PERFORMANCE OF METAL FOAM COMPOSITE PANEL SHEAR WALL

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in

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By

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May,2017

CERTIFICATE

This is to certify that project report entitled "**Performance of Metal foam Composite Panel Shear Wall**", submitted by **Shital Singh Sombria** in partial fulfillment for the award of degree of Master of Technology in Structural Engineering to Jaypee University of Information Technology, Wakhnaghat, Solan has been carried out under my supervision and guidance. This work has not been submitted partially or fully to any other University or Institute for the award of this or any other degree or diploma.

The above statement made is correct to the best of my knowledge.

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ABSTRACT

Steel plate shear wall system consists of steel infill plates connected to the boundary elements. Steel plate shear walls possess excellent initial stiffness, high ductility, redundancy, stable hysteresis loops and energy absorption capacity. These properties enable Steel plate shear walls as preferred choice for lateral load resisting system to resist high wind and seismic forces. Metallic form sandwich panels consist of thin steel/aluminum facing sheets attached to a light weight core and have high stiffness, ductility and energy absorption capacity which could be used in lieu of steel infill plates.

This study investigates the potential to use metallic form sandwich panels in lieu of conventional steel infill plates in the steel shear wall system. Numerical modeling of steel plate shear walls both using conventional steel plates and using metallic foam sandwich panels was conducted. 18 models were prepared (9 numbers steel plate and 9 numbers sandwich panels) by varying aspect ratios and storey heights to cover maximum possible cases. Non-linear monotonic push over analysis was used to evaluate the ultimate load behavior of steel plate shear walls.

The results indicated that metallic foam sandwich panels are excellent substitute for conventional steel plate shear walls. It has been observed that the use of sandwich infill plates improved the performance of steel plate shear walls and is an excellent prospect for lateral load resisting system.

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1. INTRODUCTION

Metal foams are comparatively recent material light in weight and with exceptionally good physical, mechanical, energy absorption, electric, acoustic, and thermal properties. Metal foams generally consist of two metal sheets bonded to a light weight foam core. Composite sandwich panels made up of metal foams have found their use in numerous applications. Steel plate shear walls (SPSW) comprises of steel infill plate surrounded by steel beams and columns called boundary elements. Steel plate shear walls have a high degree of redundancy and ductility and these properties make Steel plate shear walls favorable for lateral load resistance for structures coming under high seismic zones. Since metallic foams possess extremely good energy absorption capacity and are also light in weight, the use of metal foams instead of regular steel plate would improve the overall system performance. This report discusses the effectiveness of using metallic foam sandwich panel as SPSW instead of typical steel infill plates.

1.1 Steel plate shear walls

Steel plate shear walls are made up of steel infill plates bounded by columns and beams and are used as lateral load resisting system. The columns are called as vertical boundary elements (VBE) and steel beams at floor levels called as horizontal boundary elements (HBE). Conventional arrangement is as shown in the figure 1.

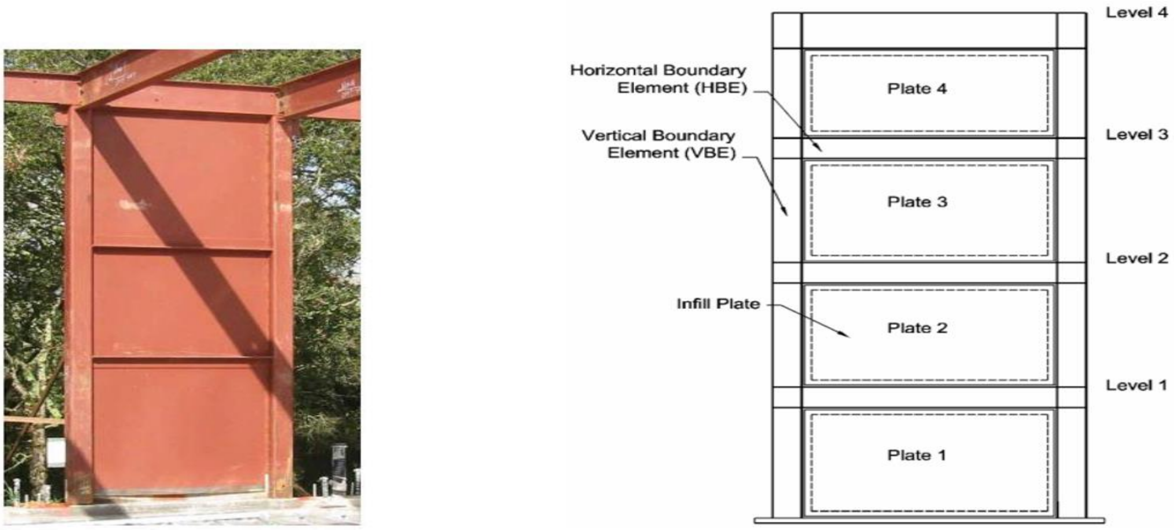


Figure 1.1 –Conventional Steel plate shear wall [1]

1.2 Action of Steel plate shear walls

SPSW behavior is identical to that of a steel plate girder. Steel plate buckles (shear buckling) under the load and after that the load carrying capacity is decided by the tension field action. Ideally, the boundary elements should remain elastic so as to develop complete tension field action of the shear plate. This is shown in figure 2 and it also shows the development of tension field in steel plate.

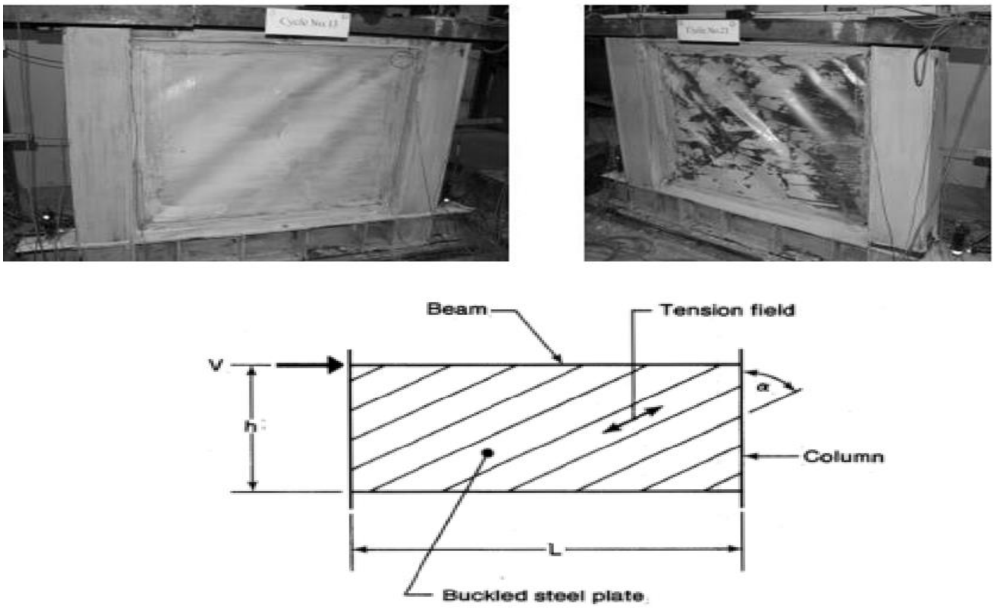


Figure 1.2 Steel plate shear wall-development of tension field action

1.3 Metal foam composite panels

Composite materials made up of two or more component materials with different properties joined together in such a way that the composite materials constituted has better properties than the individual constituents. One of the best examples is reinforced concrete. Steel is very strong in both tension and compression where concrete is weak in tension but strong in compression. Thus, the composite-reinforced concrete- formed is a versatile material with enhanced properties.

Metallic foam composite plates comprise of mainly thin steel/aluminum face sheets divided by and attached to a light weight core of foamed metal. Metallic foam sandwich panels are shown in Figure 1.3.

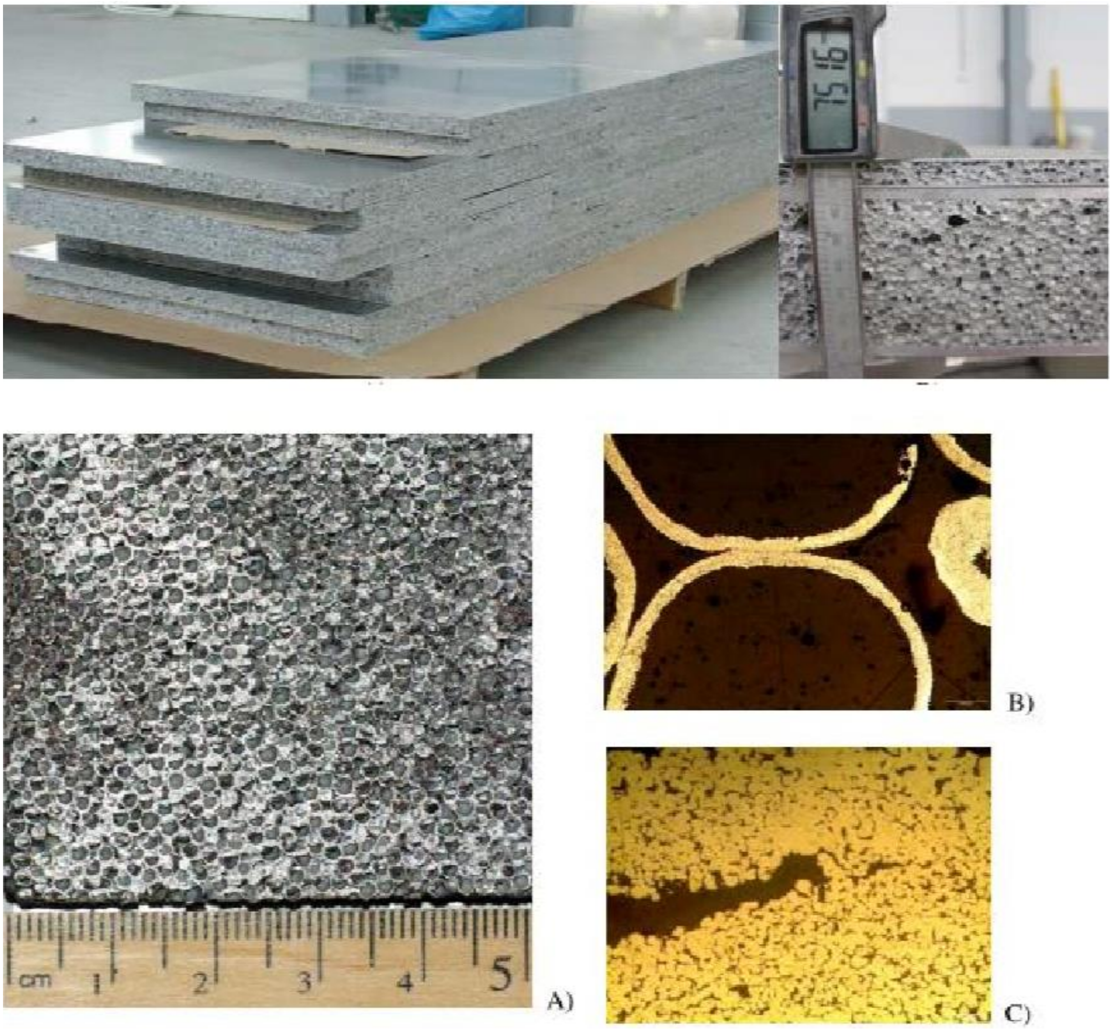


Figure 1.3 Metallic foam sandwich panels [6]

2. BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

In the recent years SPSW's have been used extensively for a variety of buildings in the countries like U.S. Canada, Japan and Mexico. Plenty of researches have been done and the design formulae and methods have been made available for design purpose. Design recommendations have been included in the AISC seismic provisions for structural steel buildings. Following research papers provide a brief review about the different method of analysis and design of the SPSW's.

2.2 Thorburn et. All (1983)

Thorburn et all gave thorough analytical investigations for unstiffened SPSW for the first time in the history of SPSW. They proved that shear buckling of the steel plates does not represent the true ultimate capacity of SPSW. There is reserve strength available after post buckling by the development of tension fields. In order to represent the tension field action, an analytical model termed strip model was developed. [2]

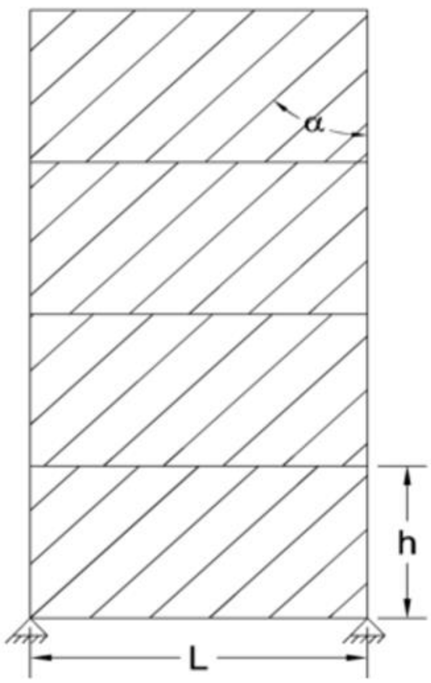


Figure 2.1 –Strip model representation of Steel plate shear wall [2]

Based on the research at University of Alberta, Canada, Thorburn et all proposed following equation for the calculation of tension field angle, α-

$$tan^4\alpha = \frac{1+\left(\frac{t\times L}{2A_c}\right)}{1+\left(\frac{t\times h}{A_b}\right)} \qquad \text{------eq 2.1}$$

where,

t = thickness of the plate,

L = span of the horizontal boundary element,

h = is the storey height,

A_c = cross sectional area of the vertical boundary element (column) and

A_b = cross sectional area of the horizontal boundary element (beam).

In the Strip model it is assumed that the boundary elements are infinitely stiff in order to fully develop the tension fields. The tension fields develop at an angle close to 45 degrees to the vertical and to represent this behavior, the infill plates are modeled as series of strip elements oriented at angle α to the vertical.

A simplified model of the SPSW considers the infill plate as equivalent steel diagonal brace of equivalent properties. Thus, SPSW in multistory buildings could be treated as Pratt truss with diagonal oriented in the direction of tension fields. The diagonal brace represents the stiffness characteristics of infill panels and the area of the brace can be calculated from:

$$A = {tL \times \sin^2 \alpha}/{2 \sin \emptyset \sin \emptyset} \qquad \text{----eq 2.2}$$

where $\emptyset$ is the acute angle of the brace from vertical boundary element.

2.3 Timler and Kulak (1983)

In order to confirm the analytical method suggested by Thorburn et .all. (1983), Timler and Kulak tested a full scale specimen-two single storey, one bay model of steel plate shear wall elements. The specimen was loaded in an incremental manner to reach service and ultimate loads respectively. A cyclic load test with a target displacement was also performed. The tests recognized that the flexural stiffness of the columns affects the value of α. Based on the tests a modified equation was proposed as follows:

$$\tan^4\alpha = \frac{1+\left(\frac{t\times L}{2A_C}\right)}{\left(\frac{1+t\times h}{2A_b}\right)+\left(\frac{h^3}{360I_cL}\right)} \text{----eq2.3}$$

Where I_c is the second moment of the area of the column about an axis perpendicular to the panel and all other parameter defined earlier.

It was observed that in case of beams that have infill plate on one side only, and hence free to bend, such as the beam at the top of a shear wall or edge of the test specimen, the flexural stiffness of the beam also affects the value of α. Thus the equation is re-derives for the infill plate at the top of a SPSW as follows:

$$\tan^4\alpha = \frac{\left(1+(t\times L)\times\left(\left(\frac{1}{2A_C}\right)+\left(\frac{L^3}{120I_BL}\right)\right)\right)}{\left(1+(t\times h)\times\left(\left(\frac{1}{2A_b}\right)+\left(\frac{L^3}{320I_cL}\right)\right)\right)} \text{---eq2.4}$$

Where I_b second moment of the area of the beam about an axis perpendicular to the panel and all other parameters defined earlier.

It was recommended that equation 2.3 be used to predict more accurately the angle of tension field, α.

2.4 Elgaaly, Caccese and Du (1993)

Elgaaly et all used finite element modles, and models based on the revised multi-strip method proposed by Timle and Kulak, to replicate results experimentally achieved by Ceccese et all. Column to beam connections were assumed as moment resisting connections. Lateral load was applied monotonically until loss of stability developed due to column plastic hinging and local flange buckling. It was observed that walls with thicker plates were not significantly stronger because column yielding was the governing factor for both cases. Using an elastic perfectly plastic stress-strain curve for the strips, the model was found to produce reasonable agreement with experimental results with respect to initial stiffness, ultimate strength and displacement at ultimate loads. [2]

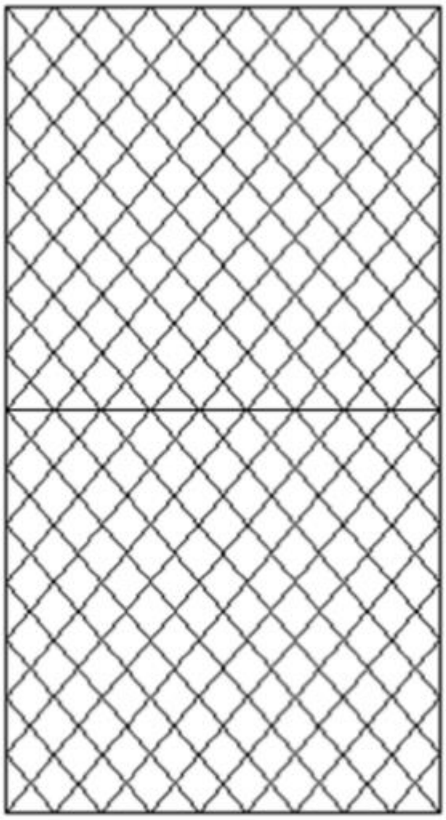


Figure 2.2 –Strip model representation of Steel plate shear wall for cyclic loading [2]

An analytical model for predicting hysteretic cyclic behavior of unstiffened thin pate shear walls was also developed. This model incorporates strips in two directions as shown in figure 2.2 to capture the cyclic behavior. The hysteretic model involved the use of an empirically derived, hysteretic, stress-strain relationship for the strips and was in good agreement with test results. [2]

2.5 Xue and Lu (1994)

Xue and Lu carried out an analytical study on a three bay, 12 storey moment resisting frame which had middle bay infilled with SPSW. The main focus of the project was to study the effect of beam to column and plate connections.

Following scenarios were considered:

1. Moment resisting beam to column connections and infill plates fully connected to the surrounding frame.

2. Moment resisting beam to column connections and infill plates connected to only at the beam interfaces.

3. Simple beam to column connections and infill plates fully connected to the surrounding frame.

4. Simple beam to column connections and infill plates only connected to the beams.

The finite element analysis model considered beam and columns as elastic beam elements and infill plates as elasto-plastic shell elements. Monotonic pushover analysis was conducted on each frame with forces applied at each storey. [2]

It was observed that the type of beam to column connection in the infill bay had insignificant effect on the global force-displacement behavior of the system. The connecting infill plates to the columns provided only modest increase in the ultimate load capacity of the system. Xue and Lu concluded that connecting the infill plates to only columns and by using simple beam to column connections in the interior bay drastically reduced the shear forces in the interior columns and assisted to avoid premature failure of the columns. However because of the limited number of analytical models a generalization of the observation is not possible. [2]

2.6 Driver et all (1997)

Driver et all tested a large scale four storey single bay SPSW specimen to identify the elastic stiffness, ductility and energy absorption capacity. The tested specimen had moment resisting beam to column connections, fishplates welded connection was used to connect the infill plates. In order to simulate the real situation gravity loading at the top of the columns were applied. Cyclic lateral loaded of equal magnitudes were applied at each floor level as per the Applied Technology Council (ATC-24) requirements. [8]

They also developed finite element models to predict the behavior of the test specimen. This analysis provided good correlation with the experimental data, but was unable to reach the full shear wall capacity.

A full response analysis was also done which gave better prediction of the ultimate strength but initial stiffness was overestimated by about 15%.

2.7 Lubell (1997)

Experiments conducted by Lubell consist of two single storey SPSW specimens and one four storey specimen. The beams to column connections were moment resisting connection in all the specimens. Steel masses were placed at each storey to simulate gravity load effects. Quasi-static cyclic testing was performed on all the specimens. For the analytical models, 15 equally spaced tension only inclined elements were used to represent infill panels and tension field angle were set to 37 degrees from the column centre line. Both monotonic and cyclic pushover analyses were carried out. For cyclic load models strips in both directions were assigned. The results of the monotonic SPSW model were found to be inconsistent with the test results. The initial stiffness and ultimate strength predicted by the model was higher (about 10% higher ultimate strength) compared to the test results. Overall behavior of the cyclic model matched with the test results. It was observed that neither could accurately describe the specimen pushover envelope behavior completely. [2]

Lubell also conducted a series of parametric study using SPSW monotonic model to investigate sensitivity of certain parameters. Lubell noted that the initial stiffness was not overly sensitive to changes in infill plate thickness, t, but the ultimate strength was found to increase as t increases. It was also found that the initial stiffness and ultimate strength of the specimen was decreased as tension field angle, α, decreases. [2]

2.8 Rezai (1999)

Rezai performed shake table testing of a four storey SPSW specimen nearly identical to the one tested by Lubell. The main objective was to study the dynamic behavior of the SPSW. This was first test of this kind conducted on SPSW specimen. The specimen was subjected to various site recorded and synthetically generated ground motions at varying intensities. Due to the limitations of the shake table, the test results remained in the elastic range. Thus the nonlinear behavior of the SPSW was not explored in detail. [2]

Rezai found that the first mode was the primary mode of vibration with very little contribution from higher modes. The top storey exhibited flexural dominated behavior, whilst bottom storey acted as a shear panel during the test sequence. Based on the load deformation plot, it was observed that, first storey dissipated the majority of energy whilst the top floors acted as a rigid body rotating about the first floor. It was also found that the intermediate level beams experienced negligible strains. Rezzai also conducted sensitivity analyses to assess the effects of various structural parameters on the value of tension field angle,α . It was observed that α did not vary significantly for any change in the beam and column cross sectional area and infill plate thickness, of 6 mm and greater.[2]

2.9 Behbahanifard et, all (2003)

Behbahanifard et, all conducted an experimental and numerical investigation of steel plate shear walls. The test specimen was taken directly from the ine tested by Driver et all with bottom panel removed due to the damage from the original test, thus creating a large-scale, three storey single bay specimen. The loading sequence was in accordance with ATC-24 guidelines. The first level beam was ruptured at the top flange and web of the beam to column connection after 50 cycles of loading prior to the achievement of the ultimate load. Since one of the objectives of the test was to observe the ultimate capacity of the SPSW and the behavior of the boundary elements, the fracture was repaired and testing continued.[2]

Ultimate capacity of the specimen was reached at a displacement of seven times that at yield point displacement, at that point the strength was started to deteriorate. The specimen displayed high elastic stiffness, excellent ductility, the ability to dissipate high amounts of energy, stable hysteresis loops, and a high degree of redundancy. A finite element model was also developed for the analysis of SPSW. A parametric study was conducted using this model to identify the parameters influencing the behavior of a single shear wall panel. It was observed that, the infill plates having an aspect ratio of 1.0 to 2.0 had negligible effect on the behavior of the shear wall panel. However, for the aspect ratios less than 1.0, both stiffness and shear capacity of the shear wall panels increase. An increase in the stiffness of the shear wall panel was observed by increasing the ratio of the axial stiffness of the infill plate ($tL/2A_c$) with negligible effect on the shear capacity of the system.[2]

2.10 Berman and Breneau (2003)

Using plastic analysis theory and the assumption of discrete strips to represent the infill plate, Berman and Breneau derived equations to calculate the ultimate strength of single and multi-storey SPSW’s with either simple or rigid beam to column connections. For estimating the ultimate strength of multi-storey shear walls, equations were developed based on two types of failure mechanisms-soft storey failure and uniform yielding of the infill plates in all storeys simultaneously. The equation was derived for single storey SPSW’s with simple beam to column connections to provide a lower bound value. This was used to predict the capacity of a variety of single and multi-storey SPSW specimens having either pinned or semi-rigid connections. This equation was found to underestimate the experimental capacity by about 6%, although it overestimated the capacity of one case by about 9%. Soft storey mechanism equation was found to overestimate the capacity of multi-storey specimens with rigid connection by about 17%. The model provides only ultimate capacity; the equations do not address the initial stiffness, the ductility, actual failure mechanism and frame forces for use in design. [2]

2.11 AISC code provisions

AISC code requires the slender unstiffened steel plates (webs) connected to surrounding horizontal and vertical boundary elements (HBE and VBE) designed to yield and behave in a ductile hysteretic manner during earthquakes. All HBE are also rigidly connected to the VBE with moment resisting connections able to develop the expected plastic moment of the HBE. Each web must be surrounded by the boundary elements. [3]

AISC code also states that, SPSW's designed according to the provisions of the code are expected to give significant inelastic deformation capacity mainly through web plate yielding and as plastic-hinge formation at the ends of horizontal boundary elements (HBE's).

The following gives a summary of the provisions to calculate the approximate sizes of the various components of SPSW.

2.11.1 Webs

The panel design shear strength, $\emptyset V_n$ (LRFD), and the allowable shear design V_n/Ω (ASD), according to the limit state of shear yielding, shall be determined as

$$V_n = 0.42 F_y\, t_w\, L_{cf} \operatorname{Sin} 2\alpha \text{---eq 2.5}$$

Where,

L_{cf} = clear distance between column flanges, in (mm)

t_w = thickness of the web, in (mm)

α angle of web yielding in degrees, as measured relative to the vertical, α, is permitted to be taken as 40°, or to be calculated as follows:

$$\tan^4\alpha = \frac{1+\frac{t_w L}{A_C}}{1+t_w h\times\left(\frac{1}{A_b}+\frac{h^3}{360 I_C L}\right)} \text{---eq 2.6}$$

Where,

A_b = cross sectional area on an HBE, in (mm^2)

A_c = cross sectional area on an VBE, in (mm^2)

2.11.2 Boundary elements

The vertical boundary elements (VBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c, not less than $(0.0031 t_w h^4)/L$. The horizontal boundary elements (HBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, not less than $(0.0031L^4)/h$ times the difference in web plate thickness above and below. [3]

Where,

L = distance between VBE centerlines, (in mm)

h = distance between HBE centerlines, (in mm)

t_w = thickness of the web, (in mm)

2.12 Metal foams

Structural element consisting of two steel plates at the faces separated by a lightweight core of foamed metal is known as metallic foam sandwich panels. The moment of inertia increases due to the separation of core, hence generating an effective structural member which is light in weight and suitable for resisting in-plane loads, bending and buckling loads. Metal foams also have exceptionally good energy dissipation and deformation capacities.

Metal foams are manufactured by various processing techniques such as powder metallurgy, hollow sphere, lotus type etc. Metallic foam properties depend on the properties of the metal, relative density and cell topology (e.g.- open or closed cell, cell size etc). A conventional compression stress-strain curve for aluminum foam is shown in figure 2.3. The slope of the initial loading portion of the curve is lower than that of the unloading curve. The reduction in the slope of the unloading curve indicates that there is localized plasticity in the specimen at stresses well below the compressive strength of the foam. Therefore, measurements of the Young's modulus should be made from the slope of the unloading curve as shown in the figure 2.3, unloading from about 75% of the compressive strength. The compressive strength of the specimen is taken to be the initial peak stress if there is one; otherwise, it is taken to be the stress at intersection of two slopes: that for the initial loading and that for the stress plateau. [4]

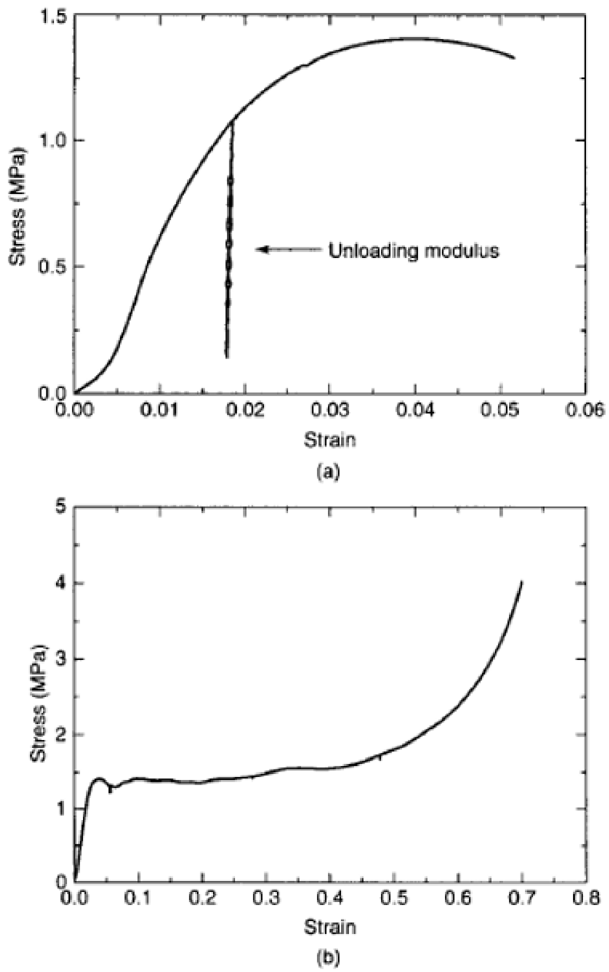


Figure 2.3 –Stress-strain curve from a uniaxial compression test on a cubic specimen of closed cell aluminum foam: a) to 5% strain b) to 70% strain [4]

3. MODELLING OF STEEL PLATE SHEAR WALLS

3.1 Introduction

All the models were prepared in SAP2000 version 19.Steel plate shear wall models of varying aspect ratio and storey heights were created. Single storey, two storey and three storey models were created with different aspect ratios ranging from 1.0 to 3.0.A total of eighteen models were created of which nine models consisted of typical steel infill plates and the rest nine were of metal foam sandwich panel. The aspect ratios of 1.0, 2.0 and 3.0 were considered to cover possible more cases. Table 3.2 gives the summary of the models. The sections of VBE's and HBE's that were used in the models are given in figure 3.1 to 3.9. The equivalent thickness of the metal foam composite panels was calculated using the expressions given in section 3.2.1. Table 3.1 provides a summary of the equivalent thickness of sandwich panels.

Table 3.1 Thickness of Steel plate and equivalent metal foam composite panel

Steel plate thickness (in mm)	Composite panel equivalent thickness (in mm)	
	Face plates	Foam core
0.9	0.315	5.67
1.5	0.525	9.45
1.8	0.63	11.34

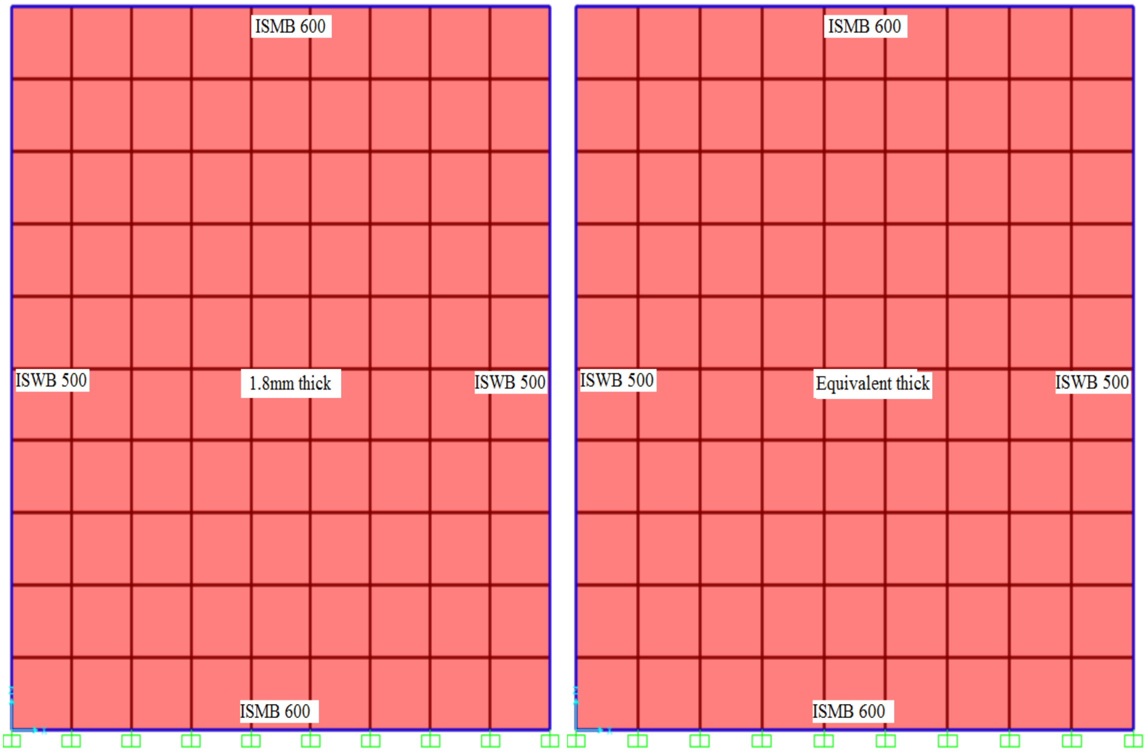
The dimensions of single storey frames were 3 m wide x 3 m high, 6 m wide x 3 m high and 9 m wide x 3 m high respectively. The width of bay and storey height for two storey and three storey models was kept constant (i.e. 3.0 m).

Table 3.2 SPSW models with different aspect ratio and storey heights

Model description			
Single Storey models			
S.No	**Aspect ratio L/h**	**Steel plate models**	**Sandwich panel models**
1	1.0	Web plate 1.8 mm thick	Equivalent web plate for 1.8mm
2	2.0	Web plate 1.8 mm thick	Equivalent web plate for 1.8mm
3	3.0	Web plate 1.8 mm thick	Equivalent web plate for 1.8mm
Two Storey models			
4	1.0	Web plate 1.8 mm thick-Ground floor Web plate 1.5 mm thick-1st Floor	Equivalent web plate (1.8 mm)–Ground floor Equivalent web plate (1.5 mm) 1st Floor
5	2.0	Web plate 1.8 mm thick-Ground floor Web plate 1.5 mm thick-1st Floor	Equivalent web plate (1.8 mm)–Ground floor Equivalent web plate (1.5 mm) 1st Floor
6	3.0	Web plate 1.8 mm thick-Ground floor Web plate 1.5 mm thick-1st Floor	Equivalent web plate (1.8 mm)–Ground floor Equivalent web plate (1.5 mm) 1st Floor
Three Storey Models			
7	1.0	Web plate 1.8 mm thick-Ground floor Web plate 1.5 mm thick-1st Floor Web plate 0.9 mm thick-2nd Floor	Equivalent web plate (1.8 mm)–GF=round floor Equivalent web plate (1.5 mm)1st Floor Equivalent web plate (0.9 mm)2nd Floor
8	2.0	Web plate 1.8 mm thick-Ground floor Web plate 1.5 mm thick-1st Floor Web plate 0.9 mm thick-2nd Floor	Equivalent web plate (1.8 mm)–Ground floor Equivalent web plate (1.5 mm)1st Floor Equivalent web plate (0.9 mm)2nd Floor
9	3.0	Web plate 1.8 mm thick-Ground floor Web plate 1.5 mm thick-1st Floor Web plate 0.9 mm thick-2nd Floor	Equivalent web plate (1.8 mm)–Ground floor Equivalent web plate (1.5 mm)1st Floor Equivalent web plate (0.9 mm)2nd Floor

Single Storey models

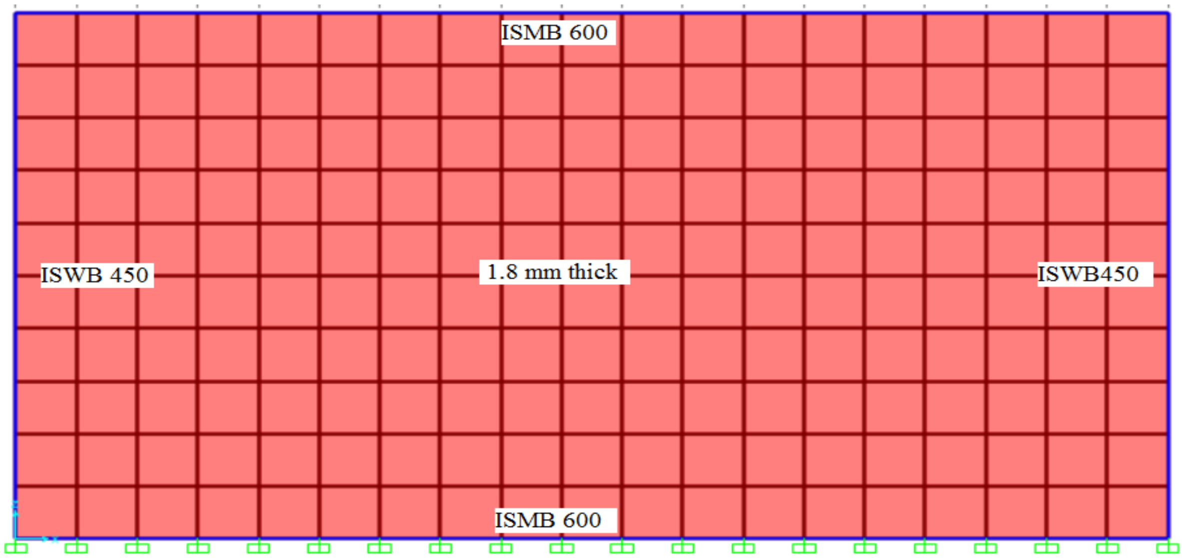
1. Aspect ratio 1.0



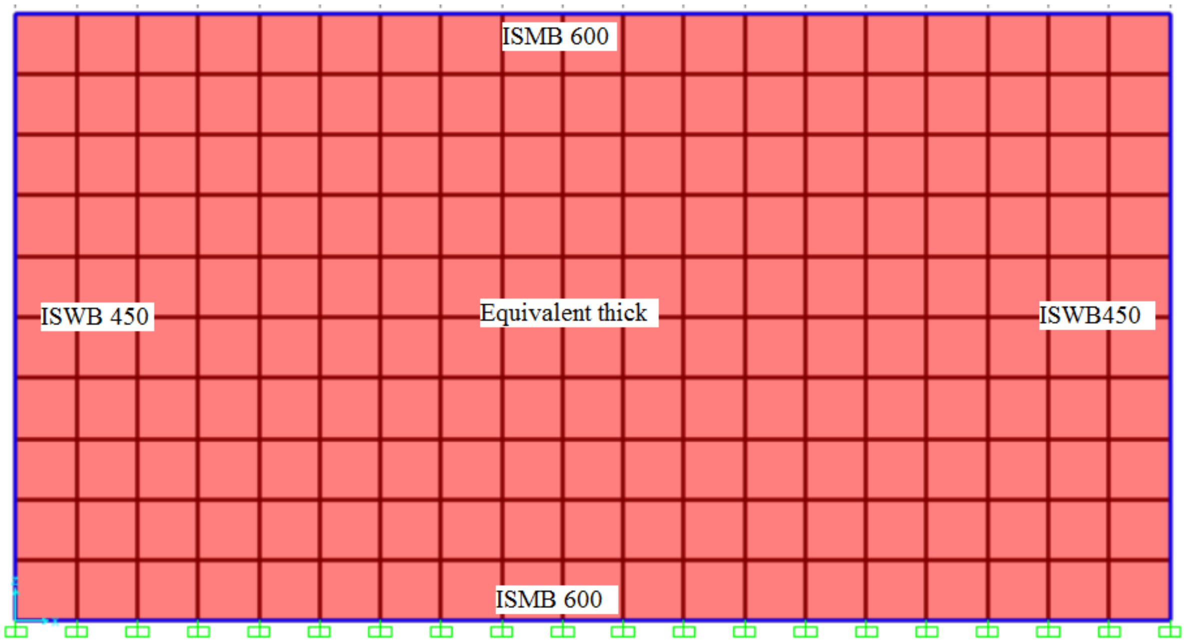
(a) (b)

Figure 3.1 Single storey with aspect ratio 1.0 - a) steel plate b) Sandwich panel

2. Aspect ratio 2.0



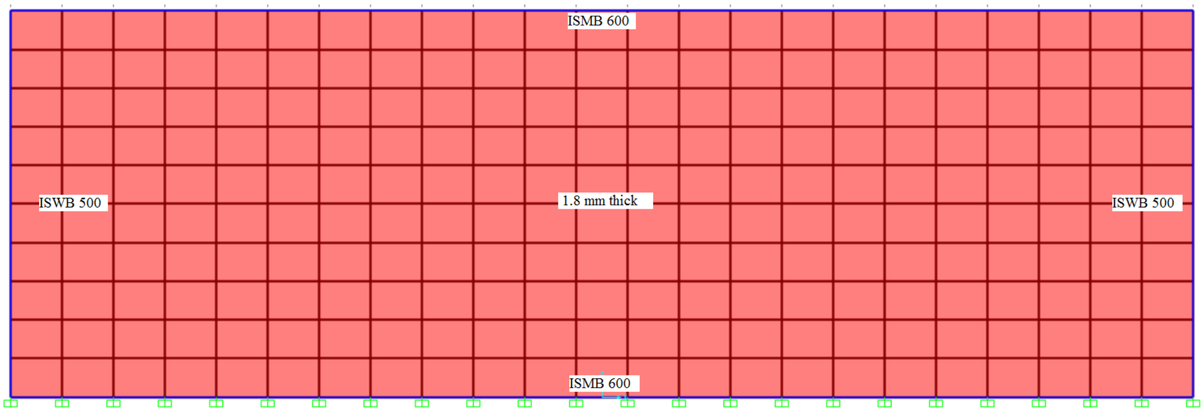
(a)



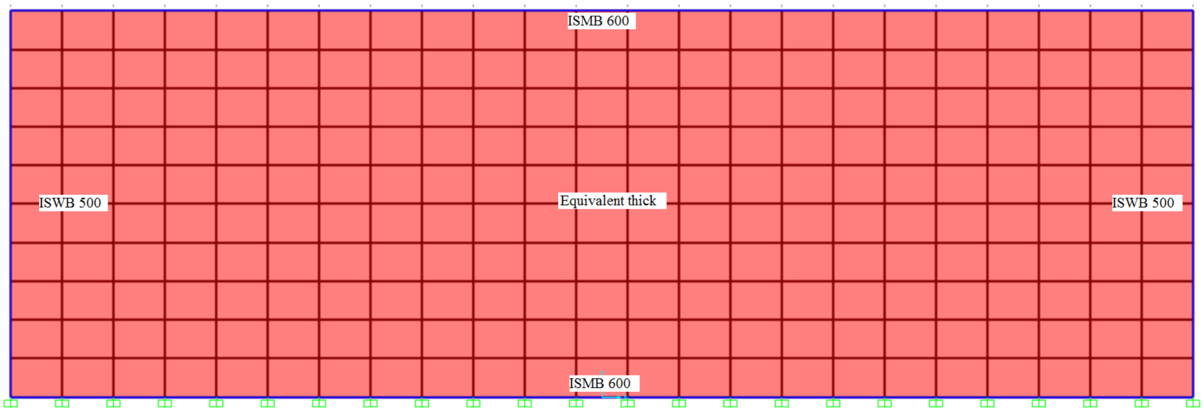
(b)

Figure 3.2 Single storey with aspect ratio 2.0- a) steel plate b) Sandwich panel

3. Aspect ratio 3.0



(a)

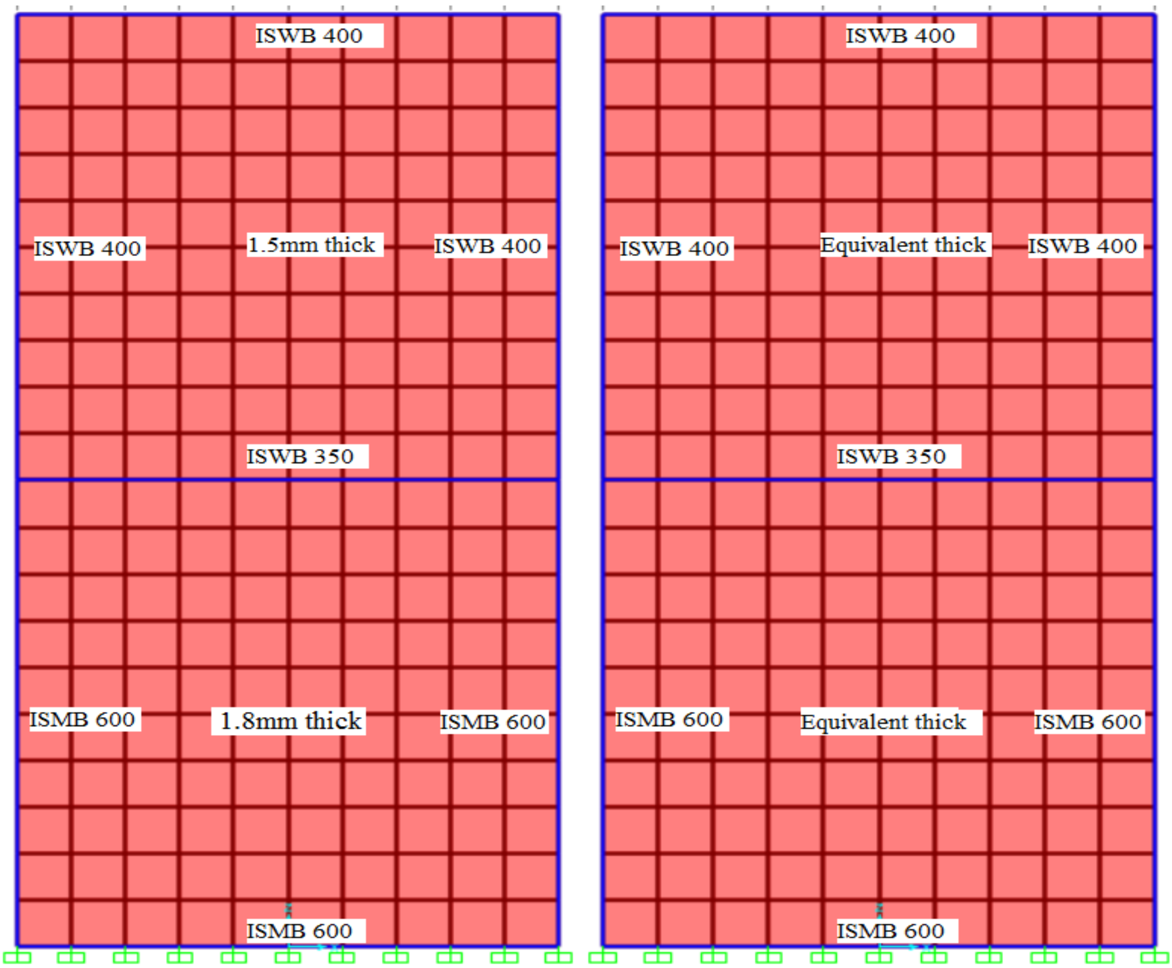


(b)

Figure 3.3 Single storey with aspect ratio 3.0- a) steel plate b) Sandwich panel

Two storey models

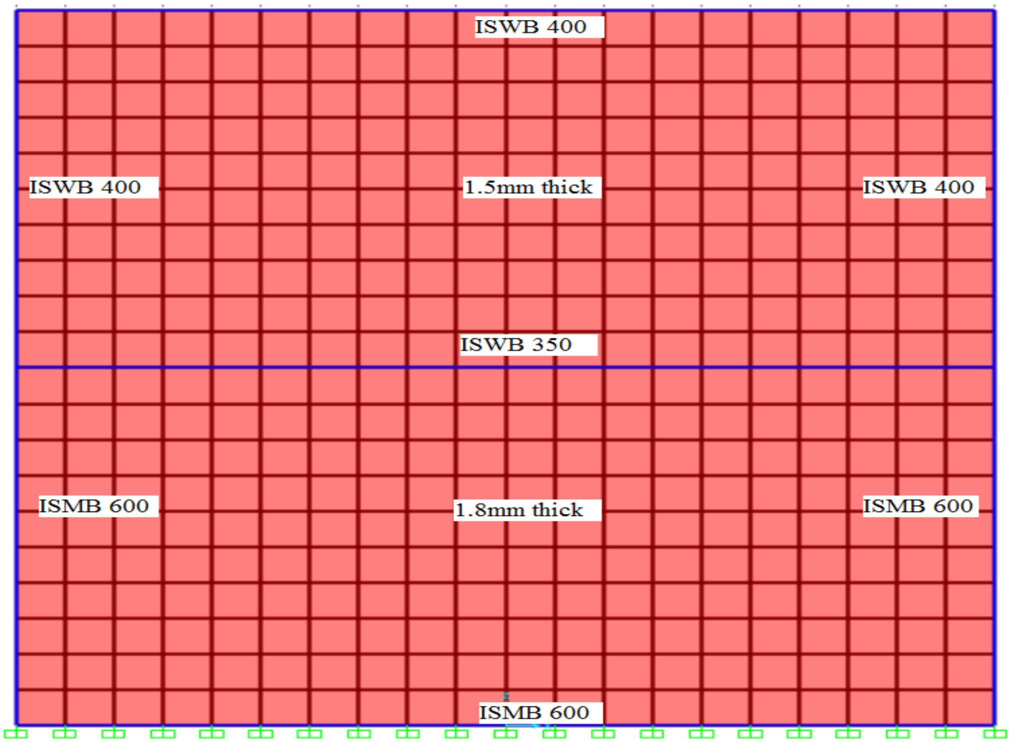
1. Aspect ratio 1.0



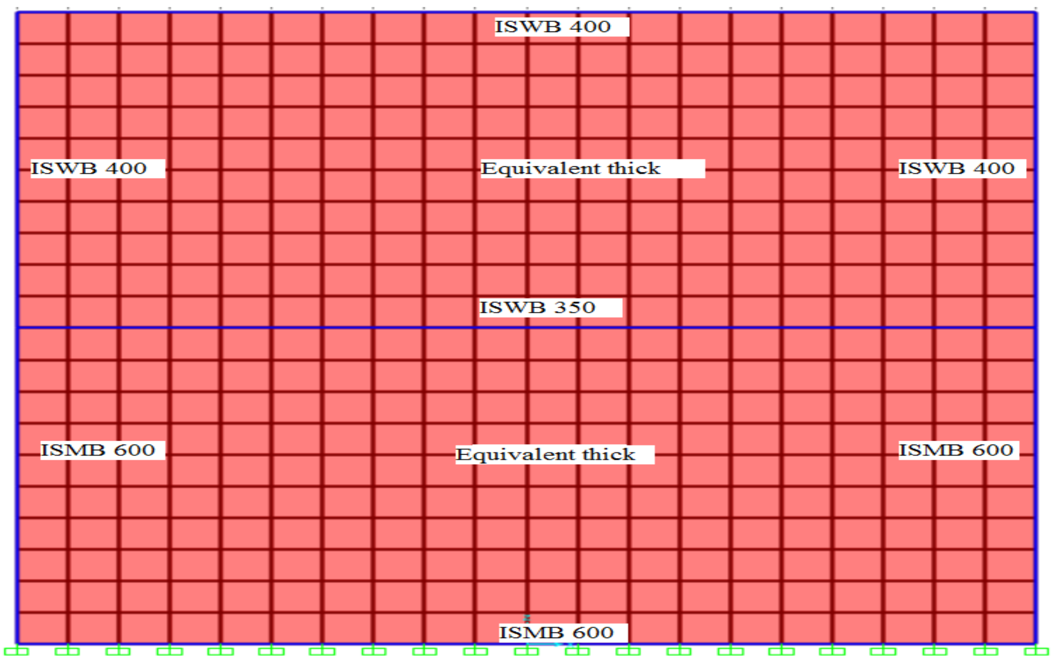
(a) (b)

Figure 3.4 Two storey with aspect ratio 1.0- a) steel plate b) Sandwich panel

2. Aspect ratio 2.0



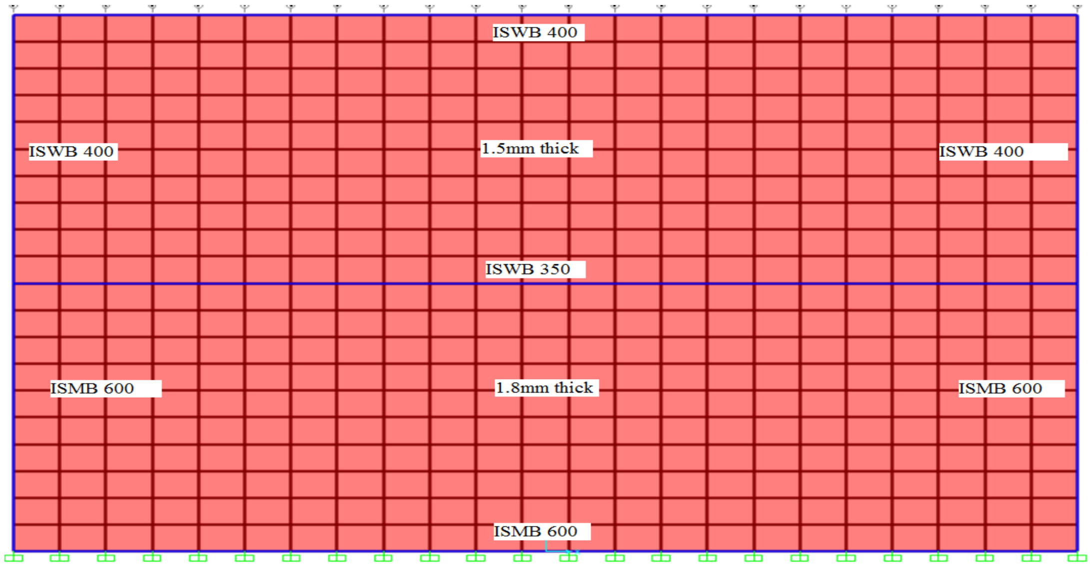
(a)



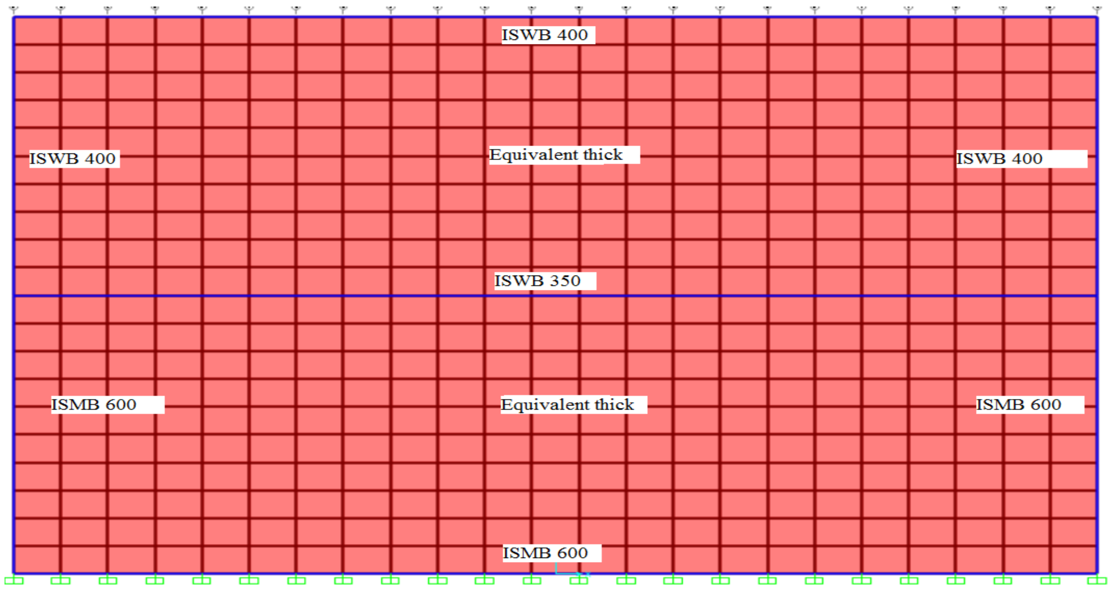
(b)

Figure 3.5 Two storey with aspect ratio 2.0- a) steel plate b) Sandwich panel

3. Aspect ratio 3.0



(a)



(b)

Figure 3.6 Two storey with aspect ratio 3.0- a) steel plate b) Sandwich panel

Three Storey models

1. Aspect ratio 1.0

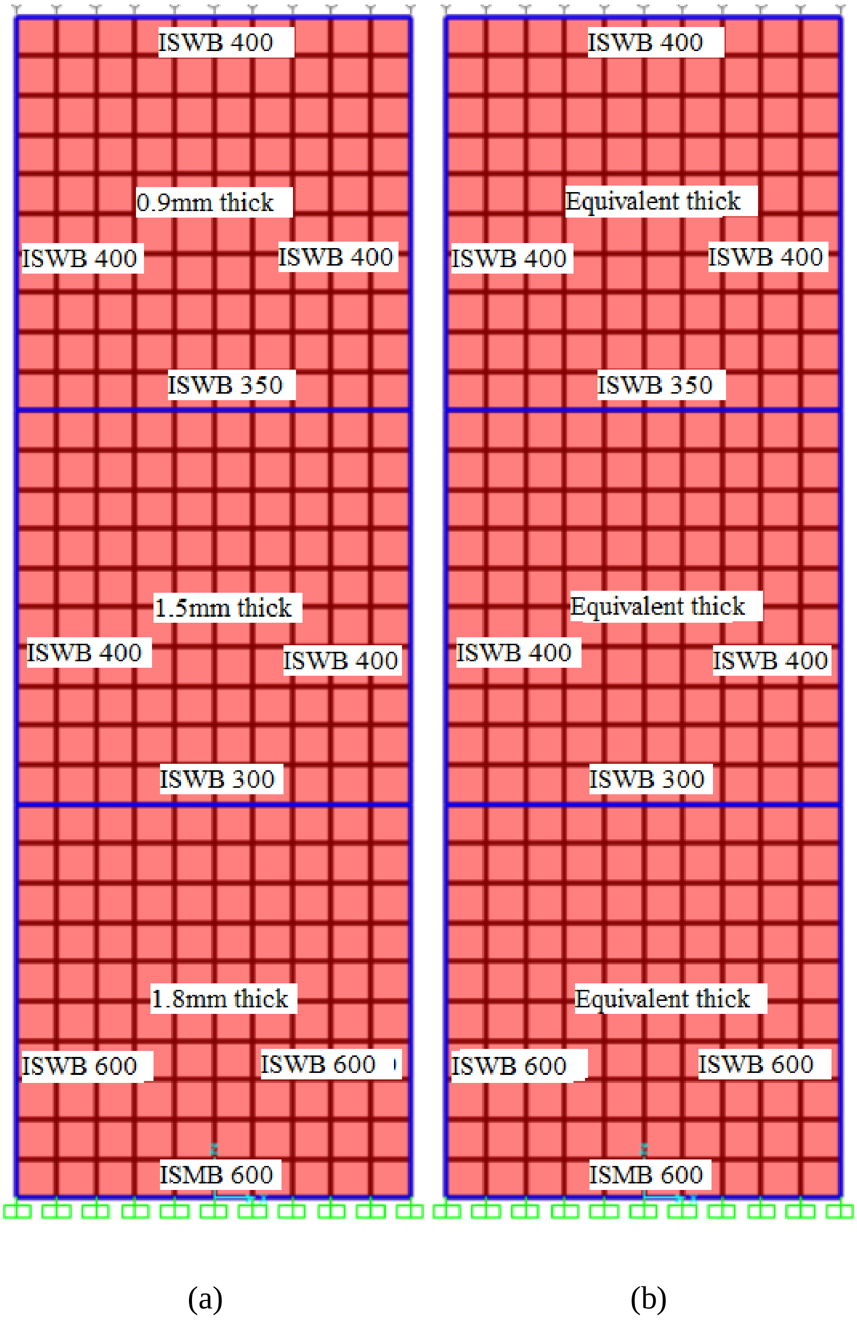
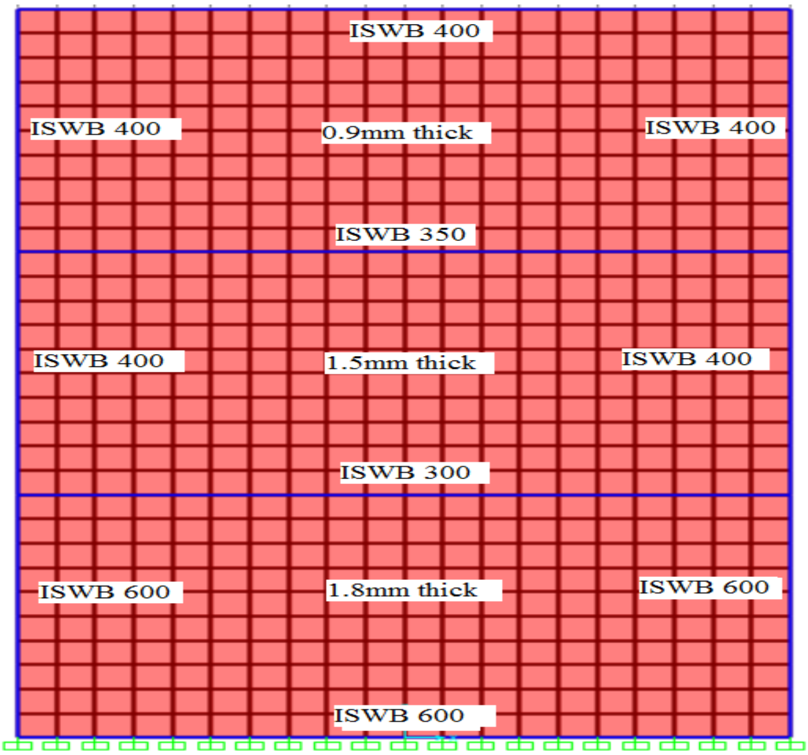
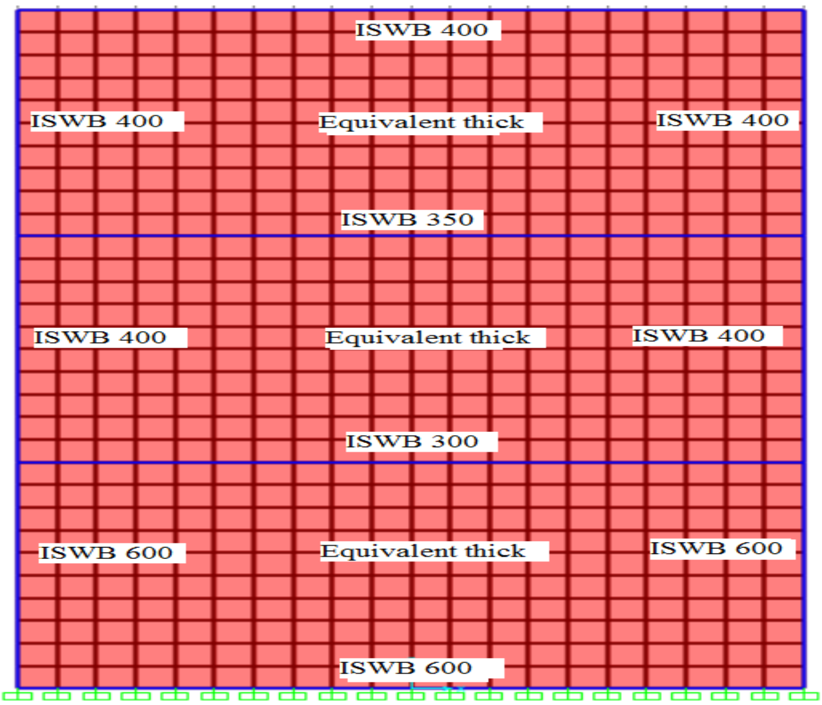


Figure 3.7 Three storey with aspect ratio- 1.0- a) steel plate b) Sandwich panel

2. Aspect ratio 2.0



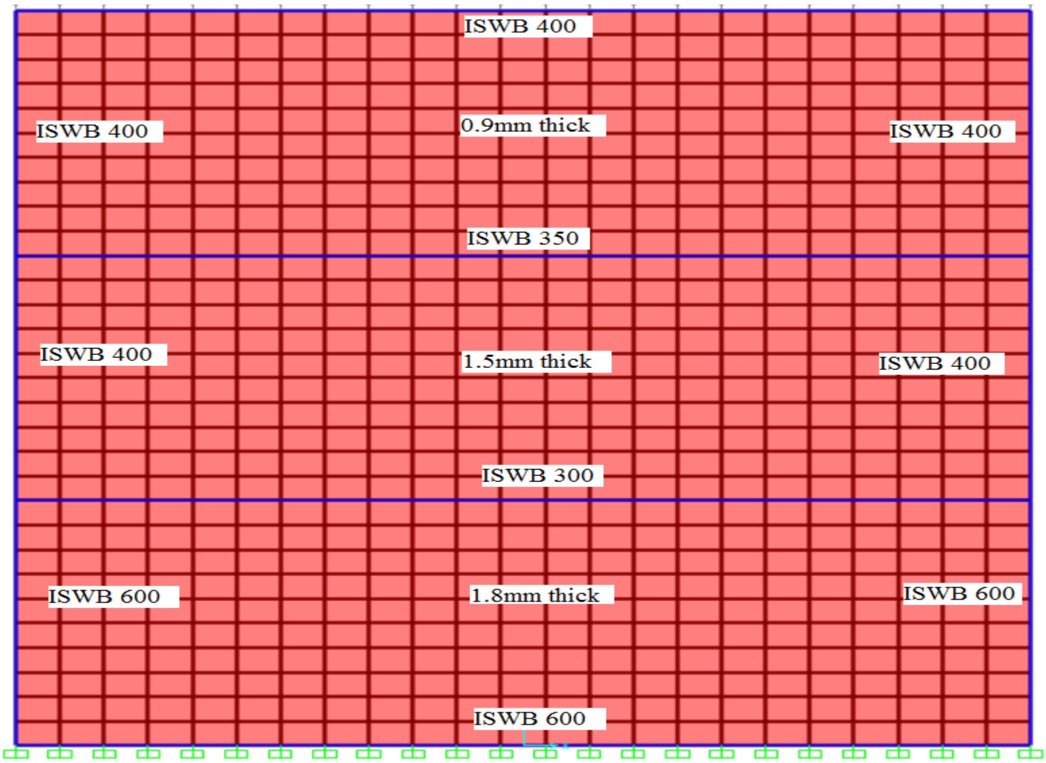
(a)



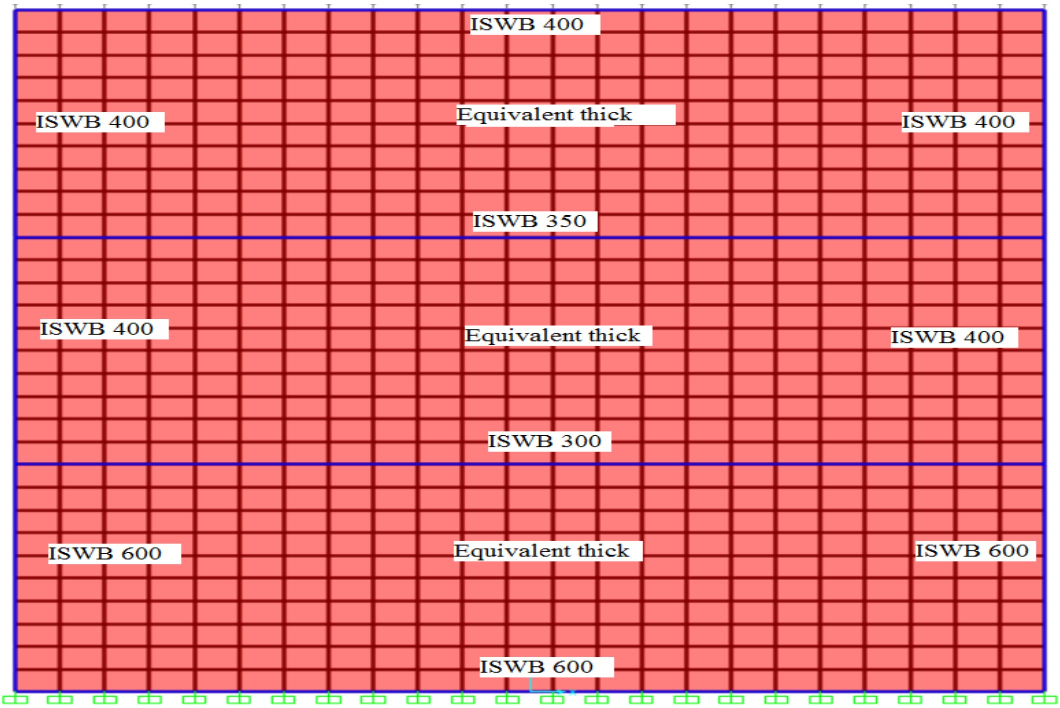
(b)

Figure 3.8 Three storey with aspect ratio 2.0- a) steel plate b) Sandwich panel

3. Aspect ratio 3.0



(a)



(b)

Figure 3.9 Three storey with aspect ratio 3.0- a) steel plate b) Sandwich panel

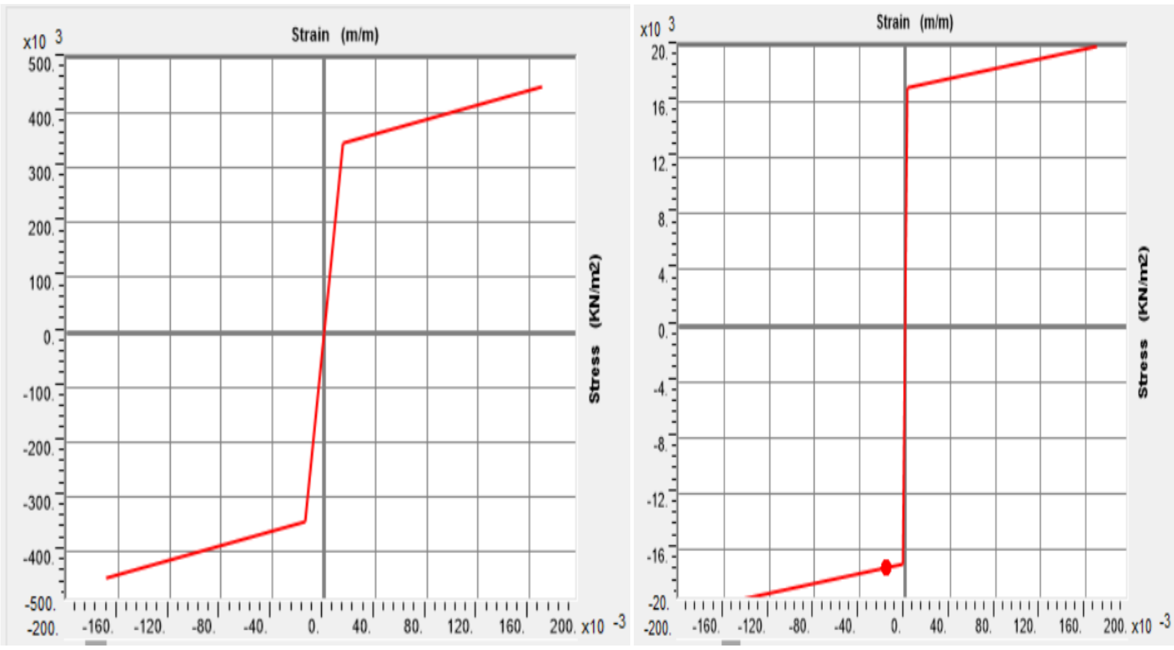
3.2 Modeling technique

Initially the SPSW’s carry the load by shear buckling of infill plates and then the load is resisted by the developing the tension field action along the tension diagonal. Hinges will be formed at specific locations in the boundary elements. The response of the infill plates to load is non linear in the post buckling stage.

3.2.1 Steel Plate Shear wall models

Analysis of all SPSW models was done in SAP2000 version 19. The Steel plate shear walls were modeled as two dimensional systems. Four noded shell elements were used to model the infill plates. Beam elements were used to model the horizontal and vertical boundary elements. Orthotropic material properties were assigned to the infill plates in all SPSW models. The default constitutive relationship in SAP2000 used a multi-linear stress-strain curve.

Simple bilinear elastic plastic stress strain curve was used for steel instead of multi-linear stress-strain curve for steel which is in-built in the software. Similarly, a bilinear stress strain curve was used for metal foam. The details of the stress strain curves are as shown in figure 3.10.



(a) Bilinear stress strain curve for steel (b) Sandwich panel-foam core-bi-linear curve

Figure 3.10 – Bilinear stress strain curves

The local axis of the shell elements was rotated by 42.80º to the vertical with complete elastic modulus applied in the local axis 1 direction i.e. in the direction of tension field and 4% of the value of elastic modulus in the local axis 2 direction i.e. in the direction of compression diagonal to obtain the tension field action. Zero shear modulli value was assigned to the entire shell elements in all the directions. Equation 2.6 was used to measure the tension field angle and an average value was used in all models.

The member properties of the boundary elements of each case were estimated based the provisions of AISC code [3] as mentioned in section 2.11.2. The beams to column connections were modeled as moment resisting connections. Vertical and horizontal loads are assigned at each floor level.

Vertical loads at storey locations are assigned to include the effect of gravity loads. Horizontal loads were assigned at each floor level, in accordance with fundamental mode. Non linear pushover analysis was used to analyse the Steel plate shear walls in SAP2000. The node at the top most level was monitored for target displacement. According to AISC code " a model of the SPSW can be constructed in which bilinear elastoplastic web elements of strength $R_yF_yA_s$ are introduced in the direction of, α. Bilinear plastic hinges also be introduced at the ends of the horizontal boundary elements. Standard push over analysis conducted with this model provides axial forces, shears and moments in the boundary frame when the web develops yielding. Separate checks are required to verify that plastic hinges do not develop in the horizontal boundary elements, except at their ends". [3]

The failure mechanism is affected by the formation of hinges in the horizontal boundary elements and hence the ultimate load carrying capacity. AISC code analysis procedure described above needs the formation of hinges at the ends of the HBE's, but, as stated by Purba and Bruneau, the code does not mentions an analysis procedure to ensure that this intent is met.[5] In order to determine the possible hinge positions, a non-linear analysis of the model without any hinges on the boundary elements was performed. The bending moment diagram gives the locations of maximum moment which conforms to possible hinge locations. The hinge locations obtained on this basis was assigned on the boundary elements and final bending moment diagram were inspected to make sure that the values do not exceed plastic moment capacities along the member length. In order to quicken the modeling process in addition to hinges at the ends, further the hinges were assigned on the Horizontal boundary elements at the quarter, middle and third-quarter points.

3.2.2 Composite Panel Shear wall models

The metallic foam sandwich panel considered in the current work comprises of two thin steel plates attached with light weight core. Metal foam has greater in-plane strength and bending rigidity as compared to that of a solid steel plate. Considering a plate of initial thickness t_{ini}, if the entire plate is foamed, the final thickness t_f is given as-

$$t_f = {}^{t_{ini}}/_{\rho} \text{-----eq 3.1}$$

Where, ρ is the relative density of the foamed steel. If thickness, $t_{ini} = 1$, this corresponds to a steel plate.[6]

Sandwich panels comprise of only the central fraction of foam which is represented by, α (value of α ranges from 0 to 1.0). Assuming that the relative density of the central core is ρ then the core thickness, t_c, increased from the initial solid plate thickness, t_{ini}, is-

$$t_c = \alpha \times \left({}^{t_{ini}}/_{\rho}\right) \text{----eq 3.2}$$

The remaining portion of the initial solid sheet is divided evenly between two face sheets of thickness, t_s-[6]

$$t_s = \left({}^{(1-\alpha)}/_{2}\right) \times t_{ini} \text{---eq 3.3}$$

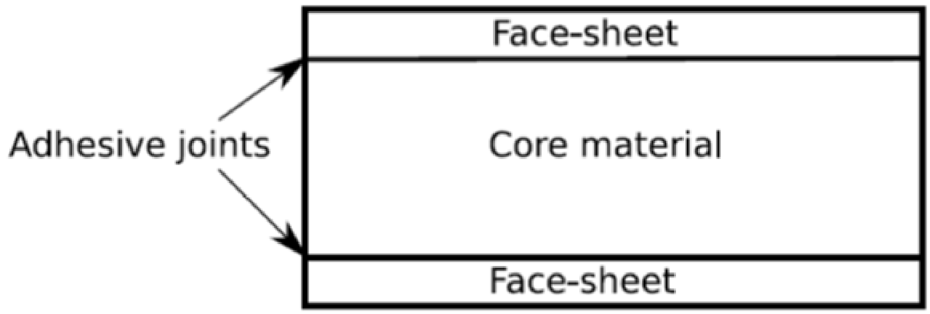


Figure 3.11 – Schematic representation of sandwich panel [12]

The material properties of the metal foam sandwich panels used for current work are given below-

Young's modulus of core, E_f = 7.4 MPa

Yield strength of core, σ_y = 17 MPa

Central fraction of foam which is represented by,α= 0.3

Relative density, ρ = 0.05 [4]

Based on the above material properties and by the use of equations 3.1 to 3.3, the values were used in the present work are as follows-

Equivalent thickness of facing sheets, t_s = $0.35\times t_{ini}$

Thickness of core, t_c = $6.3\times t_{ini}$

Thus if, ts is 1.8 mm, equivalent thickness of facing sheets, t_s= 0.35×1.8 = 0.63 mm and thickness of core, t_c= 6.3×1.8 = 11.34 mm.

Four noded shell elements were used to model sandwich panel. In order to have sandwich panel having steel plate facing and core of foamed metal multi-layered shell option of SAP2000 was used. Beam elements were used to model boundary elements. The local axes of sandwich panel shell elements were also rotated by 42.80º as in the steel plate shear wall model to capture tension field action. Other modeling features were kept identical to that of steel plate shear wall model as mentioned in previous section. Since there are no guidelines for design of metal foam panel shear walls so the boundary elements were kept similar as that of steel plate shear walls and loading conditions were also the same to have a common base for the comparison of both the shear walls.

4. ANALYSIS RESULTS OF STEEL PLATE SHEAR WALLS

4.1 Introduction

Analysis of both Steel plate shear wall and Metal foam sandwich panel shear wall were carried out in SAP2000. The summary of the results of the analysis are described in the next section.

4.2 Analysis Results

4.2.1 Single storey models

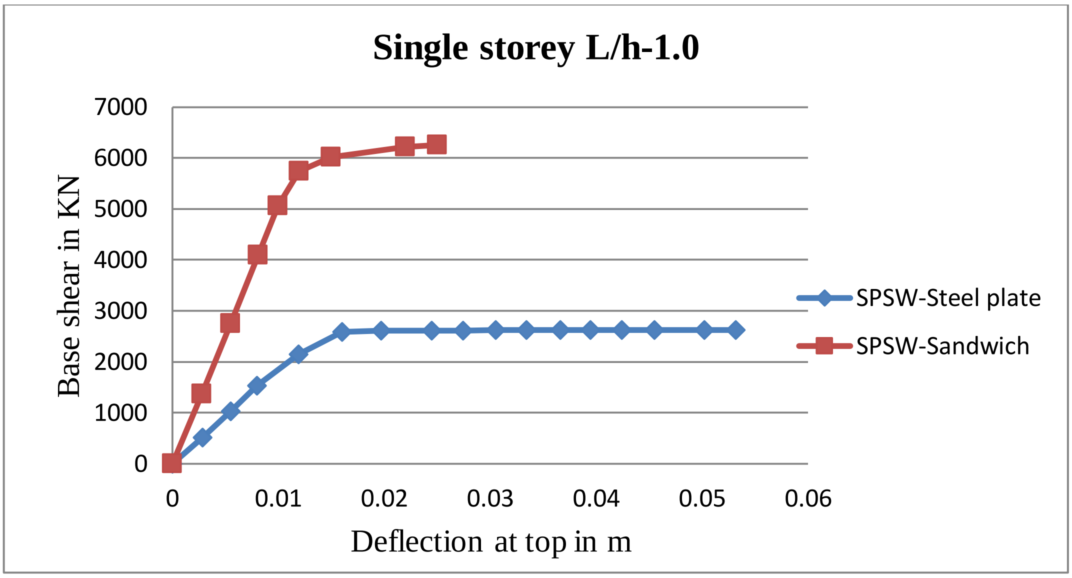
Monotonic pushover analysis was performed for all the SPSW models. Target displacement was taken as 10% of the storey height i.e. 300mm and it was monitored at the top most level of the storey. The push over analysis increases the load monotonically and it yields load versus displacement graphs until the specified target displacement is achieved. The results of single storey shear walls analysis are shown in figure 4.1

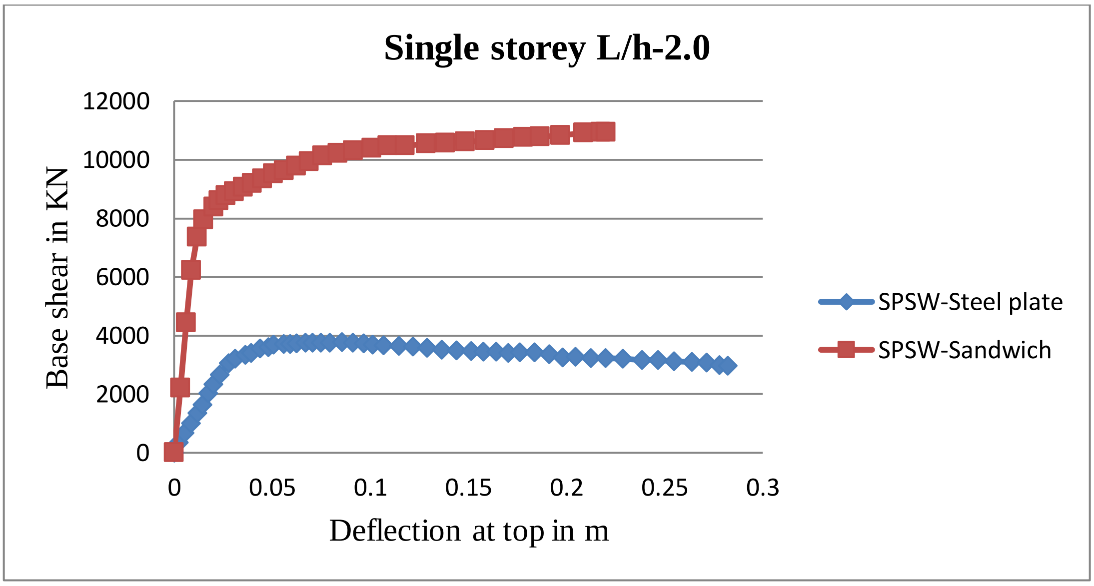
4.2.2 Two storey models

Analysis results of two storey shear walls are shown in figure 4.2.

4.2.3 Three storey models

Analysis results of three storey shear walls are shown in figure 4.3.





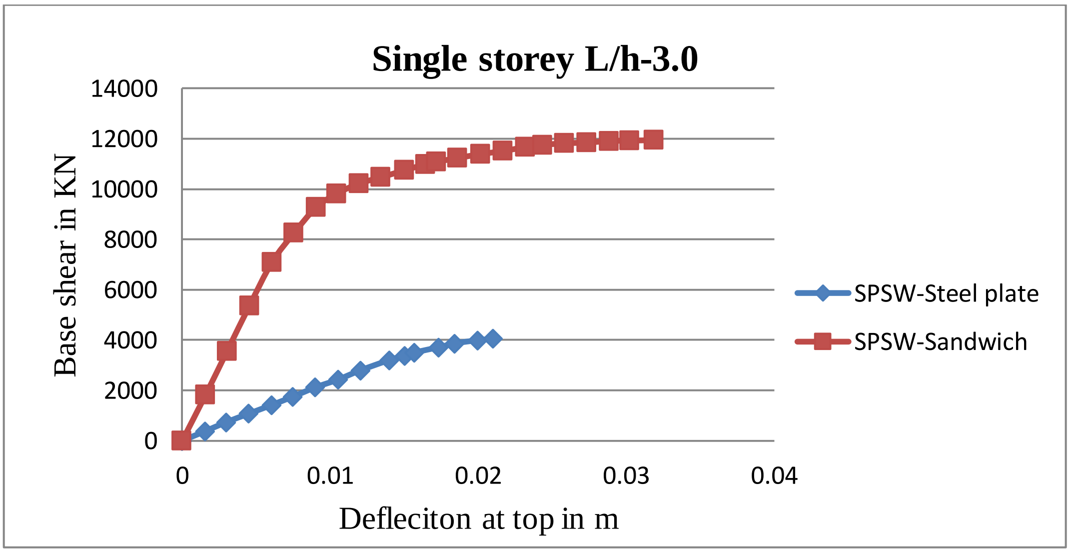


Figure 4.1 – Load-displacement plot of Single storey Steel & sandwich models

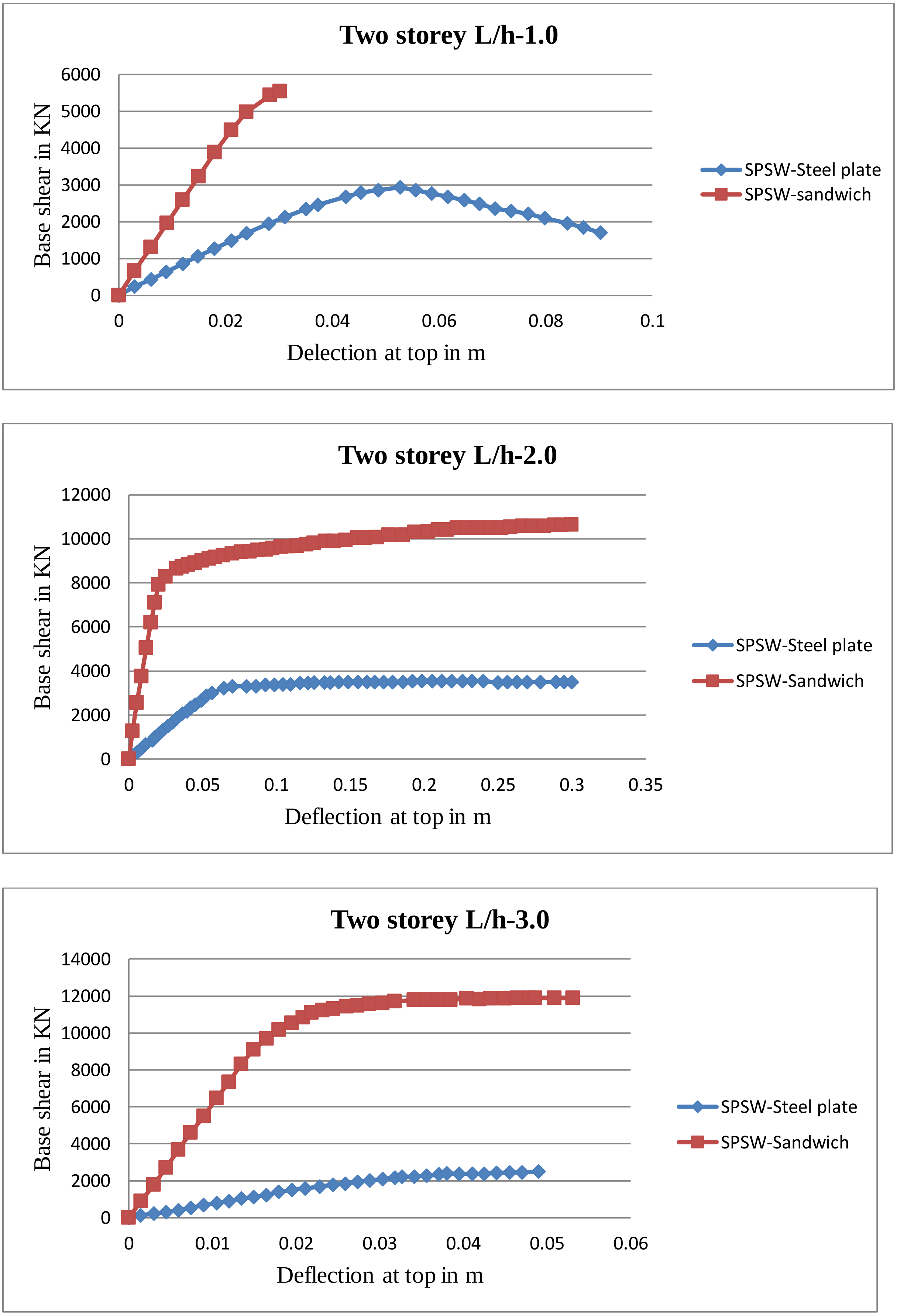
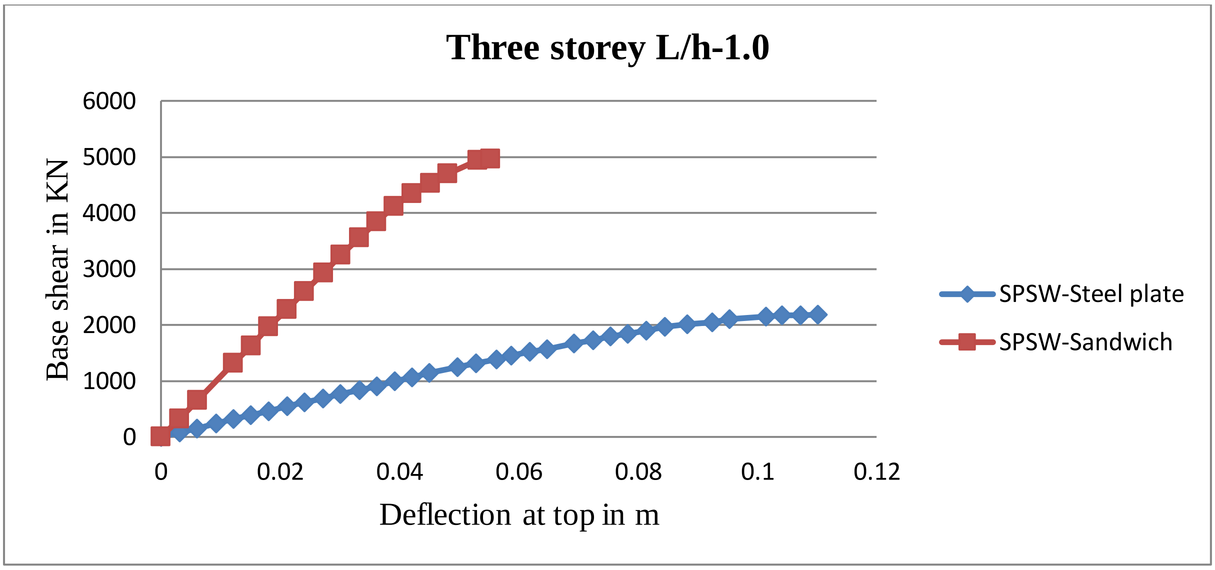
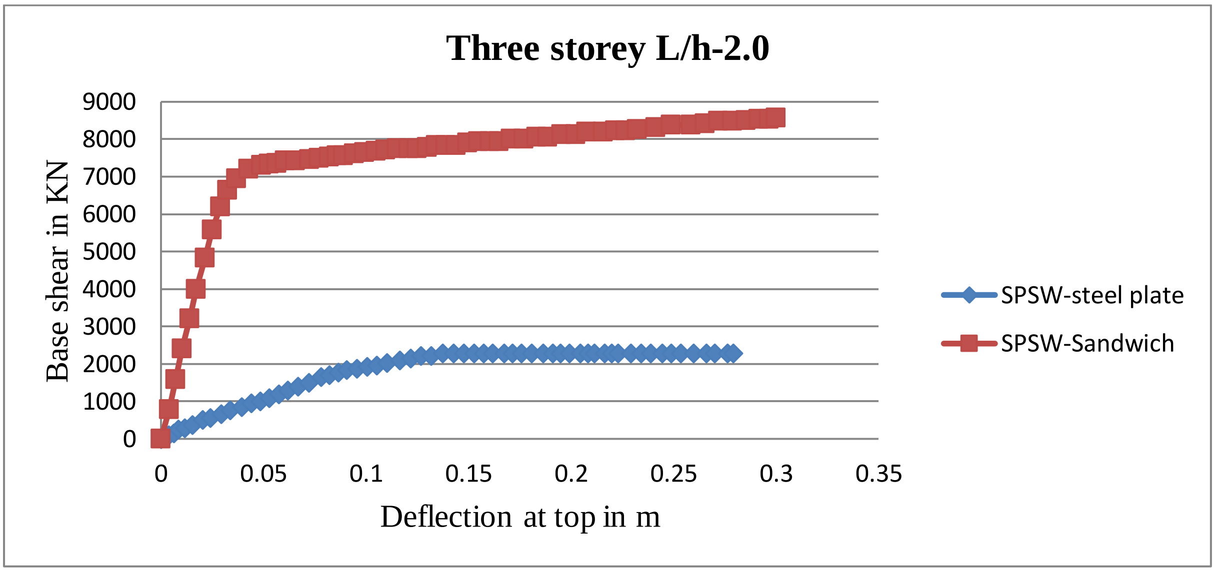


Figure 4.2 – Load-displacement plot of Two storey Steel & sandwich models





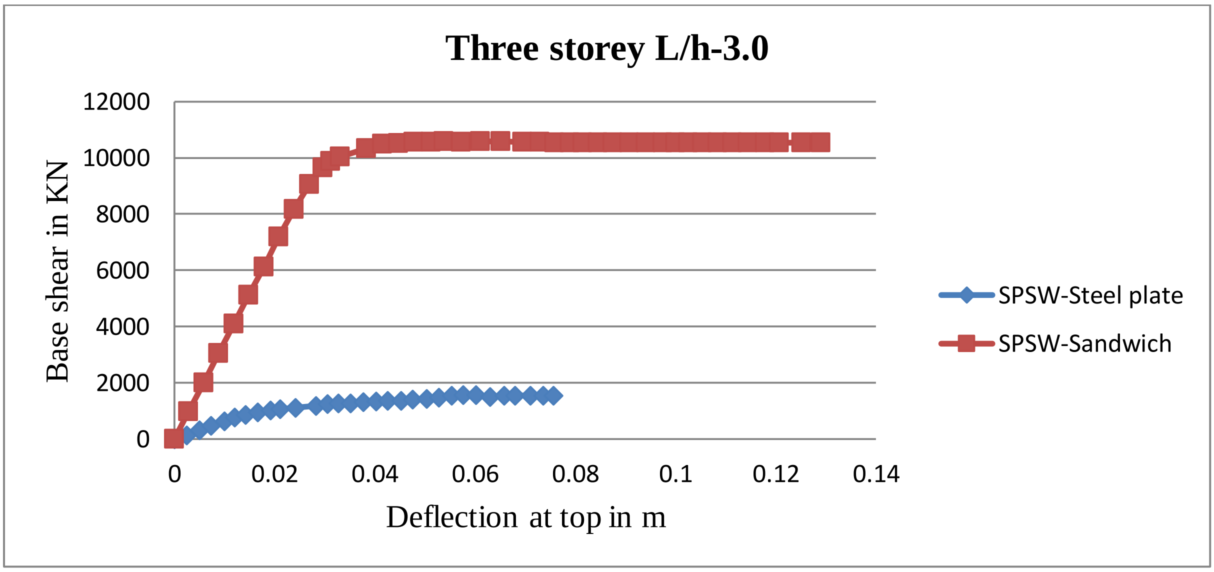


Figure 4.3 – Load-displacement plot of Three storey Steel & sandwich models

Clearly it visible from the graphs that in some cases the specified target displacement of 300mm was not achieved. This might be because of early failure of the system mainly due to the strength degradation of the boundary elements at the hinge locations. Certainly the sandwich panels have higher stiffness as compared to that of steel plate shear wall. Currently there are no design codes available for calculating the required properties of the boundary elements for metal foam sandwich panel infill plate shear walls. Therefore the present work is based on certain assumptions and it uses the same boundary elements for sandwich panels as those used in steel plate shear wall.

5. DISCUSSION OF ANALYSIS RESULTS

5.1 Introduction

SPSW models-both created by using conventional steel plates and metallic foam sandwich panels were analysed using SAP2000. The discussion of the results are summarized in the following sections,

5.2 Single storey models

The yielding of steel plate with aspect ratio 1.0 took place at a load of 2587 KN with a displacement of 16 mm which is about 0.53% of the storey height and the ultimate load before the failure was calculated as 2627 KN with a displacement of 60 mm which is about 2% of the storey height . For the same aspect ratio of 1.0 the sandwich panel model attained a yield load of 5744 KN with a displacement of 12 mm which is about 0.4 % of storey height and ultimate load attained was 6,257 with corresponding displacement of 25 mm which is about 0.83 % of storey height.

For aspect ratio 2.0 the steel plate shear wall the yielding took place at a load of 3407 KN with a displacement of 39 mm which is about 1.3% of the storey height and the ultimate load before the failure was calculated as 2970 KN with a displacement of 282 mm which is about 9.4% of the storey height. The sandwich panel model for the same aspect ratio of 2.0 attained a yield load of 7939 KN with a displacement of 14 mm which is about 0.46 % of storey height and ultimate load attained was 10930 KN with corresponding displacement of 220 mm which is about 7.30 % of storey height. It is clear from figure 4.1 that the sandwich panel models possess high initial stiffness and ultimate load capacity. Similarly for model with aspect ratio 3.0 the results are tabulated in table 5.1.

Table 5.1 Results of single storey model

Model with steel infill plate						
S.No	L/h	V_y,KN	Δ_y,mm	V_{max},KN	Δ_{max},mm	Initial Stiffness,KN/m
1	1.0	2587	16	2627	60	161687
2	2.0	3407	39	3787	282	87358
3	3.0	3208	14	4050	21	229142
Model with sandwich panel infill panel						
4	1.0	5744	12	6257	25	478667
5	2.0	7939	14	10930	220	567071
6	3.0	8250	7.5	11950	32	1100000

5.3 Two storey models

The yielding of steel plate with aspect ratio 1.0 took place at a load of 2458 KN with a displacement of 37 mm which is about 1.23% of the storey height and the ultimate load before the failure was calculated as 1700 KN with a displacement of 90 mm which is about 3% of the storey height . For the same aspect ratio of 1.0 the sandwich panel model attained a yield load of 4974 kN with a displacement of 23 mm which is about 0.76 % of storey height and ultimate load attained was 5540 with the corresponding displacement of 30 mm which is about 1% of storey height. Both models did not achieve the target displacement of 300 mm. This is mainly because of the strength degradation of boundary elements at hinge locations.

For aspect ratio 2.0 the steel plate shear wall the yielding took place at a load of 3012 KN with a displacement of 56 mm which is about 1.8% of the storey height and the ultimate load before the failure was calculated as 3507 KN with a displacement of 300 mm which is about 10% of the storey height. The sandwich panel model for the same aspect ratio of 2.0 attained a yield load of 7920 kN with a displacement of 20 mm which is about 0.67 % of storey height and ultimate load attained was 10640 KN with corresponding displacement of 300 mm which is about 10 % of storey height. It is clear from figure 4.2 that the sandwich panel models possess high initial stiffness and ultimate load capacity. Similarly for model with aspect ratio 3.0 the results are tabulated in table 5.2.

Table 5.2 Results of Two storey model

Model with steel infill plate						
S.No	L/h	V_y,KN	Δ_y,mm	V_{max},KN	Δ_{max},mm	Initial Stiffness,KN/m
1	1.0	2458	37	2930	90	66432
2	2.0	3012	56	3507	300	53785
3	3.0	1687	22	2490	49	76681
Model with sandwich panel infill panel						
4	1.0	4974	23	5540	30	216260
5	2.0	7920	20	10640	300	396000
6	3.0	9699	16	11897	53	606187

5.4 Three storey models

The yielding of steel plate with aspect ratio 1.0 took place at a load of 1572 KN with a displacement of 64 mm which is about 0.53% of the storey height and the ultimate load before the failure was calculated as 2180 KN with a displacement of 110 mm which is about 2% of the storey height . For the same aspect ratio of 1.0 the sandwich panel model attained a yield load of 4119 KN with a displacement of 39 mm which is about 0.4 % of storey height and ultimate load attained was 4959 with corresponding displacement of 59 mm which is about 0.83 % of storey height. Both models didn’t achieve the target displacement of 300 mm. As mentioned earlier, this is mainly because of the strength degradation of boundary elements at hinge locations.

For aspect ratio 2.0 the steel plate shear wall the yielding took place at a load of 1510 KN at 72 mm displacement which is about 2.4 % of storey height and ultimate load was calculated as 2291 KN with a displacement of 279 mm which is 9.3 % of storey height. The sandwich panel model achieved a yield point load of 6634 KN and ultimate load was calculated as 8571 KN at a corresponding of displacement of 32 mm (0.106 % of storey height) and 300 mm (10

% of storey height) respectively. Similarly for model with aspect ratio 3.0 the results are tabulated in table 5.3.

Table 5.3 Results of Three storey model

Model with steel infill plate						
S.No	L/h	V_y,KN	Δ_y,mm	V_{max},KN	Δ_{max},mm	Initial Stiffness,KN/m
1	1.0	1572	64	2180	110	24562
2	2.0	1510	72	2291	279	20972
3	3.0	1120	24	1540	75	46666
Model with sandwich panel infill panel						
4	1.0	4119	39	4959	59	105615
5	2.0	6634	32	8571	300	207312
6	3.0	9053	26	10540	129	348192

5.5 Comparison of Initial Stiffness

The initial stiffness was calculated using the slope of the linear part of the push over curve which can approximately expressed as:

Initial stiffness = load at yield/deflection at yield = V_y/Δ_y

The results for each case were summarized in the following bar charts

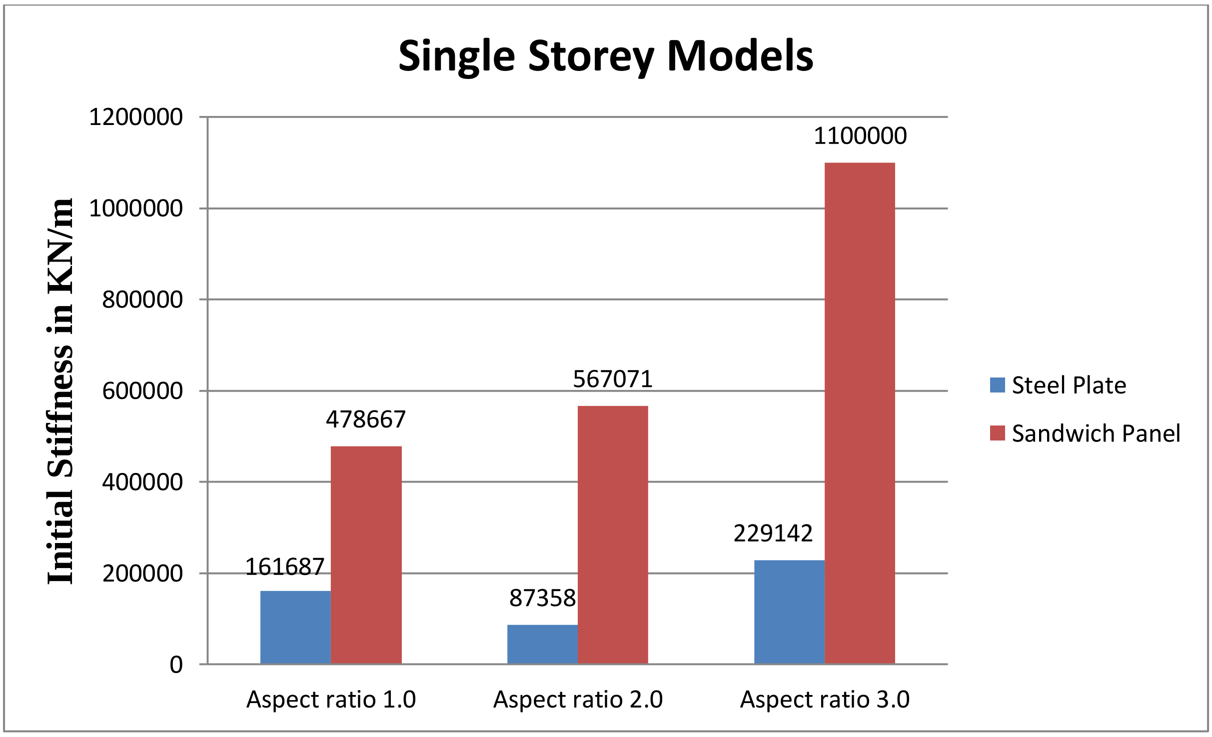


Figure 5.1 – Single storey models-Comparison of initial stiffness

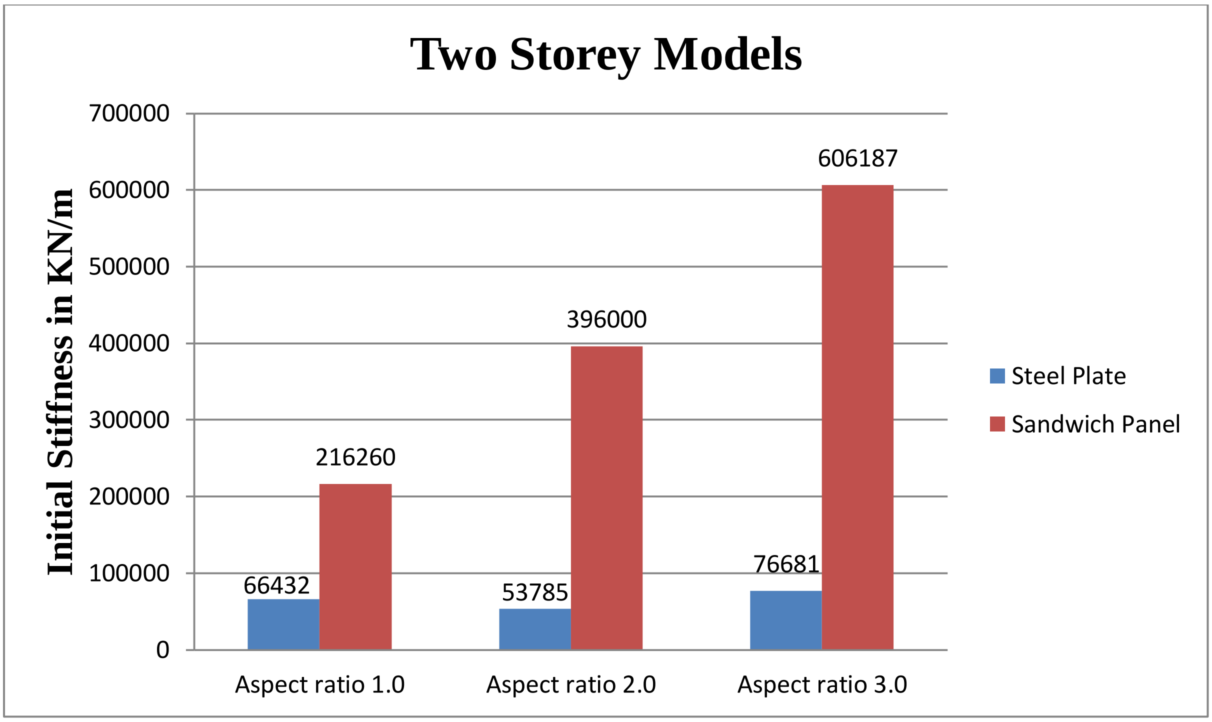


Figure 5.2 – Two storey models-Comparison of initial stiffness

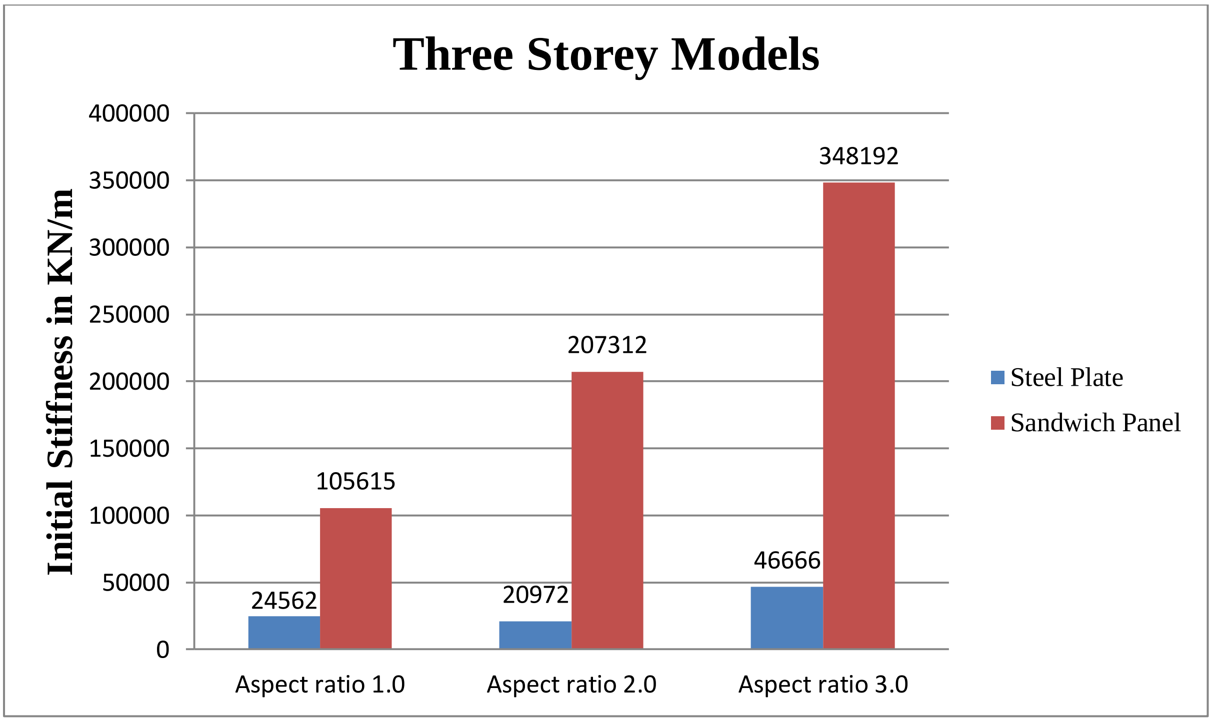


Figure 5.3 – Three storey models-Comparison of initial stiffness

It is clearly visible from the bar charts that sandwich panels possess way more stiffness as compared to their counterpart steel plate shear walls. On comparing it can be concluded that the stiffness is about 5 to 7 times greater than that of steel plate infill shear wall.

5.6 Comparison of weight:-

As Metal foams are light in weight, the total weight of Sandwich panels is lighter as compared to the weight of steel plates. This is summarized in the following tables-

Table 5.4- Weight of SPSW in Single Storey models-

S.no	Aspect Ratio L/h	Steel Plate (in Kg)	Sandwich Panel (in Kg)
1	1.0	127.2	90.46
2	2.0	254.34	182.169
3	3.0	381.51	271.188

Table 5.5-Weight of SPSW in Two Storey models-

S.no	Aspect Ratio L/h	Steel Plate (in Kg)	Sandwich Panel (in Kg)
1	1.0	233.75	165.78
2	2.0	466.29	339.821
3	3.0	700	497.88

Table 5.6- Weight of SPSW in Three Storey models-

S.no	Aspect Ratio L/h	Steel Plate (in Kg)	Sandwich Panel (in Kg)
1	1.0	296.76	210.97
2	2.0	593.46	423.211
3	3.0	890.755	633.47

From the values shown in the table it can be concluded that Sandwich panels are lighter than steel plates and hence they offer light weight construction along with much higher stiffness than Steel plate shear walls.

6. CONCLUSION AND POSSIBLE FUTURE WORK

6.1 CONCLUSION

In the present work, the main objective was to check the potential use of Metal foam panels in place of steel plates. Different models were created in SAP2000 using both conventional steel plates and sandwich infill plates and push over analysis was performed on them. The parameters on which both the models were compared with each other are Ultimate load carrying capacity, Initial Stiffness and Weight. From the results obtained the following inference can be drawn that the sandwich panels performed better than conventional steel plate counter parts for all the parameters that were considered.

On calculating the ratio of maximum deflection to the deflection at yield point it was observed that the Sandwich panels had undergone a maximum displacement of about 9.3 to 15.7 times the displacement at yield point where as the ratio of maximum displacement to the yield point displacement for typical steel plate models was about 3.8 to 7.2. This ratio helps in evaluating the ductility of the system. Results are summarized in the following table.

Table 6.1 Mean values of Displacement and Stiffness ratio

Model	$\Delta_{max,sand}/\Delta_{Y,sand}$	$K_{i,sandwich}/K_{i,steel}$
Single storey	15.71	4.81
Two storey	15	6.17
Three storey	9.375	7.21

Where, Δ_y is the deflection at yield point Δ_{max} is the maximum displacement achieved and K_i is the initial stiffness.

On the basis of results derived from the analysis, it can be inferred that the sandwich panel infill plates have better ultimate load capacity, initial stiffness, redundancy and ductility capacities as compared to the conventional steel infill plate shear walls. Moreover they are lighter in weight. Metal Foam Sandwich panels have outperformed Steel infill plates in every aspect. All the characteristics that these Sandwich Panels possess can be utilized in resisting the lateral loads exerted by wind and seismic forces on buildings.

6.2 POSSIBLE FUTURE WORK

Even though the results of models created in the current work suggest that Sandwich panels have better properties, it is required that experimental work should be conducted to cross check the results and verify that the results obtained are reliable. The prototype of Metal foam Sandwich panel shear wall should be prepared and experiments should be performed on numerous models to prepare a database for this type of shear wall. The type of boundary elements required, connection between infill panel and boundary elements, construction technique etc needs to be generated to meet specific requirements for sandwich panels. Further studies should be conducted and codes should be prepared so as to set up some standards for the use of structural engineers working in design office.

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