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Push Over Analysis of Unstiffened Steel Plate Shear Wall

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Abstract—***Steel Plate Shear Wall (SPSW) is made from thin steel plate which in turn are framed by the beams and columns of structural system. In recent year they are proved to be very effective and economical for resisting lateral loads such as earthquake and wind loads.The present work targets to study the behaviour of single frame SPSW with different plate thicknesses, rigid and non-rigid frame members, pinned and moment resisting joints. For this purpose, pushover analysis is performed using ANSYS. Push-over curves and distribution of stresses in the plate is demonstrated. A Comparative study for different stiffness of shear wall to resist the lateral loads is carried out. Ultimate load carrying capacity of the plate is determined for different model. It is found that the effect of stiffness of the beams and columns on the ultimate load carrying capacity of SPSW may be ignored until the members are strong enough to resist the force applied on them before tearing failure of plate. The ultimate load carrying capacity increases linearly with increasing the thickness of steel plate. For moment resisting frame with no shear plate, the initial stiffness is low, but it is increased by many folds when steel plate is also used to resist the lateral load.***

Keywords— ***Steel plate shear wall, push over analysis, stiffness, load-displacement curve, stress***

I. INTRODUCTION

A steel plate shear wall (SPSW) is a lateral load resisting system consisting of vertical steel plate infill connected to the surrounding beams and columns and installed in one or more bays along the full height of the structure to form a cantilever wall. SPSW subjected to cyclic inelastic deformations exhibit high initial stiffness, behave in a very ductile manner, and dissipate significant amounts of energy. These characteristics make them suitable to resist and dissipate seismic loading. SPSWs can be used not only for the design of new buildings but also for the retrofit of existing constructions. Beam-to-column connections in SPSWs may in principle be either of the simple type or moment-resistant. Prior to key research performed in the 1980‟s, the design limit state for SPSW was considered to be out-of-plane buckling of the infill panel. To prevent buckling, engineers designed steel walls with heavily stiffened infill plates that were not economically competitive with reinforced concrete shear walls. However, several experimental and analytical studies using both quasi-static and dynamic loading showed that the post-buckling strength of thin SPSW can be substantial. The web plates in steel plate shear walls are categorized according to their ability to resist buckling. The web plates can be sufficiently stiffened to preclude buckling and allow the full shear strength of the web to be reached. These are known as "stiffened" web plates. While stiffening increases the effectiveness of a web plate, it is typically not as economical as the use of the "unstiffened" web plate in which buckling of the web plate is expected. The vertical steel plate connected to the columns and beams is referred to as the web plate. The columns in SPSW are referred to as vertical boundary elements (VBE) and the beams are referred to as horizontal boundary elements (HBE). In typical designs the webs of steel plate shear walls are unstiffened and slender. The webs are therefore capable of resisting large tension forces, but little or no compression. As lateral loads are imposed on the system, shear stresses develop in the web until the principal compression stresses (oriented at a 45° angle to the shear stress) exceed the compression strength of the plate. At this point, the web plate buckles and forms diagonal fold lines.

The first comprehensive analytical investigation of conventional unstiffened steel plate shear wall was conducted at University of Alberta. Thorburn *et al*. [18] recognized that it is overly conservative to neglect the post buckling strength of shear wall bounded by columns and beams. An unstiffened steel panel of usual dimension buckle immediately upon being loaded. Strength of the post buckled tension field is the primary stress-resisting mechanism and that the shear resistance prior to buckling can be neglected. Caccese *et al.* [21] investigated the behaviour of unstiffened thin SPSW and concluded that frames with thinner plates exhibit an inelastic behaviour that is controlled primarily by yielding of the thin plate, and the nonlinear system behaviour is predominantly due to the stretching of the plate and the formation of a diagonal tension field. The frames with thicker plates show an inelastic behaviour that is primarily governed by the columns, and the capacity of the frames with the thickest plate is limited by the instability of the column.Recent research on the post buckling strength of plate girder webs using finite element analysis also provided insight into the post buckling behaviour and bending-shear interaction of steel plate shear walls having web width-to-thickness ratios comparable to typical plate girders (i.e., 350 or less) (Shahabian [22]). Choi and Park [9] experimentally and analytically showed that the steel plate walls with relatively thin plates show shear-dominated behaviour by the moment–frame action. On the other hand, steel plate walls with thick plates show flexure-dominated behaviour by the cantilever action. The shear-dominated walls show better ductility. An evaluation method for the deformation mode of the steel plate walls was studied. Shear capacity V_s (base–shear capacity controlled by the shear-dominated behavior) and flexural capacity V_f (base–shear capacity controlled by the flexure-dominated behaviour) were defined. If $V_s \geq V_f$, the steel plate wall shows shear -dominated behaviour; otherwise, it shows flexure-dominated behaviour. Choi and Park [11] performed an experimental study to investigate the potential maximum ductility and energy dissipation capacity of steel plate walls with thin infill plates. Three specimens of a three-story steel plate wall were tested. A concentrically braced frame (CBF) and a moment-resisting frame (MRF) were also tested for comparison. To maximize the ductility and energy dissipation capacity of the steel plate walls, ductile details were used. The test parameters were the aspect ratio of the infill plate and the shear strength of the column. The steel plate walls exhibited much better ductility and energy dissipation capacity as compared to the CBF and MRF and shear strength and energy dissipation capacity of the steel plate walls increased in proportion to the depth of the infill steel plate. Choi and Park [12] studied the effect of the connection method between the boundary frame and the infill plate (welded connection versus bolted connection), length of the welded connection (full connection versus partial connection), and opening in the infill plate (solid wall versus coupled wall). The result indicated that for architectural reasons and enhancement of constructability, various infill plate designs can be used in practice without a significant reduction in the structural capacity of the steel plate walls. Shishkin *et al*. [16] refined the, widely used, strip model based on phenomena observed during loading of steel plate shear walls in the laboratory. Since the original strip model was proposed as an elastic analysis tool, refinements were made primarily to achieve an accurate representation of yielding and eventual deterioration of the wall, although moderate improvements in initial stiffness predictions were also made. In assessing each of the proposed refinements, modelling efficiency was evaluated against the accuracy of the solution. A parametric study using the modified strip model examined the effect of varying the angle of inclination of the tension strips on the predicted inelastic behaviour of the model. Notably, it was found that the ultimate capacities of steel plate shear wall models with a wide variety of configurations vary little with the variation of the inclination of the strips. The tears in the infill plate of the test specimen that arose principally due to the kinking of the stretched plate during load reversals were observed to have formed and propagated primarily in the corners of the infill plates. The tearing of the infill plate contributed to the gradual deterioration in the strength of the specimen.

In the present study, the effect of thickness of the steel plate on the initial stiffness of steel plate shear wall against lateral loads and ultimate lateral load is analyzed. The analysis also carried out to study the effect of making beam-column connections pinned or moment resisting on the initial stiffness of steel plate shear wall against lateral loads and ultimate lateral loads bearing capacity.

II. MODELLING

Three sets of SPSW specified by RB, NRB, and M are modelled in ANSYS. The length and height of all the models are 1500 mm and 1000 mm respectively. In the first set the supporting beams and columns were made perfectly rigid and connections between them were pinned connection. This set consisted of five models having plate thicknesses 1mm, 2mm, 4mm, 6mm and 8 mm and are denoted as RB1, RB2, RB4, RB6 and RB8 respectively as shown in Fig. 1.

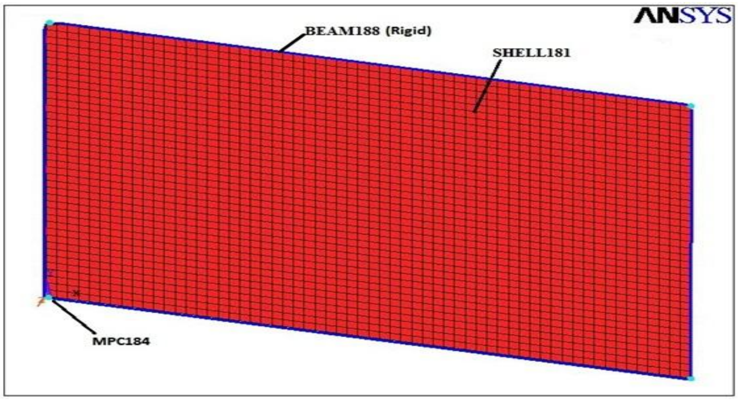

Fig. 1: Model RB1, RB2, RB4, RB6, RB8

For the second set, the beam-column connections are same as the first one but instead of rigid frame members, steel members with a cross section of 200 mm X 200 mm is used. These are denoted as NRB1, NRB2 and NRB4 with plate thicknesses 1mm, 2mm and 4mm as shown in Fig. 2.

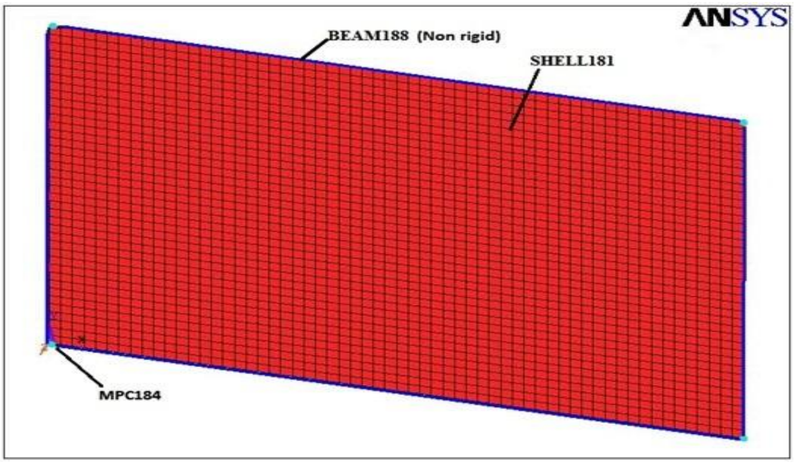

Fig. 2: Model NRB1, NRB2, NRB4

For the third set of models, other conditions are same as the second set but the beam column connection are made moment resisting. The third set has four models. The first one(M0) without any shear plate and the rest having plate thicknesses 4mm, 6mm and 8mm and denoted M4, M6 and M8 respectively shown in Fig. 3 and 4.

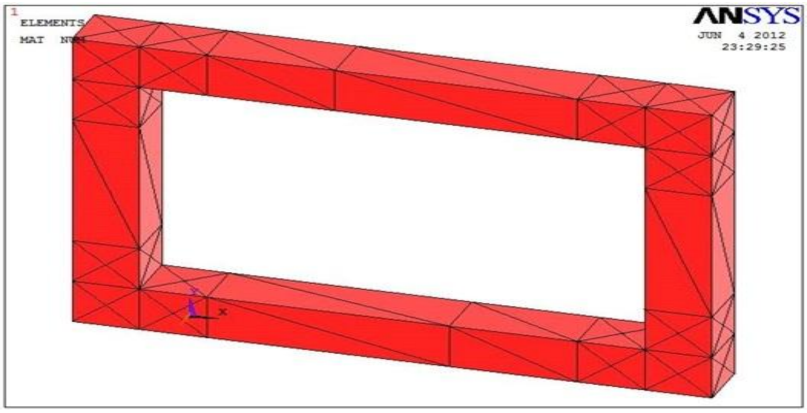

Fig. 3: Model of M0

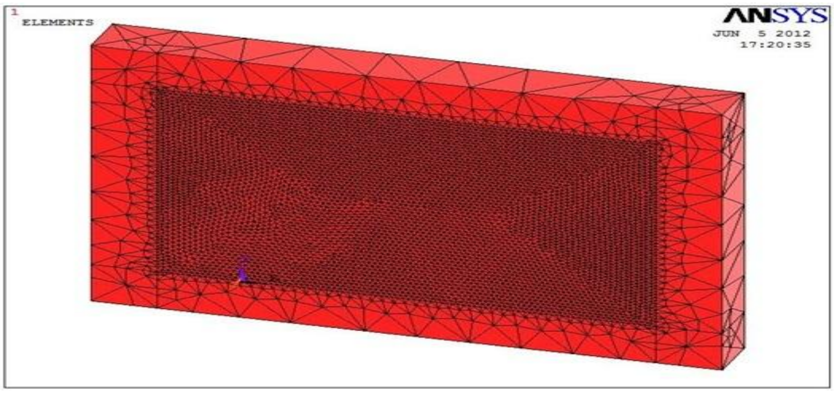

Fig. 4: Model M4, M6, M8

The parameters are specified in Table 1 for different plate. The analysis is carried out on the basis of these values.

TABLE 1: PARAMETERS FOR ANALYSIS OF THE MODEL

Model	*Thickness (mm)*	*Model*	*Thickness (mm)*
RB1	*1*	*NRB2*	*2*
RB2	*2*	*NRB4*	*4*
RB4	*4*	*M0*	*No plate*
RB6	*6*	*M4*	*4*
RB8	*8*	*M6*	*6*
NRB1	*1*	*M8*	*8*

In models- RB1, RB2, RB4, RB6 and RB8 the connections were pinned connections and the frame members were made rigid. These were modelled using the following elements in ANSYS

1. Shell181 (4 node shell) to model the plate.
2. Beam188 (2 node beam) to model rigid frame members with cross section 200mmX200mm.
3. MPC184 (revolute joint) to model the pinned connections between frame members.

In models- NRB1, NRB2 and NRB4, the connections were pinned connections and the frame members were not made rigid. These were modelled using following elements –

1. Shell181 (4 node shell) to model the plate.
2. Beam188 (2 node beam) to model non rigid frame members with cross section 200mmX200mm.
3. MPC184 (revolute joint) to model the pinned connections between frame members.

In models- M0, M4, M6 and M8 the connections were rigid connections and the frame members were not made rigid. The twenty-node solid elements (solid186) were used to model both plate and frame members by the auto meshing of volumes.

The material properties of steel plate were the modulus of elasticity E_s=200 GPa, yield stress f_{py}=300 MPa, strain-hardening ratio =*Ep* □*Es*= 0.003, and Poisson‟s ratio = 0.3. The von Mises yield criterion was used to define the yield strength of the steel plate. The linear kinematic hardening model was used to address the Bauschinger effect.

A. Element Properties stipulated in ANSYS software

The BEAM188 element is suitable for analysing slender to moderately stubby/thick beam structures. This element is based on Timoshenko beam theory. Shear deformation effects are included. It is a linear (2-node) beam element in 3-D with six degrees of freedom at each node. The degrees of freedom at each node include translations in x,y, and z directions, and rotations about the x,y, and z directions. Warping of cross sections is assumed to be unrestrained. The beam elements are well-suited for linear, large rotation, and/or large strain nonlinear applications.

SHELL181 is suitable for analysing thin to moderately-thick shell structures. It is a four-noded element with six degrees of freedom at each node: translations in the x, y, and z directions, and rotations about the x, y, and z axes. The degenerate triangular option should only be used as filler elements in mesh generation. It is well-suited for linear, large rotation, and/or large strain nonlinear applications. Change in shell thickness is accounted for in nonlinear analyses. In the element domain, both full and reduced integration schemes are supported. It accounts for follower effects of distributed pressures.

SOLID186 is a higher order 3-D 20-node structural solid element. SOLID186 has quadratic displacement behaviour and is well suited to modelling irregular meshes (such as those produced by various CAD/CAM systems). SOLID95 is a similar element; however, SOLID186 provides more nonlinear material models and uses consistent tangent stiffness for large strain applications. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions. SOLID186 may have any spatial orientation. The element supports plasticity, hyper-elasticity (the Arruda-Boyce model), creep, stress stiffening, large deflection, and large strain capabilities.

MPC184 rigid link/beam element can be used to model a rigid constraint between two deformable bodies or as a rigid component used to transmit forces and moments in engineering applications. This element is well suited for linear, large rotation, and/or large strain nonlinear applications.

The kinematic constraints are imposed using one of the following two methods:

- The direct elimination method, wherein the kinematic constraints are imposed by internally generated MPC (multipoint constraint) equations. The degrees of freedom of a dependent node in the MPC equations are eliminated in favour of an independent node.
- The Lagrange multiplier method, wherein the kinematic constraints are imposed using Lagrange multipliers. In this case, all the participating degrees of freedom are retained.

III. RESULTS AND DISCUSSIONS

The load displacement curve for model RB1, RB2, RB4, RB6, RB8 is presented in Fig. 5. The push-over analysis of SPSW with rigid frame-members and pinned connections showed that their ultimate lateral load carrying capacities are proportional to the thickness of the plates. Initial stiffness is varying linearly with the thickness of plate. The ultimate lateral load carrying capacities of SPSW with pinned beam-column connections increase linearly with increase in the thickness of the steel plate.

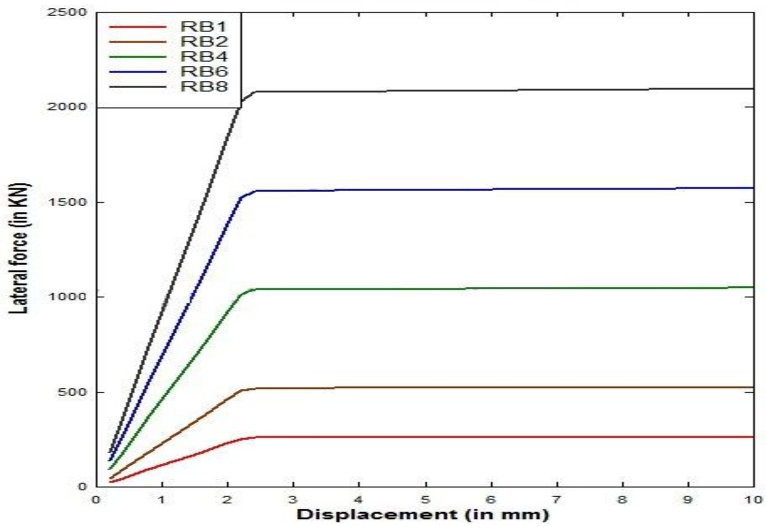

Fig. 5: Load displacement curves for RB1, RB2, RB4, RB6, and RB8.

The lateral load carrying capacity of the RB model is presented in Table 2 for different thickness of the plate model.

TABLE 2: LATERAL LOAD CAPACITY PER UNIT PLATE THICKNESS FOR RB MODELS.

Model	Thickness of plate (mm)	Ultimate lateral load capacity (KN)	(Lateral load capacity)/(Thickness) (KN/mm)
RB1	1	262.1589	262.1589
RB2	2	524.3179	262.1589
RB4	4	1048.6369	262.1592
RB6	6	1572.9614	262.1602
RB8	8	2097.3036	262.1630

The initial stiffness per unit of plate thickness is presented in Table 3 for RB model.

TABLE 3: INITIAL STIFFNESS PER UNIT OF PLATE THICKNESS FOR RB MODELS.

Model	Thickness of plate (mm)	Initial stiffness (KN/mm)	(Initial stiffness)/(Thickness) (KN/mm)
RB1	1	115.3869	115.3869
RB2	2	230.7739	115.3870
RB4	4	461.5499	115.3875
RB6	6	692.3256	115.3876
RB8	8	923.1034	115.3879

The load displacement curve for RB moldel and NRB model is presented in Fig. 6 for different diplacement at the top corner of the SPSW model keeping parameter similar as previous.

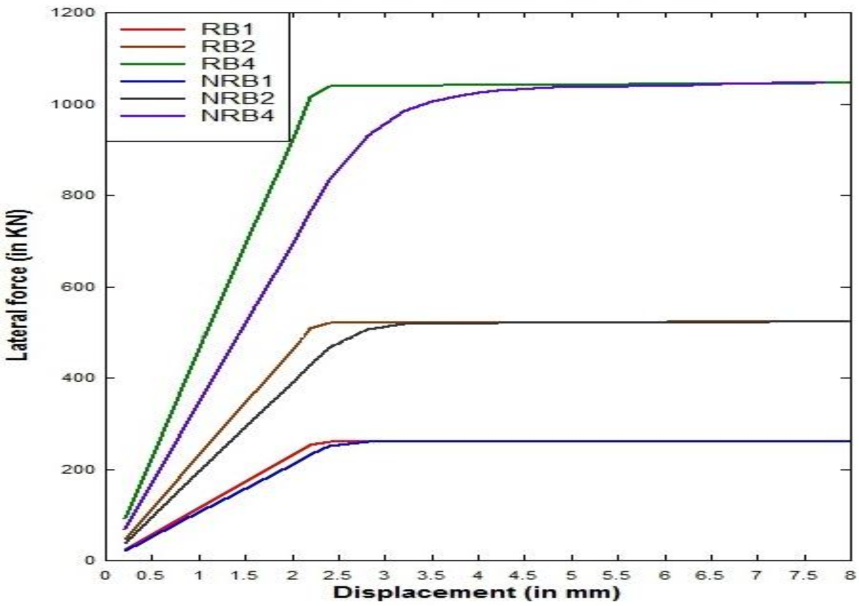

Fig. 6: Load displacement curves for RB1, RB2, RB4 and NRB1, NRB2, NRB4.

When rigid frame members of cross section 200mm X 200mm are used, the ultimate load carrying capacity slightly decreased. The difference in the ultimate load carrying capacity is so low that the effect of stiffness of the frame members can be ignored, until the members are strong enough to resist the force applied on them by the plate and do not fail before the tearing of plate.

TABLE 4: PERCENTAGE DECREASE IN ULTIMATE LATERAL LOAD

Model	Ultimate lateral load (KN)	Model	Ultimate lateral load (KN)	% Decrease in Ultimate lateral load (KN)
RB1	262.1589	NRB1	261.6745	0.184
RB2	524.3179	NRB2	521.2504	0.585
RB4	1048.6369	NRB4	1046.6570	0.188

Though, there is not much effect on the ultimate load carrying capacity of the SPSW, but the initial stiffness is considerably reduced. Moreover, the %decrease in the initial stiffness of the SPSW increased with increase in the thickness of the plate as presented in Table 4 and 5.

TABLE 5: PERCENTAGE DECREASE IN INITIAL STIFFNESS.

Model	Initial stiffness (KN/mm)	Model	Initial stiffness (KN/mm)	% Decrease in Initial stiffness (KN)
RB1	115.3869	NRB1	105.2397	8.79
RB2	230.7739	NRB2	195.1850	15.42
RB4	461.5499	NRB4	347.0703	24.8

The displacement curve for different lateral forces in different model is presented in Fig. 7 and 8. On comparing the load displacement curves of all the models with plate thickness of 4mm, it is found that M4 had the maximum stiffness, whereas NRB4 is the least stiff. Figure 7 and 8 shows that the shear wall with moment resisting beam-column connections has higher stiffness as well as higher load carrying capacity than the one with pinned connections. The former has higher bending stresses, as a fraction of the lateral load is transferred to the frame members as moment.

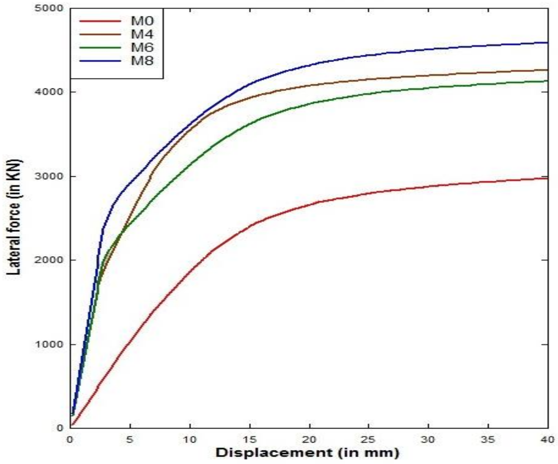

Fig. 7: Load displacement curves for M0, M4, M6, M8

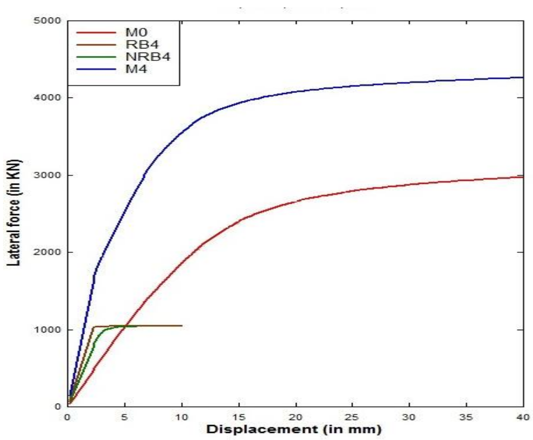

Fig.8. Load displacement curves for M0, RB4, NRB4, M4**.**

This behaviour can be attributed to the fact that strain is not same throughout the plate for NRB models. So, initially stress also varies and the plate is not used at its full capacity. As a result there is a decrease in the initial stiffness of the SPSW. But, as the strain reaches the plastic zone, most of the portion has the same stress, i.e. ultimate stress, and thus the ultimate load carrying capacity is nearly equal to the corresponding RB model as shown in Fig. 9.

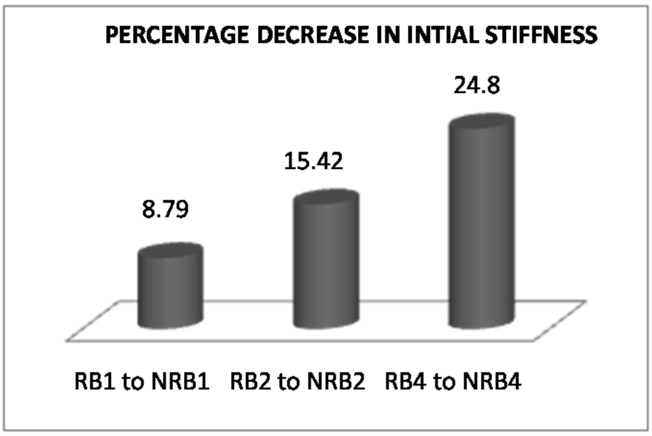

Fig. 9: Percentage decrease in the initial stiffness if frame members are made non-rigid.

It is observed from Fig. 10 that, if the frame members are considered to be rigid, then, at a particular instance of time, stress at every point in the plate is same. Stress contours of NRB1amd M0 indicate that if the beams and columns do not behave like perfectly rigid members, then more strain is developed in the middle region of the plate and decreases gradually towards the two corners in the perpendicular direction to the tension field.

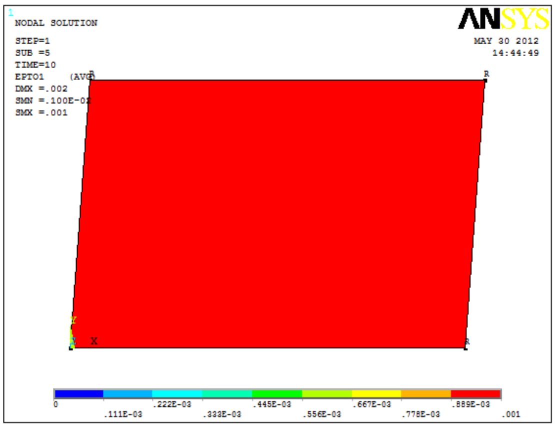

Fig. 10(a) Stress distribution for model RB1

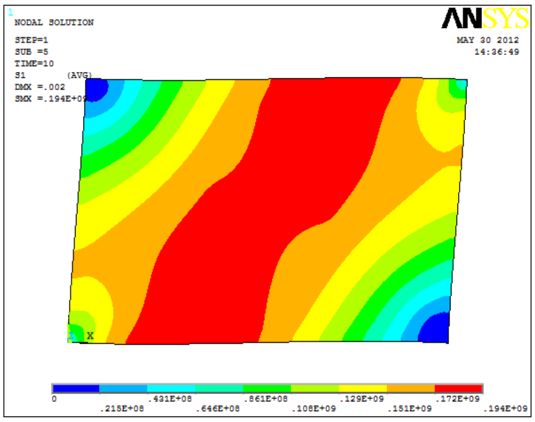

Fig. 10(b) Stress distribution for model NRB1

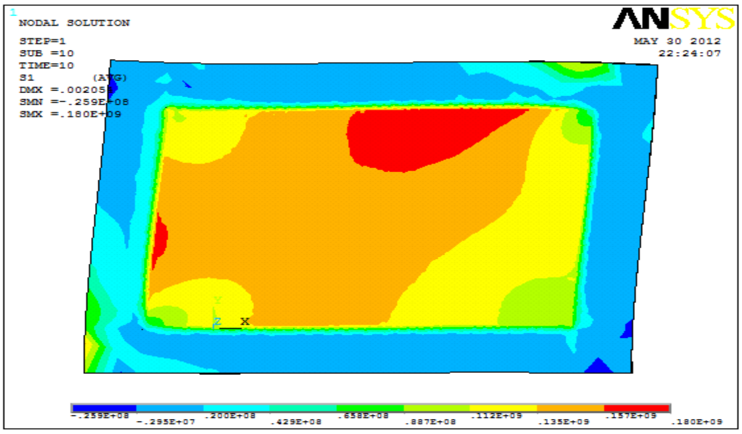

Fig. 10(c) Stress distribution for model M4 for a displacement of 2 mm

IV. CONCLUSIONS

The steel plate wall has a large initial stiffness and load-carrying capacity. SPSW with relatively larger aspect ratio exhibits the greater load-carrying capacity and deformation capacity than the one with smaller aspect ratio. Both strength and stiffness ratios are approximately the same as the ratio of their aspect ratios. In the shear-dominated walls, most infill steel plates yield early across the wall height, and at a large plastic deformation, plastic hinges are developed at the frame members by the moment frame action. In the flexure-dominated walls, the columns in the first storey yield and a plastic hinge are developed at the wall base. SPSW systems are usually more flexible in comparison to concrete shear walls, primarily due to their flexural flexibility. Therefore, when using SPSW in tall buildings, the engineer must provide additional flexural stiffness. Large composite concrete infill steel pipe columns can be used at all corners of the core wall to improve the system‟s flexural stiffness as well as its overturning capacity. Load deflection curve of SPSW is linear for smaller values of displacement but it attains a constant value, known as the ultimate lateral load carrying capacity of the SPSW with increase in displacement. If the beams and columns do not behave like perfectly rigid members, then more strain is developed in the middle region of the plate and decreases gradually towards the two corners in the perpendicular direction to the tension field. The ultimate lateral load carrying capacities of SPSW with pinned beam-column connections increase linearly with increase in the thickness of the steel plate. The initial stiffness of SPSW with pinned beam-column connections increase linearly with increase in the thickness of the steel plate. For SPSW with pinned connections, the effect of stiffness of the frame members on the ultimate load carrying capacity of SPSW can be ignored, until the members are strong enough to resist the force applied on them by the plate and do not fail before the tearing of plate. But it increases linearly with increase in the thickness of steel plate used.

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