EFFECT OF FLEXURAL STIFFNESS OF END BEAM ON THE FAILURE BEHAVIOR OF STEEL SHEAR WALLS UNDER CYCLIC LOADING

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ABSTRACT :

Steel plate shear walls are known as suitable systems for resisting lateral loads. In the last few decades, they have been used in the construction of new buildings and retrofit of existing buildings. In this paper, the behavior of a true-scale, single-bay single-story panel was investigated through a three-dimensional nonlinear finite element analysis. The finite element models include a steel shear wall with different end beams. For the loading (displacement) time history, the ATC-24 guidelines have been used. This loading has applied to the top nodes of the beams. Various parameters such as initial stiffness, strength, and energy absorption were obtained and the effect of the flexural stiffness of the end beam on these parameters was investigated and discussed. The results show that the cyclic behavior of steel shear walls is affected by the stiffness of the end beam. If this stiffness is low, the diagonal tension field cannot be performed properly. Therefore the ductility and energy absorption capacity of the panel is reduced.

KEYWORDS: Steel shear wall, failure behavior, cyclic loading, end beam.

1.INTRODUCTION

Steel plate shear walls are an innovative lateral load resisting system capable of effectively bracing a building against both wind and earthquake forces. The system consists of a vertical steel infill plate one story high and one span wide connected to the surrounding beams and columns. The plates are installed in one or more spans for the full height of a structure to form a stiff cantilever wall. Steel plate shear walls are well-suited for new construction, and they offer a relatively simple means for the seismic upgrading of existing steel or concrete structures.

The most important advantages of this system are: high stiffness and strength, suitable ductility and energy absorption, stable behavior at large deformation, easy construction and low cost with respect to other lateral load resisting systems. During the last 3 decades, several experimental and theoretical studies have been made.

Steel shear walls can be either stiffened or unstiffened plates. In the unstiffened case, the plate buckles and sustains the lateral load via tension field action. In this paper an unstiffened single-story single span surrounded by beams and columns is modeled using the finite elements method by ANSYS software.

2.FINITE ELEMENT MODELLING AND COMPARING WITH EXPERIMENTAL RESULTS

In the finite element modeling of frame members (beam and columns), the BEAM188 element is used. This element is based on the Timoshenko beam theory and shear deformation effects are included. BEAM188 has 2 nodes and can be linear or quadratic in 3-D. It has 6 degrees of freedom at each node (3 translations and 3 rotations). For modeling the shear plate, the SHELL181 element is used. This element is suitable for analyzing thin to moderately-thick shell structures. It is a 4-node element with 6 degrees of freedom at each node (as the

BEAM188 element) [1]. The material and geometric nonlinearity is considered in the analysis.

The columns are fully fixed at their bases, and beam to column connections are considered rigid. Moreover, the steel plate is connected to its surrounding members directly and rigidly. The top beam is protected against out-of-plane deformation by constraining U_z in all of its nodes.

In order to certify the correctness of modeling and analysis, a finite element model was built and the results were compared with the results of two experimental studies. The first experiment has been done at the university of British Columbia, Canada [2] where the specimen was a single-story, single-bay structure. The structure was modeled by the finite element method. A gradual horizontal displacement up to 50 mm was applied at floor level of the structure, as in the test. The story shear versus displacement from the test and finite element models are shown in figure 1. This figure shows good agreement between test and finite element modeling.

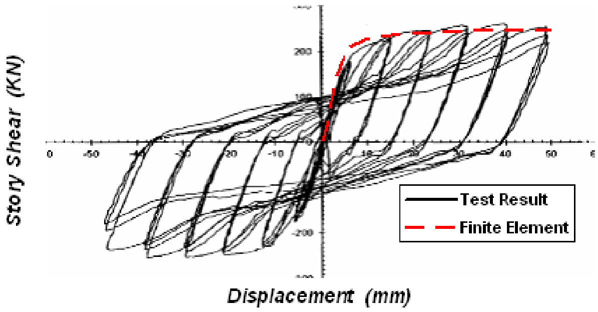

Figure 1 Comparison of FE analysis with the results of single story test specimen [2]

The second test was performed by Elgaaly et.al. This structure was a single-bay, three stories ([3] & [4]). The loading was a gradual horizontal displacement up to 40 mm applied at the third floor. This structure and loading was also modeled by finite element. The results are shown in figure 2. This figure shows good agreement between test and finite element modeling. A small difference is acceptable and due to the difference between test arrangement and finite element model.

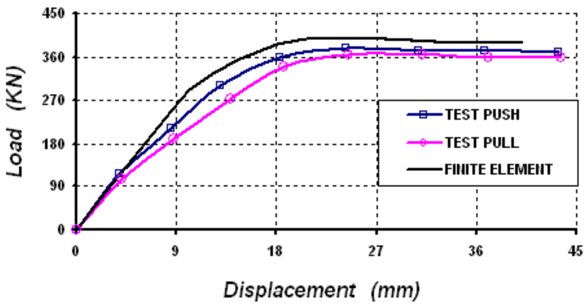

Figure 2 Comparison of FE analysis with the results of three story test specimen [3]

3.LOADING AND FAILURE CRITERIA

All of the analytical cases have been analyzed using ATC-24 [5] method (figure 3). According to this guideline, to define a cyclic loading history the yield deformation (δ_y) should be determined. This value may be assigned experimentally (from a monotonic load test) or predicted analytically. In order to define δ_y analytically, the specimens were analyzed in a force-controlled manner and the yield deformation values of each of them were achieved. Other parameters of figure 3 are: $n_0=6$, $n_1=n_2=n_3=3$, $n_1=n_1=\cdots=2$, $\Delta=\delta_y$.

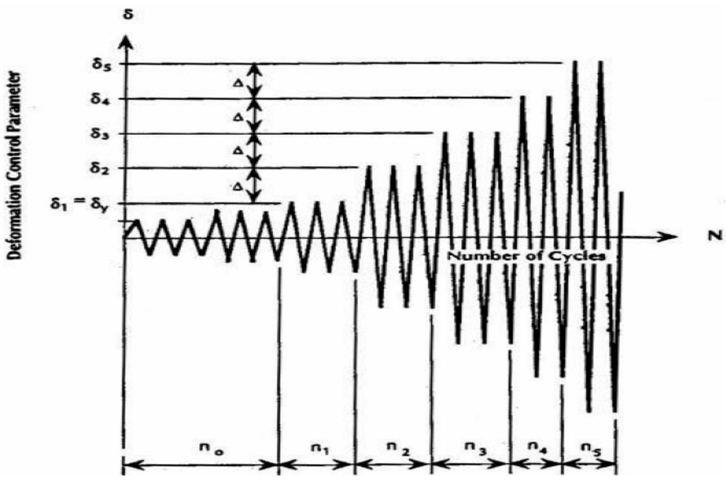

Figure 3 Deformation history for multiple step test [5]

In order to terminate the analysis procedure, a failure criterion should be defined. Several criteria such as fracture at connections, buckling of columns, yielding of frame members and fractures of infill plate may be considered. In this research, a decrease in the strength of the specimen at a specific "stepwise increasing deformation" relative to its previous stepwise has been assumed as the failure criteria. For example, the envelope of the load versus displacement for one of the specimens is shown in the figure 4. In this case, the strength (load) has decreased at step 10 relative to 9; therefore the analysis has been terminated at step 10 and the maximum load is considered as the value of step 9.

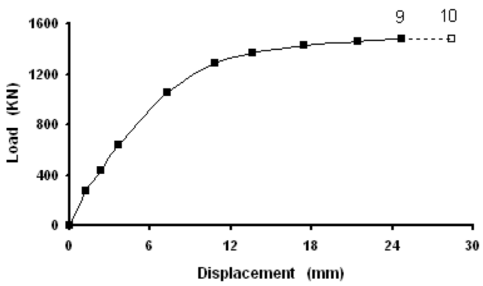

Figure 4 Envelope diagram of one of the specimens and assumed failure criteria

4. MATERIAL PROPERTIES

The stress-strain relationships of frame members and the infill plate are assumed to be bilinear (strain hardening). The modulus of elasticity of initial and secondary parts are considered $E_1 = 206(KN\ /mm^2)$ and $E_2 = 2.06(KN\ /mm^2)$, respectively. The yield stress is assumed $F_Y = 0.235(KN\ /mm^2)$. These properties represent the behavior of mild steel.

The Von Misses yield theory, which is suitable for mild steel, is used for the material yield criterion and a kinematics hardening rule is used to simulate the Bauschinger's effect in the cyclic runs. Figure 5 shows the pattern of the models.

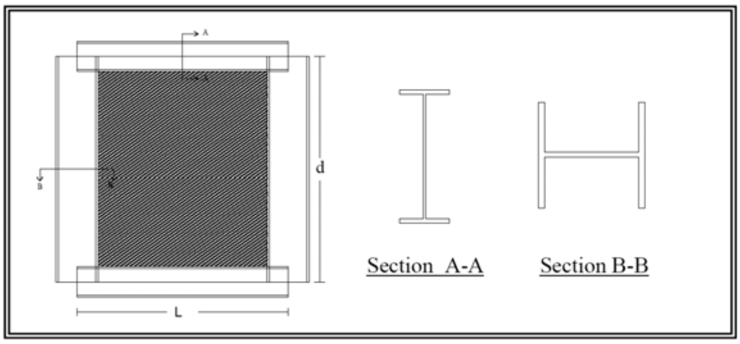

Figure 5 Pattern of the models

In order to present the results simply, a non-dimensional parameter named stiffness parameter (SP) for beams and columns has been defined as follows:

$$SP=\frac{EI/L}{D}$$

(1)

Where E is the modulus of elasticity of beams and columns, I is the moment of inertia of beam and columns, L is the length of the beam, and D is the stiffness coefficient of the plate:

$$D=\frac{Et_p^3}{12\left(1-\upsilon^2\right)}$$

(2)

Where E is the modulus of elasticity of plate, t_p is the thickness of plate and taken 5 mm. υ is the Poisson's ratio and taken 0.3.

5. EXAMPLES

To investigate the flexural stiffness effects of the beam, four cases are considered. The purpose of case A is the selection of sections in case B, C and D, so that the section of beam is taken constant and the optimum section of columns can be determined. In this case the span length of beam (L) is considered 2000 mm, and the height of column (d) is considered 3000 mm. The properties of case A are shown in table 1.

In cases B, C, and D the section of columns is maintained constant while the section of beam varies. The length and height of case B is the same as case A, the length and height of case C is 3000 mm, and the length and height of case D are 4500 mm and 3000 mm respectively. The properties of these cases are shown in tables 2.

Table 1 properties of case A

No.	Beam Section	Column section	Column moment of inertia (mm^4)	SP(column)
ASSW1	IPE400	HEB550	$133.085*10^7$	38755
ASSW2	IPE400	HEB500	$104.255*10^7$	30359
ASSW3	IPE400	HEB450	$77.555*10^7$	22584
ASSW4	IPE400	HEB400	$55.871*10^7$	16270
ASSW5	IPE400	HEB360	$41.756*10^7$	12159
ASSW6	IPE400	HEB340	$35.384*10^7$	10304
ASSW7	IPE400	HEB320	$29.707*10^7$	8651

Table 2 properties of case B

No.	Beam Section	Column section	Column moment of inertia (mm^4)	SP(beam)
BSSW1	HEB360	HEB450	$32.14*10^7$	14309
BSSW2	HEB360	HEB400	$21.876*10^7$	9555
BSSW3	HEB360	HEB360	$15.524*10^7$	6781
BSSW4	HEB360	HEB330	$11.145*10^7$	4868
BSSW5	HEB360	HEB300	$7.999*10^7$	3494
CSSW1	HEB360	HEB500	$46.207*10^7$	13455
CSSW2	HEB360	HEB450	$32.14*10^7$	9359
CSSW3	HEB360	HEB400	$21.876*10^7$	6370
CSSW4	HEB360	HEB330	$11.145*10^7$	3245
CSSW5	HEB360	HEB300	$7.999*10^7$	2329
DSSW1	HEB360	HEB400	$21.876*10^7$	4247
DSSW2	HEB360	HEB450	$32.14*10^7$	6239
DSSW3	HEB360	HEB500	$46.207*10^7$	8970
DSSW4	HEB360	HEB550	$63.965*10^7$	12418
DSSW5	HEB360	HEB600	$88.236*10^7$	17147

5.RESULTS

5.1. Results Of Case A

Four samples of the hysteresis curves of case A are shown in figure 6. Figure 7 shows the maximum load-displacement, initial stiffness, strength, and energy dissipation of this case. It should be noted that the maximum load diagram is obtained by the maximum load in each cycle at the hysteresis curve, initial stiffness is considered as the initial slope of this curve, the strength is considered as the maximum load in the hysteresis curve, and energy dissipation is the shown by the area of the hysteresis loop. As shown in these figures, increasing the column stiffness causes an increase in the initial stiffness and strength and improves the behavior of the steel shear wall. If an increase in the column stiffness exceeds a specific amount, the energy dissipation decreases.

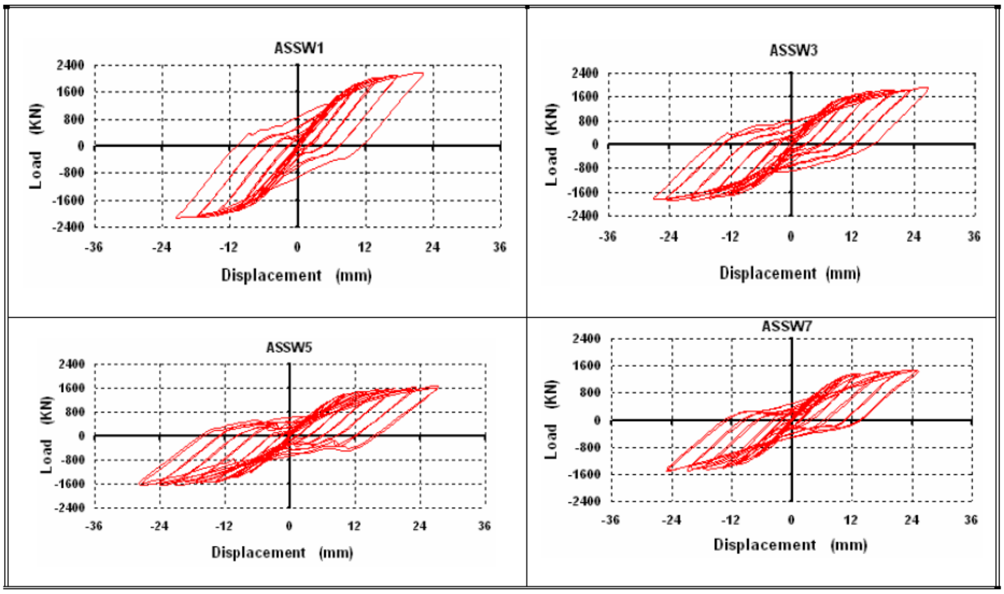

Figure 6 sample hysteresis curves of case A

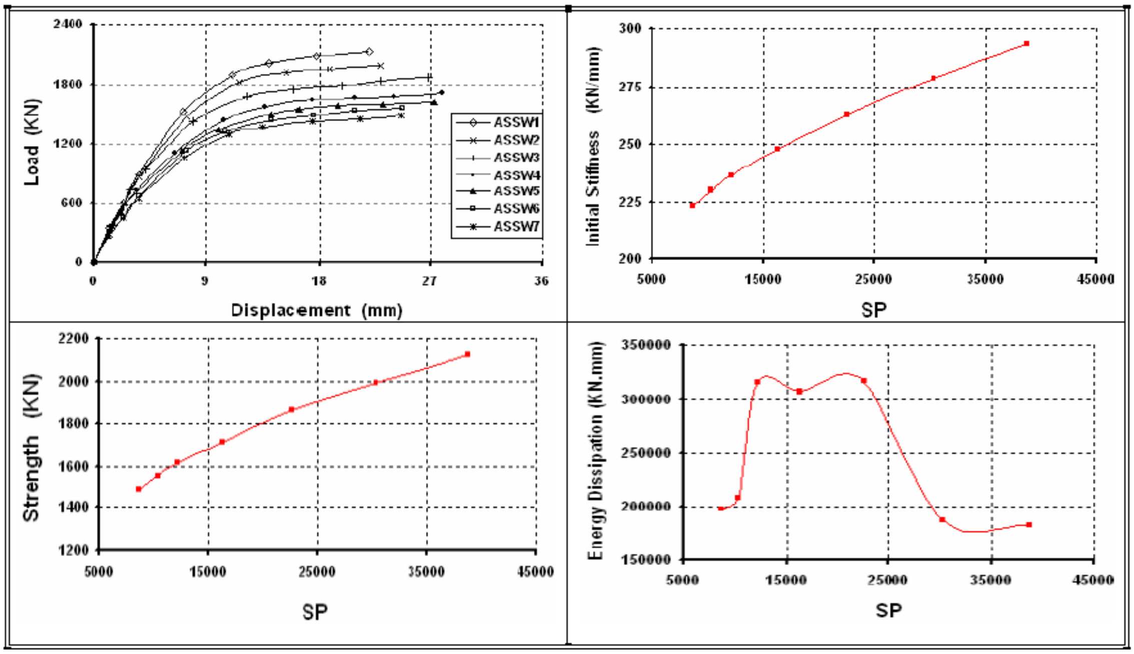

Figure 7 maximum load, initial stiffness, strength, and energy dissipation of case A

5.2. Results Of Case B

The hysteresis curves of case B are similar to case A and not shown in this paper. The maximum load-displacement, initial stiffness, strength, and energy dissipation are shown in figure 8. As shown in this figures, an increase in the stiffness of beams causes an increase in the stiffness and strength and improves the energy dissipation of the structure. This is due to the improvement of tension field action.

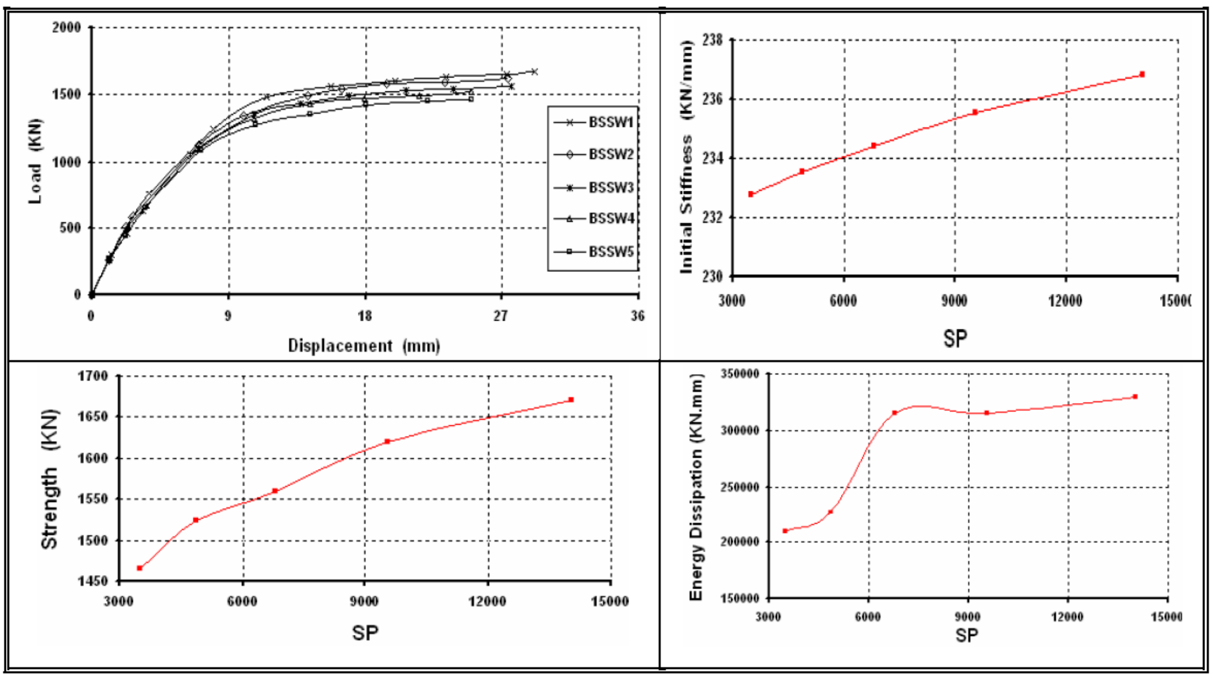

Figure 8 maximum load, initial stiffness, strength, and energy dissipation of case B

Figure 9 shows the deformation of the BSSW1 specimen at the end of the 24th cycle where failure occurs. In this case the base of the right column has failed.

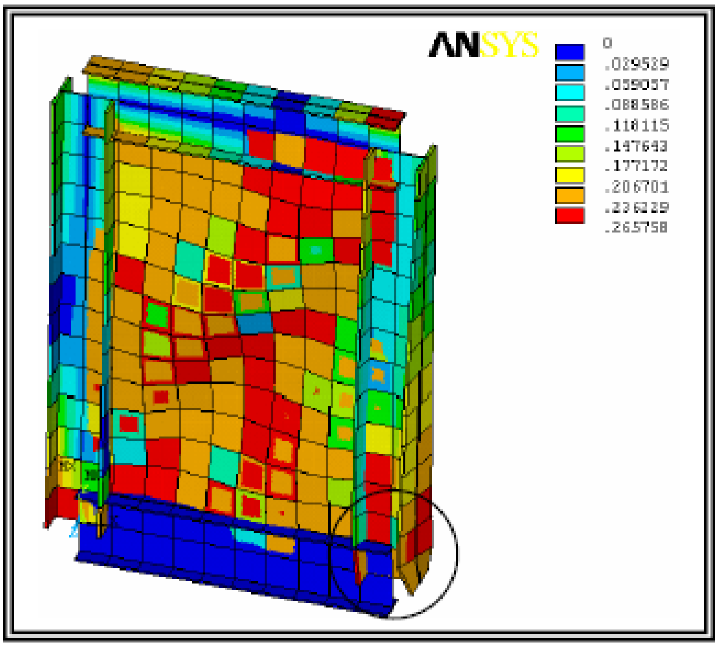

Figure 9 The failure state of one sample of case B

5.3. Results Of Case C

The results of case C are shown in figures 10. It can be seen that the seismic behavior of the structure will be improved by increasing the flexural stiffness of the beam. The stronger the beam, the more energy dissipation the structure can sustain.

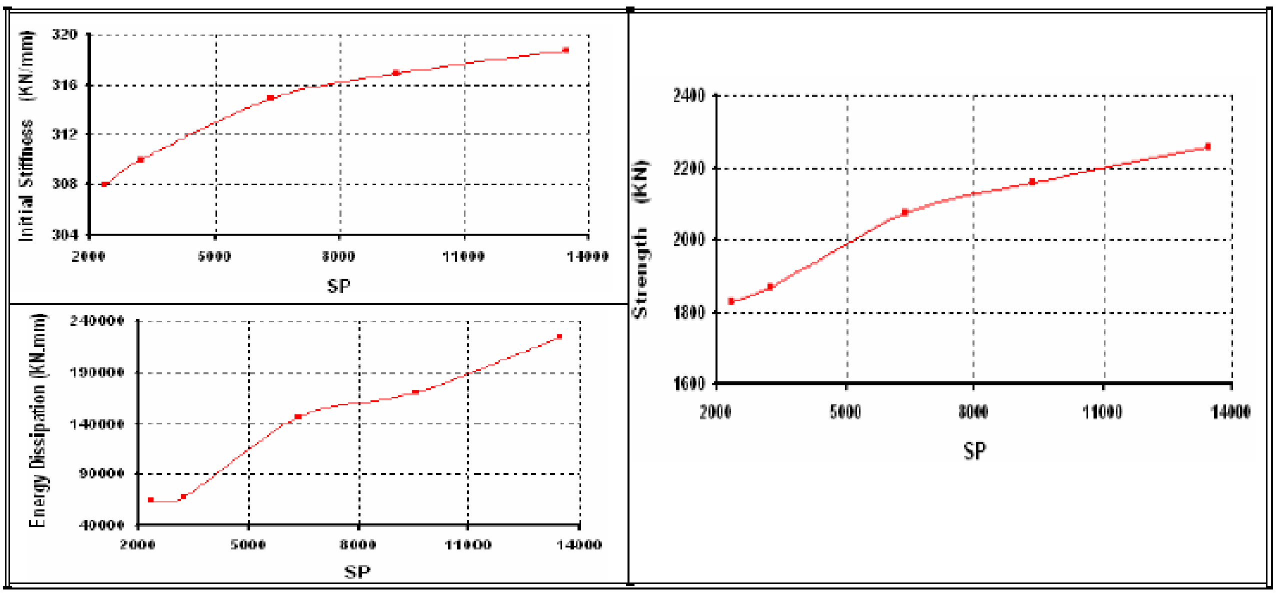

Figure 10 maximum load, initial stiffness, strength, and energy dissipation of case B

4.4. Results Of Case IV

Results of this case are similar to case III and not shown in this paper. As in case III, the seismic behavior and energy dissipation of the structure will be improved by increasing the flexural stiffness of the beam. Figure 11 shows a case where all properties of the specimens are similar except the span length (2000, 3000, 4000 mm, respectively). This figure shows that increasing the beam length increases the stiffness and strength of the shear wall.

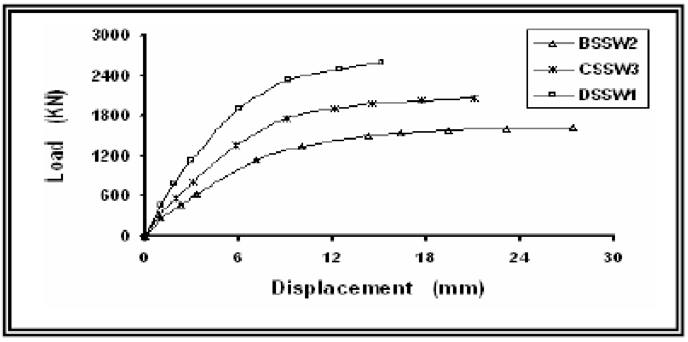

Figure 11 Effect of span length

6.CONCLUSION

In this paper the effect of beam flexure on the failure behavior of the steel shear wall is investigated. Four different cases were analyzed with ANSYS software. The following results are concluded from the analysis:

- Increasing the flexural stiffness of beams will increase the initial stiffness of the models, and the diagonal tension field will be performed properly. Also, the energy absorption capacity of the steel shear panels will be increased. If the flexural stiffness of end beams increases from a specified value, the stiffness will not be affected considerably.
- Increasing the aspect ratio of panels will increase the initial stiffness and strength considerably. If the end beams have adequate flexural stiffness, increasing the aspect ratio causes an increase in absorbed energy.
- It seems that if the ratio of moment inertia to the height of the end beam's section (I_b/h) is more than $L^2/9$ (millimeter), the diagonal tension field can be perform properly and flexural stiffness of end beams will not affect the energy absorption of the structure. It should be noted that this conclusion was made from a steel shear wall with 5 mm thickness.
- Increasing the column stiffness causes an increase in the initial stiffness and strength and improves the behavior of the steel shear wall. If the increase in the column stiffness exceeds a specific amount, the energy dissipation decreases.

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