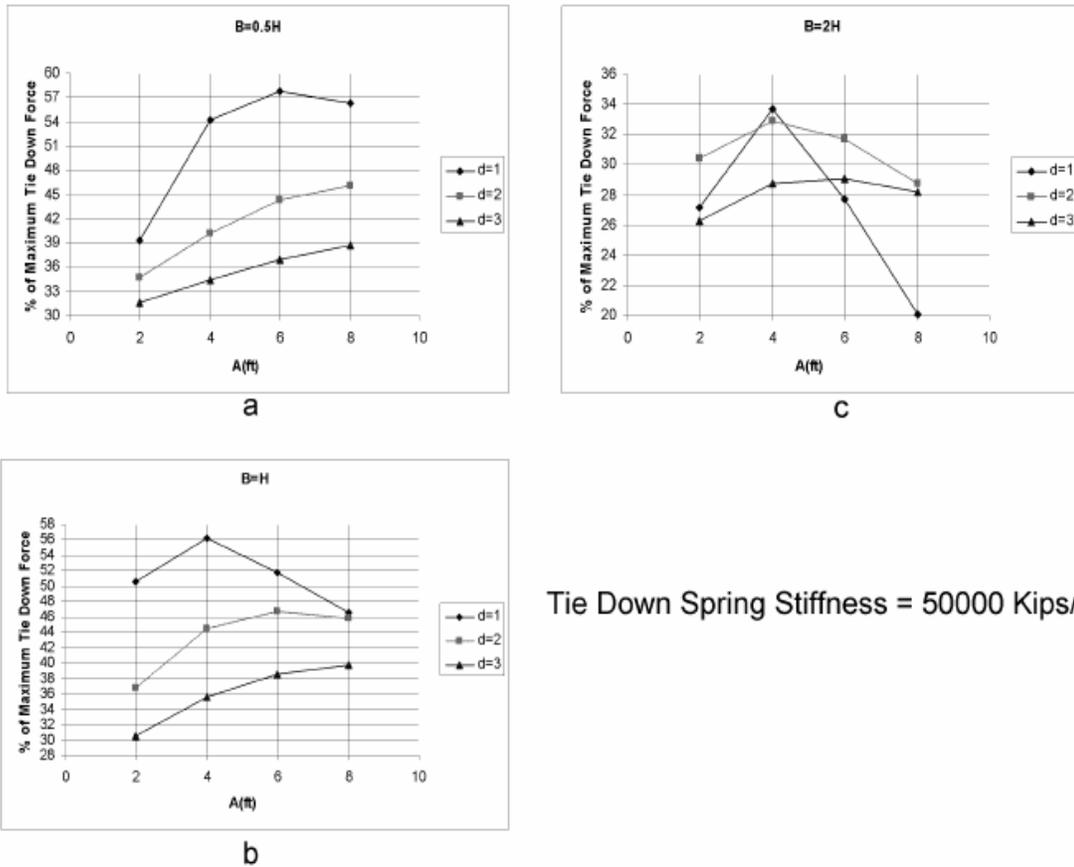
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Tie Down Spring Stiffness = 50000 Kips/in.

Figure 3.13: Stud wall modeled as spring support with stiffness = 19 kips/in. and tie down spring stiffness = 50,000 kips/in.



Tie Down Spring Stiffness = 50000 Kips/in.

Figure 3.14: Stud wall modeled as spring support with stiffness = 19 kips/in. and tie down spring stiffness = 500,000 kips/in.

Also it is found that as we move from case 1 to case 3 there is an increase in the tie down force which is due to an increase in the vertical flexibility of the stud wall, which increases the tie down anchor forces.

All the structures were modeled with several varying independent parameters. The study of the plots in the appendices shows that modeling of the stud wall as discussed in the three cases above will not give 100 % of the maximum tie down force. The maximum and minimum percentage of tie down force obtained for all the above cases is 57% and 12%, respectively.

Though the more accurate approach would have used the finite element model of the entire structure, the complexity of the models and the absence of any automatic mesh generator was the main reason behind such simplification. To facilitate the generation and analysis of complete finite element models of houses, an automatic mesh generator

and a solver program, named WoodFrameMesh and WoodFrameSolver respectively, were developed at Virginia Tech. A discussion of the development of these programs and their features is made in the following chapters.