Zhongguo Kuangye Daxue Xuebao
Journal of China University of Mining and Technology

2024 | Vol 29 | Issue 2 | Page 53-76
Journal Homepage: https://zkdx.ch/
DOI: 10.1654/zkdx.2024.29.2-5

Behavior of Steel Plate Shear Wall Subjected To Lateral Loads

Alaa Abdelsalam*
Department of Construction Engineering, Misr University for Science & Technology, First 6th of October, Giza Governorate, Egypt
*Corresponding author

Arafa El-Helloty
Department of Civil Engineering, Al-Azhar University, Nasr City, Cairo Governorate, Egypt

Ayman Sammra
Department of Civil Engineering, Al-Azhar University, Nasr City, Cairo Governorate, Egypt

Abstract
In recent years, the need for lateral loads upgrades in building structures has increased due to the growing importance of steel plate shear walls (SPSW) for earthquake and wind resistant design. SPSW systems offer advantages such as robust post-buckling strength, substantial ductility, stable hysteretic characteristics, and high initial stiffness compared to conventional lateral load resisting systems. They allow for less structural wall thickness and lesser building weight compared to concrete shear walls, reducing construction time, and allowing for fast erecting without a curing period. Structures can be subjected to dynamic loads such as wind, waves, traffic, earthquake, and blasts, which can cause critical stresses in buildings, leading to excessive lateral sway and undesirable stresses and vibrations. Design and structural evaluation of building systems subjected to lateral loads are crucial. The present generation faces the challenge of providing adequate strength and stability of buildings against lateral loads. Different lateral load resisting systems are used in high-rise buildings due to the concern of earthquake-induced lateral loads. Steel plate shear walls (SPSW) have been extensively used as lateral load resisting systems in the past few decades. They consist of steel infill plates surrounded by boundary beams and columns and can be constructed in two types: unstiffened and stiffened infill steel plates. A cantilevered vertical plate girder is idealized as a SPSW system, with the steel infill plates acting as the web, boundary columns as flanges, and boundary beams as transverse stiffeners. SPSW systems have been researched since the early 1970s, with the most common research and application in North America being the unstiffened, thin SPSW system, while stiffened SPSW systems are more common in European area. Regardless of the system used, the determination of whether a SPSW system is the right application in general is important.

Keywords
Steel plate shear wall, Dynamic Analysis, Seismic load, Wind load, Modal Analysis, Stiffened steel plate shear wall, Unstiffened steel plate shear wall, Frequency

1. Introduction
Dynamic loads, such as wind, waves, traffic, earthquake, and blasts, can cause critical stresses in buildings, leading to excessive lateral sway and undesirable stresses and vibrations. Design and structural evaluation of building systems subjected to lateral loads are crucial for the present generation, as they face the challenge of providing adequate strength and stability against these forces. Steel plate shear walls and steel bracings systems are commonly used as lateral load resisting systems in high-rise buildings, as earthquakes pose a concern. Steel plate shear walls are known for their ability to dissipate high energy and provide ductility in extreme load events. These systems consist of steel infill plates surrounded by boundary beams and columns and can be constructed in two types: unstiffened and stiffened infill steel plates. They can be idealized as a cantilevered vertical plate girder, with steel infill plates acting as the web, boundary columns as flanges, and boundary beams as transverse stiffeners. Steel plate shear walls are an innovative lateral load resisting system that effectively braces a building against wind and earthquake forces. The system consists of vertical steel infill plates connected to the surrounding beams and columns, installed in one or more bays for the full height of a building to form a stiff cantilever.

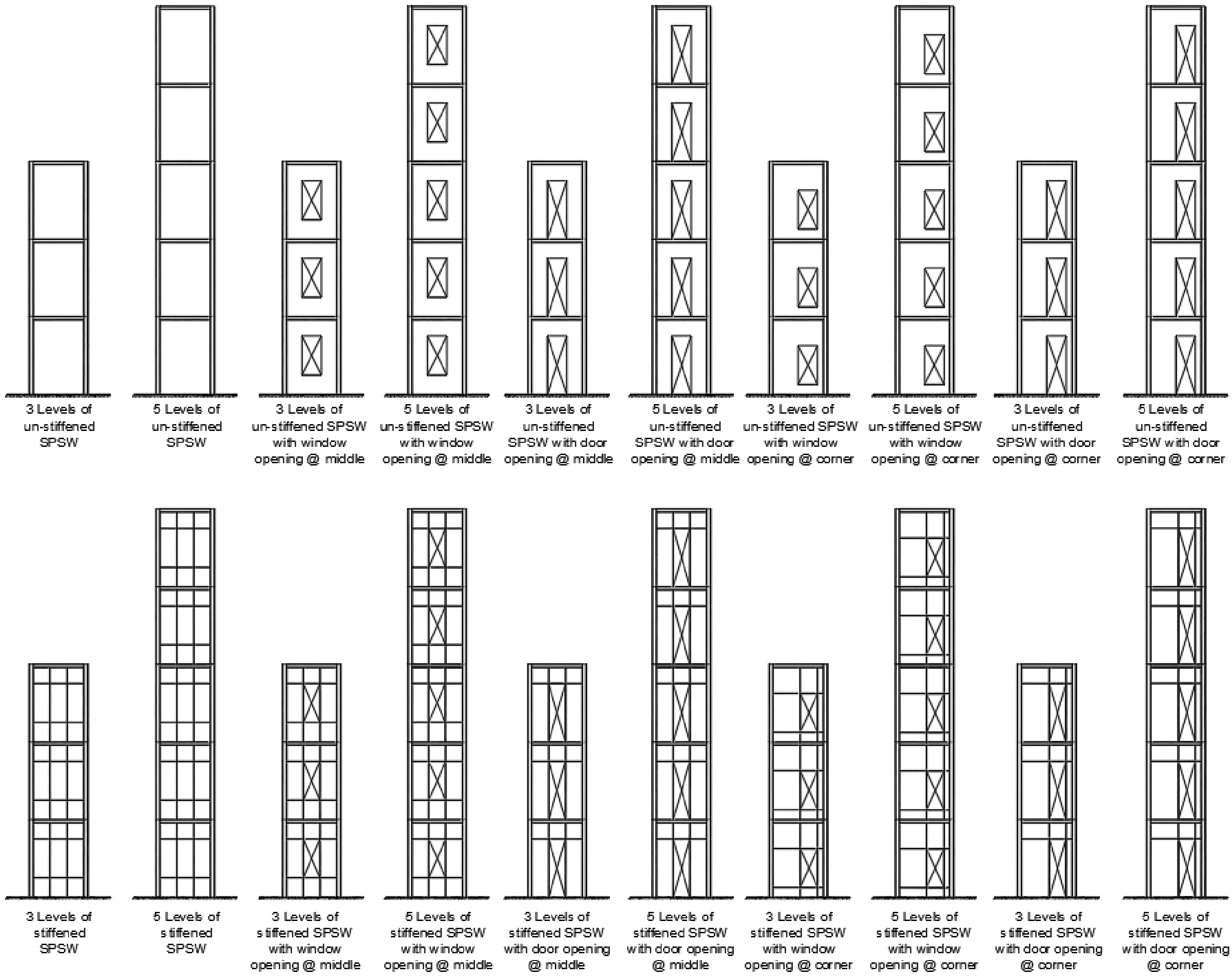


Fig. 1 Studied SPSW Models.

2. Literature Survey

Please Yuben Zhang and Xun Zhan (2019) Low cyclic loading tests were conducted on welded steel frame-steel plate shear wall structures and assembled steel frame-steel plate shear wall structures with discontinuous cover-plate connections. The assembled structure with DCPC has a full shuttle-shaped hysteretic curve, stable hysteretic performance, and good ductility. The structure conforms to the seismic design concept of "strong frame, weak wallboard, strong column, and weak beam." The energy dissipation capacity is better than the welded structure, and the assembly reduces the number of connecting bolts and is easy to assemble.

Yang Lv , Ling Li and Di Wu (2019) Four scaled SPSWs were tested to investigate the impact of gravity load on their cyclic performance. Results showed that the shear resistance capacity decreases with vertical load increase, specimen failure under heavy gravity load is due to global out-of-plane buckling, the analytical model overestimates the stiffness of the SPSW, and the model can predict the axial stress distribution of the infill steel plate.

Dr. B P Annapoorna and Mohammed Ali Boodihal (2019) found out that the minimum thickness for plate girder design, with a maximum lateral displacement of 131.2 mm, is outlined in the Codal provisions IS 1893: 2002. Within these limits, the maximum displacement is found for all five plate thicknesses. In terms of minimizing in-plane displacement, double diagonal stiffeners are slightly better than single diagonal stiffeners. When used in conjunction with the other five thicknesses, 8US is more efficient. While 12US and 16US plates do not work well with single or double diagonal stiffeners, they do with 3mm, 5mm, and 8mm plates. For buildings with ten stories or more, steel plates thicker than 8 mm are advised.

H. Bakhshi and H. Khosravi (2019) found that the steel shear wall system is effective in sacrificing the shear wall for other structure members, even if the plastic hinge colors exceed the allowable limit. The maximum drift values for 3, 6, and 12-story structures were 1.35, 1.19, and 1.48 percent, which are lower than the 1.5 percent allowable limit of the code. The steel shear wall is also one of the best lateral bracing systems in high-rise buildings, as it absorbs and dissipates lateral force. By reducing the weight of the steel used, the system reduces the base shear imposed on the structure, which is more critical in short and intermediate-rise buildings. The displacements of the roof center of mass under every three earthquake records are within the code allowable range, indicating the steel wall system's good performance.

Cao et al. (2020) According to the experimental findings, the X-shaped reinforced SPSW's initial stiffness, load carrying capability, and energy dissipation are more significant by 21, 11, and 27%. On the other hand, SPSW with a thin thickness exhibits a more stable hysteresis.

Xiaoming Ma's paper (2022) presents a study on four types of Steel Plate Shields (SPSWs) and their seismic performance before and after corrosion using Fourier Transform Infrared (FEM) models. The results show that the OSPSW specimens have higher ultimate shearing resistance, initial stiffness, and ductility under cyclic loading, and a 38% higher total energy dissipation value than the FSPSW. Setting stiffeners can improve the seismic performance of OSPSW. Under atmospheric corrosion, the OSPSW's performance is less affected, but its peak load and energy dissipation capacity remain higher than the BSPSW. Slotting at the middle part effectively inhibits the reduction in seismic performance after corrosion.

The study by He, Li, Chen, and Xian (2023) developed a finite element model of Stiffened Stiffener Walls (SPSWs) using Abaqus and bidirectional progressive structural optimization. The results showed that the improved algorithm can be used for multiregional optimization, increased buckling bearing capacity, and significantly improved stiffness and initial stiffness compared to unstiffened SPSWs. The optimized stiffened SPSW also reduced noise and improved comfort.

Zhang et al. (2021) According to the experimental findings, the specimen's isolated steel sheet has strength comparable to AISI S100, and the connection's ductility—which adapts to the bearing—is good.

The study by Verma and Sahoo (2017) proposes a design methodology for staggered Static Structural Shear walls (SPSWs) with similar overstrength to conventional SPSWs. The base shear reduction factor is reduced, and a computer-aided linear static analysis procedure is used to estimate forces in VBEs. Dynamic analyses show that wider or staggered web plates reduce vertical forces on column footings by 50% to 60%, reducing steel tonnage and foundation cost. The proposed methodology may be more economical than conventional SPSWs and provides more open wall area than wide SPSWs.

Hamza Basri's (2022) study analyzed the maximum ductility, energy absorption, and stiffness degradation of two frames equipped with SPSW and CB. The results showed that the SPSW frame had a more symmetric and stable hysteresis curve, demonstrating its excellent deformation capacity. The steel plate shear wall increased lateral strength and ductility, creating symmetry in steel plate resistance during loading, allowing it to withstand more seismic energy compared to traditional braces. The study's findings suggest the potential of steel plate shear walls in seismic energy management.

Lu and colleagues (2021) Studies demonstrate that a structure featuring a thicker steel plate and flexural link layer has superior energy dissipation and a higher ultimate bearing capacity. But even so, its ability to recenter has declined.

Omid Haddad (2017) his paper examines the cyclic behavior of stiffened and unstiffened SPSWs. The study consists of five test specimens, including two unstiffened plates (aluminum and steel), and three specimens stiffened using cross, circular, and diagonal stiffeners. The aluminum plate (AL-SPSW) showed good cyclic performance before yielding, but brittle failure occurred due to the material-hardening effect. The unstiffened steel plate (US-SPSW) was ductile and had excellent deformation capacity, with energy-dissipation performance better after a 6.47% drift. The installation of stiffeners significantly increased shear strength, with the cross-stiffened specimen (CS-SPSW) showing the validity of the suggested equations. The stiffeners increased shear stiffness, energy dissipation, and ductility, especially for the cross-stiffened specimen (CS-SPSW). The stiffness of an unstiffened specimen was influenced by its material.

Yuqing Yang's research (2022) explores the impact of connectivity ratio on steel plate shear wall strength. He found that infill plate connections to beams decrease shear strength by 50%. However, central plate connections to columns increase load-bearing capacity. A 67% connectivity ratio results in almost equivalent shear strength. Shear strength increases with increasing connectivity ratio, reaching over 95% of full connection type. Multi-segment connections have higher shear strength than one-segment connections.

Mojtaba Gorji Azandariani's research (2021) focused on the structural behavior and performance of SPSWPCs systems using analytical and numerical methods. The study involved 45 structures with different plate-column detachment lengths, plate thickness, and width-to-height ratios. The finite element method was validated and accurate in predicting the system's cyclic behavior, deformations, and failure modes. The results showed that increasing infill plate thickness improved strength and stiffness performances at larger aspect ratios. However, complete removal of the plate column connection could lower system strength and stiffness capacities by 23% and 26%, respectively. The analytical method was found to be valid for predicting the ultimate shear capacities of SPSW-PC systems.

According to Wang et al. (2022), the addition of a damper considerably increases lateral stiffness and energy dissipation based on experimental and numerical results.

V. Broujerdian (2022) presents a new configuration of a semi-supported shear wall with steel plates and secondary columns inclined relative to the vertical state. He found that oblique models with a sharp angle with the bottom horizon line performed better than conventional vertical-side steel plates. A semi-analytical approximate model was proposed to calculate the pushover curve of the system using SAP 2000 engineering software. The results showed that reducing the steel wall angle without changing the steel plate thickness and area increased the strength, energy absorption, and stiffness of the frame. Thinner steel plates had a greater effect on lateral strength and stiffness. An approximate method for nonlinear analysis was presented, which reduced analysis time and showed acceptable accuracy in predicting the system's strength, stiffness, and load deformation curve.

Shi et al. (2022) his experimental findings demonstrate the high shear capacity and stable mechanical behavior of two types of shear walls: CFS-C-SW and CFS-S-SW. By using a hat-section end column or installing a vertical stiffener, the CFSCSW's shear capacity can be greatly increased. Reducing the distance between the peripheral screws increases the CFS-S-SW's shear and deformation capacities, but altering the spacing between the middle studs has no effect on the outcome.

Wang et al. (2022) According to experimental and numerical results, the specimens added to the sandwich panel greatly increase the hardness and shear bearing capacity of the composite wall but decrease its ductility.

Shi and associates (2022) According to the experimental results, the seismic performance of CFS shear walls with reinforced end columns (CFSRW-R) is superior to that of ordinary CFS shear walls (CFS-RW) due to their superior geometric design.

Yigit Ozcelik's (2018) study investigates the seismic performance of B-SPSWs, an alternative SPSW configuration designed for low-seismic regions. The study involves designing 3-, 6-, and 9-story buildings in Boston using web plate strength and boundary frame demand equations. Two design approaches are considered: R = 3 without seismic design considerations and R = 3.25 using a capacity design methodology. The results show that B-SPSWs show acceptable seismic performance for both approaches, with the seismically detailed designs showing better performance. Further research is needed to verify the response modification factor, evaluate shear yielding in beam ends, and develop column design methodologies accounting for flexural demands from differential drifts.

The study by Ebadi and Farajloomanesh (2020) compared the optimal percentage of shear transfer and capacity of steel shear walls in a 10-story building with the demand shear capacity of each story. The results showed that designing the wall for higher shear percentages and neglecting columns' role in story shear capacity led to a non-economic system. The study concluded that determining the precise contribution of the steel plate and its peripheral frame for earthquake demand story shear could lead to more optimized sections in SPSW designs.

Meisam Safari Gorji and J.J. Roger Cheng's (2017) study explores the use of outrigger beams to enhance the overturning stiffness of slender and tall SPSWs. They studied four different SPSW-O configurations, focusing on seismic design and behavior. The study found that SPSW-O systems have higher stiffness, shorter fundamental periods, and reduced axial force demands on VBEs, allowing for lighter sections. The RR system was the most effective in reducing drift demands due to overturning moments. The SPSW-O configuration offers potential for improved seismic performance and material efficiency, while accommodating architectural needs.

Yigit Ozcelik and Patricia M. Clayton (2017) conducted a two-phase numerical study to propose a strip model for Building Structural Panel Sweeps (B-SPSWs). The model is based on Thorburn et al.'s strip model, but with two important parameters revisited: the inclination angle of the PTF and the compressive strength of strips. The study analyzed B-SPSWs with various aspect ratios and thin web plates under monotonic and cyclic loading. The proposed strip model showed superior performance compared to existing literature and accurately matched finite element results. The study suggests that considering web plate compressive strength in design could lead to significant material savings, making B-SPSWs a viable and cost-competitive earthquake-resistant system.

Akbar Vasseghi (2020) developed a new analytical model for seismically welded structural steel structures (SPSWs) with unstiffened infill plates. This model is useful for nonlinear response history analysis and is applicable to well-designed SPSWs. Experimental results show load-displacement characteristics of end panels differ significantly from intermediate panels, even when anchor HBE is designed according to capacity design principles. Further studies are needed to relate softening stiffness to the geometrical properties of the infill plate and boundary frame.

Mark Sarkisian's study on steel plate shear walls and ductile moment resisting frames was successful in structuring the 329.6m tall Tianjin Jinta Tower. The design was reviewed by seismic and wind experts in China, leading to enhanced analysis and performance goals. The project is nearing completion in the "construction documents" phase, with excavation and foundation construction underway.

The analysis by Rafi Ramdhani Aziz and Irpan Hidayat reveals that existing conditions in high-rise buildings have story drift and cross-sectional capabilities that do not meet design requirements or exceed the allowable limit. The use of a steel plate shear wall system with a thickness of 18.7 mm significantly affects these factors, reducing story drift values by 3-6 mm. However, increasing the wall thickness does not significantly impact beam element cross-section.

The study by Hosseinzadeh, Kontoni, and Babaei found that increasing the buckling resistance of infill steel plates increases the thickness, but also increases stress transferred to the boundary frame. They investigated the effects of steel types (A36 and LYP) on the behavior of double corrugated steel plate shear walls. Results showed that LYP steel improves seismic behavior of corrugated double walls. By increasing the e, seismic performance of the wall is reduced, but the effect of 0 on structural parameters is more significant for LYP steel. The ultimate strength and energy absorption are also reduced. The study suggests using walls with a L/H ratio greater than 1 for both A36 and LIT walls.

3. Design Methods and System Properties

This section discusses the behavior and design of Special Plate Shear Walls (SPSW) and other steel plate shear walls. It covers fundamental mechanics and analytical methods used to derive design forces for SPSW members and estimate displacement in line with building code requirements. The methods of analysis are presented in this section.

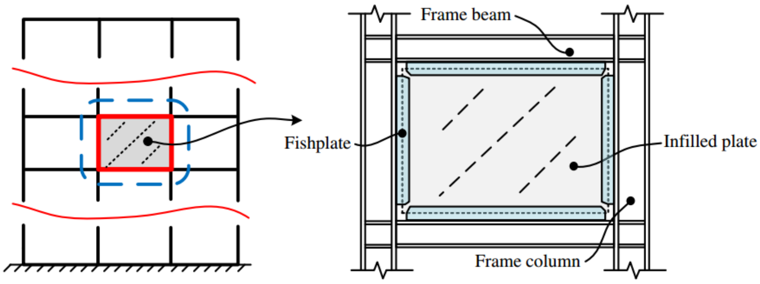


Fig. 2 Typical elements of SPSW

3.1 Mechanics

Steel plate shear walls primarily resist lateral loads through diagonal tension in the web plate and overturning forces in adjacent columns. However, slender-web steel plate shear walls (SPSW) must be examined more closely. Web plates in SPSW are unstiffened and very slender, with negligible compression strength. Stiffened steel plate shear walls, primarily used in Japan, are used to resist seismic forces by introducing stiffeners, which increase web plate strength but are not as economical as unstiffened web plates. The SPSW system is based on unstiffened, slender webs.

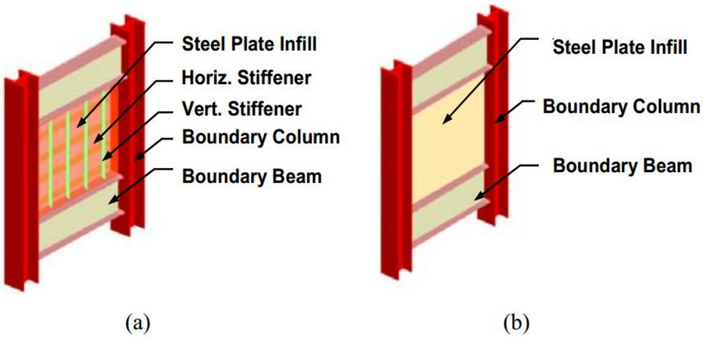


Fig. 3 Typical Stiffened and Unstiffened SPSW (a) Stiffened SPSW (b) Unstiffened SPSW

3.1.1 *Unstiffened SPSW*

SPSW (Structured Steel Frame Sweep) is a type of structural steel frame that consists of slender webs that can resist large tension forces but little or no compression. This behavior is analogous to tension-only bracing, which relies on beams in compression to transmit the horizontal component of a brace force to the brace at the level below. In contrast, SPSW's web plates work almost entirely in tension, but the beams and columns around the web plate are designed differently. The tension in the web plate acts along the length of the boundary elements, rather than only at the intersection of beams and columns. As a result, large inward forces can be exerted on the boundary elements.

Both HBE and VBE are designed to resist web-plate tension forces acting inward on the SPSW at an angle determined from the frame geometry and member section properties. These inward forces, and the resistance provided by the boundary elements, are fundamental to the understanding of SPSW behavior. The tensile forces in the web plate induce flexure in the VBE, in addition to the axial forces due to overturning of the wall. If the transverse stiffness of the VBE is small, uniform tension cannot be developed across the web plate and the strength of the system is significantly reduced. However, if the transverse stiffness of the VBE is high, web plates can develop their full tension strength at the vertical interfaces with the VBE. The restraint provided by HBE enables the VBE to resist the flexure caused by web-plate tension. HBE typically occur at floor levels, where they also serve as the beams or girders supporting the deck.

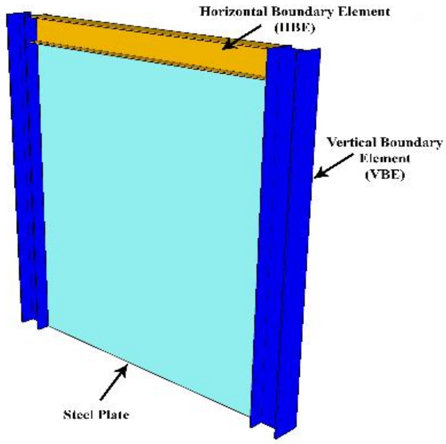


Fig. 4 Typical elements of Un-stiffened SPSW

3.1.2 *Stiffened SPSW*

Essential compression forces can be developed in the web plate by stiffened steel plate shear walls in addition to the tension forces that can be developed in an unstiffened SPSW. As a result, boundary element design does not incorporate such large flexural forces. Walls can be made sufficiently stiff to prevent any inward forces from acting on the boundary elements, and the web plate interfaces can be made specifically for pure shear. A combination of tension-field action and shear buckling can be used in walls that have been less stiffened. When the web-plate slenderness ratio exceeds, shear-buckling strength must be determined, supplemented by tension-field action. Procedures for plate girder design can be applied to stiffened steel plate shear walls. The VBE of steel plate shear walls acts like plate girders, with dimensions and elements being vertical and horizontal. Vertical stiffeners are added to reduce web-plate slenderness, and greater wall strength can be calculated using SPSW procedures. In large-spacing steel plate shear walls, shear buckling is permitted, and horizontal stiffeners are typically added.

Plate-girder procedures do not allow tension-field action in the end panel of the girder due to the stiffener's flexural stiffness. Steel plate shear walls can be strengthened with a resisting beam at the base and a strong beam at the roof, allowing for tension-field action in strength calculations. Plate-girder procedures often underestimate the strength of steel plate shear walls due to the neglect of tension-field action. This underestimation can lead to underestimation of the maximum overturning moments that the wall can resist and axial forces in the columns. Additionally, plate-girder procedures do not account for VBE flexural forces resulting from tension-field action in the web plate, which could result in larger than-calculated axial forces in columns. To ensure conservative design, the web shear strength should be taken as the full expected shear yield stress, and the design of the connection of the web plate to the boundary elements should be based on the expected tension strength of the web plate.

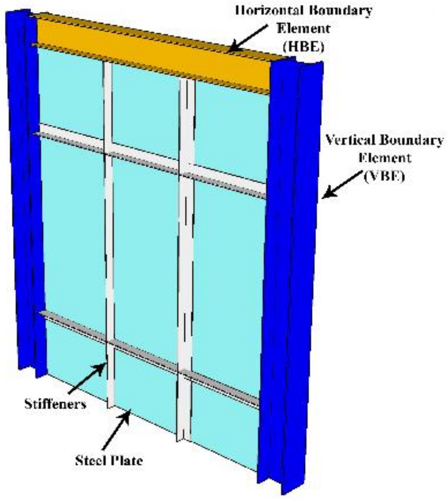


Fig. 5 Typical elements of Stiffened SPSW

3.2 Analysis

Modeling a system serves two purposes: first, to determine forces in system elements for design, and second, to estimate lateral displacement of the frame. For seismic design, forces in HBE and VBE must be determined for the condition with the web plate fully yielded in tension. All elements must have sufficient available strength to resist the forces determined by analysis. Various modelling techniques have been proposed, but this Design steps focuses on two approaches most suitable for practicing structural engineers: strip models, where the web plate is replaced by diagonal tension members, and orthotropic membrane models, which use non isotropic membrane elements to model the compression and tension resistance of the web plate. Orthotropic membrane modelling is used in design examples and is recommended for typical applications when software with this capability is available.

There are three ways to study the analysis of SPSW: (Strip model analysis, Orthotropic membrane model, and Nonlinear analysis) but we are going to describe the strip model analysis only.

3.2.1 Strip Model Analysis

The tension-strip method is a modeling technique used to analyze SPSW assemblies, which is included in Canadian design provisions for SPSW (CSA. 2001) and AISC 341. The method requires a minimum of 10 strips to model the web plate, estimating the effects of distributed load on the frame's boundary elements. Under lateral loads, tension in diagonals results in axial and flexural forces. Figure 5 illustrates a tension-strip model, where the beam's intersections may not align, necessitating multiple segments for accurate tension stress modeling. The length of the beam segments required for n strips (considering only a single web plate) is

$$\Delta \mathrm{x} = \frac{1}{n}\,[L + h\tan(\alpha)] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(1)$$

Δ_x = the length of beam segment between nodes
L = width of panel
h = height of panel
n = number strips

The location of the nodes on the columns must be calculated from the resulting locations of nodes on the beams. The area of the equivalent sips given by

$$As = \frac{[L\cos(\alpha) + h\sin(\alpha)]\, t_w}{n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

A_s = area of a strip

Because of the dependence of strip models on the angle α, they are prone to somewhat tedious alteration of the model. A new method has been suggested a method to simplify the strip-model method by averaging the angle of tension stress over the building's height. This method is most accurate when the bay width and story heights are similar. However, the authors recommend using this approach when the calculated angle of tension stress is within 5° of the average angles. If the angle at a story deviate by more than this, the difference in angles may have a significant effect.

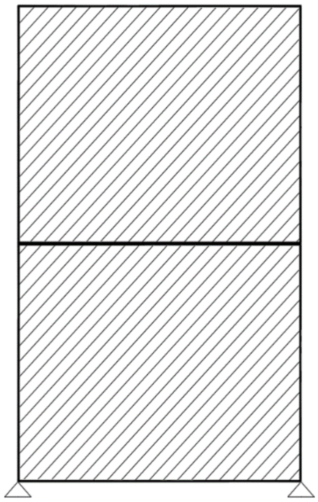


Fig. 6 Typical tension strip model

3.3 General design requirements

This section addresses the basic design of SPSW for strength. As such, it applies to the high-seismic (R>3) of SPSW as well as low-seismic design (R = 3). Proportioning and detailing requirements necessary for system ductility in the seismic of SPSW.

3.3.1 *Preliminary Design*

Web plate, VBE, and HBE preliminary sizes need to be chosen using an equivalent braced frame or based on force distribution. It is necessary to assume the angle of tension stress and ensure that web plates can withstand shear throughout the frame. The necessary thickness of the web plate is computed using an assumed angle (α), which is typically 45°. Under this supposition, the nominal strength of web plates can be computed using AISC 341 Equation.

$$V_u = 0.42\, F_y\, t_w\, L_{cf}\, \sin(2\alpha)$$

The equation predicts nominal strength slightly lower than theoretical strength due to uneven elastic distribution of stress, resulting in a difference between the first significant yield and full yield. The equation can be rearranged to determine the necessary web-plate thickness.

$$t_w \geq \frac{V_u}{\Phi\, 0.42\; F_y\, L_{cf}\, sin(2\alpha)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Vu = the required shear strength
Φ = the resistance factor given in AISC 341 (0.9

Thinner web plates require more fabricator and erector effort, but benefits outweigh the effort. Preliminary selection of VBE based on AISC 341 Section 17.4g stiffness requirements.

$$I_c \geq 0.00307\, \frac{t_{w}\, h^4}{\mathrm{L}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Low-seismic design often relies on the required moment of inertia for VBE stiffness. If this is challenging, an intermediate strut can be introduced to provide web plates with the necessary stiffness. This strut must have sufficient out-of-plane stiffness to prevent web-plate buckling. The angle o should be calculated based on the proportions of individual web plates and panel above and below the strut.

Intermediate struts should not be rigidly connected to the VBE, as it may not be stable. Instead, a connection with rotational flexibility should be considered. SPSW are limited to frames with an aspect ratio between 0.8 and 2.5, which can be increased with intermediate struts.

Web plates with higher aspect ratios have not been thoroughly studied, and the applicability of design recommendations for typical proportions is unclear. The flexibility of long HBE is a concern. Forces imposed by web plates can be derived from the same angle as the selection of the web plate, and the load on the HBE due to lateral loading is the difference in tension effects.

$$w_r = \left[\frac{V}{L_{cf}\tan(\alpha)}\right]_i - \left[\frac{V}{L_{cf}\tan(\alpha)}\right]_{i+1} \dots\dots\dots\dots\dots\dots\dots\dots\dots (5)$$

w_r = the required strength, w_u as a distributed load on the beam due to web plate tension
V= the required strength, Vu
[i] = the effect due to the web plate at level i
[i+1] = the effect due to the web plate at level i+1

This is a seismic load effect and is combined with other loads according to the appropriate load combinations.

$$w_r = \frac{V_{(i)} - V_{(i+1)}}{L_{cf}} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (6)$$

The load on the HBE due to lateral loading of the frame can be simplified to current design requirements. The HBE design, combined with another web plate, should provide adequate stiffness to achieve intended behavior. However, the effectiveness of plate yielding at the desired response level may not be assured and should be verified during the design process.

$$I_{HBE} \geq 0.003\,\frac{(\Delta t_w)\,L^4}{h} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (7)$$

Δt_w = the difference in web plate thickness above and below the HBE

The selection of HBE sections to resist loading from pervious equation, gravity loads, and meet the Stiffness requirement is sufficient for preliminary design. However, research is needed to determine the effect of HBE and VBE flexibility on required stiffness. Nonlinear analysis can demonstrate that required web-plate strength can be achieved within the design story drift with more flexible members. Gravity loading on HBE may cause vertical tension in the web plate, but the shear strength of the web plate is not significantly reduced if the angle of tension stress is not changed by more than a few degrees. For long spans, transverse loading due to web-plate tension may be difficult, and the loading at the bottom HBE is typically more severe. An alternative to reduce the required strength of all HBE is a series of vertical struts at mid-span at every level of the SPSW. Vertical struts should be designed for axial forces corresponding to HBE reactions, using the same fraction of bay length at each level. This equals the upward force required to resist the pull of the bottom web plate, assuming fixed-fixed HBE.

$$P_{(i)} = \sum_{i}^{n} \frac{1}{2} w_{r(i)} L_{cf} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (8)$$

Vertical struts are used for seismic load effects in wide bays, where beam span may prevent economical use of SPSW. They require a strong HBE and VBE for strong-column/weak-beam proportioning. Thorburn et al. (1983) proposed a simplified preliminary design method, involving a tension brace to resist frame story shear.

3.3.2 *Final Design*

Preliminary selections of web plates and boundary elements are made, and a frame model is constructed. Member design forces are obtained for specified lateral loads. For low-seismic design, the designer can use forces from the model for sizing web plates, HBE, and VBE, or design elements assuming a uniform distribution of average stress in the web plate. Connections of web plates to boundary elements are designed based on plate stresses, which cannot exceed the expected yield strength. Orthotropic models report tension stress directly. The effective force (acting at the angle) per unit length on the connection of the web plate to the HBE is.

$$r_{HBE} = \sigma \cos(\alpha)\, t_w \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (9)$$

r = r_u = force per unit length of the connection

The effective force per unit length on the connection of the web plate to the VBE is.

$$r_{VBE} = \sigma \sin(\alpha)\, t_w \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (10)$$

AISC 360 provides an expression for fillet weld strength based on the angle of loading to the longitudinal axis for fillet-welded connections.

$$R_n = F_w A_w \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots. (11)$$

$$F_w = 0.6F_{EXX}[1 + 0.5\sin^{1.5}(\theta)] \dots\dots\dots\dots\dots\dots (12)$$

A_w = the area of the weld
F_{EXX} = the electrode classification number
θ = the angle of loading with respect to the fillet weld axis

To calculate fillet-weld strength per unit length, substitute weld size (w) times √2/2 for weld area (A_w), Resulting in the fillet-weld nominal strength expression.

$$r_n = 0.6F_{EXX}[1 + 0.5\,sin^{1.5}(\theta)]\frac{\sqrt{2}}{2}w \dots\dots\dots\dots\dots.. (13)$$

w = the weld size
for the web-plate connection to the HBE:
$\theta = 90° - \alpha$
for the web-plate connection to the VBE:
$\theta = \alpha$

Thus, the required fillet-weld size for the connection of the web plate to the HBE is.

$$w_{HBE} = \frac{\sigma\cos(\alpha)\, t_w\sqrt{2}}{\Phi 0.6F_{EXX}[1 + 0.5\cos^{1.5}(\alpha)]} \dots\dots\dots\dots\dots\dots\dots (14)$$

The required fillet-weld size for the connection of the web plate to the VBE is.

$$w_{VBE} = \frac{\sigma\sin(\alpha)\, t_w\sqrt{2}}{\Phi 0.6F_{EXX}[1 + 0.5\sin^{1.5}(\alpha)]} \dots\dots\dots\dots\dots\dots\dots\dots (15)$$

Fillet-welded connections between web plates are made in the field to a thicker fish plate, which is then welded in the shop to the VBE and HBE. The diagonal tension force is shared equally between the two welds. Web plates in SPSW can buckle under small loads or their own weight, so either welds must be present on both edges to prevent this. Local boundary elements are required to resist forces from stress in the web plate. Connections between local boundary elements and the HBE and VBE provide more stable hysteretic performance.

JCUMT

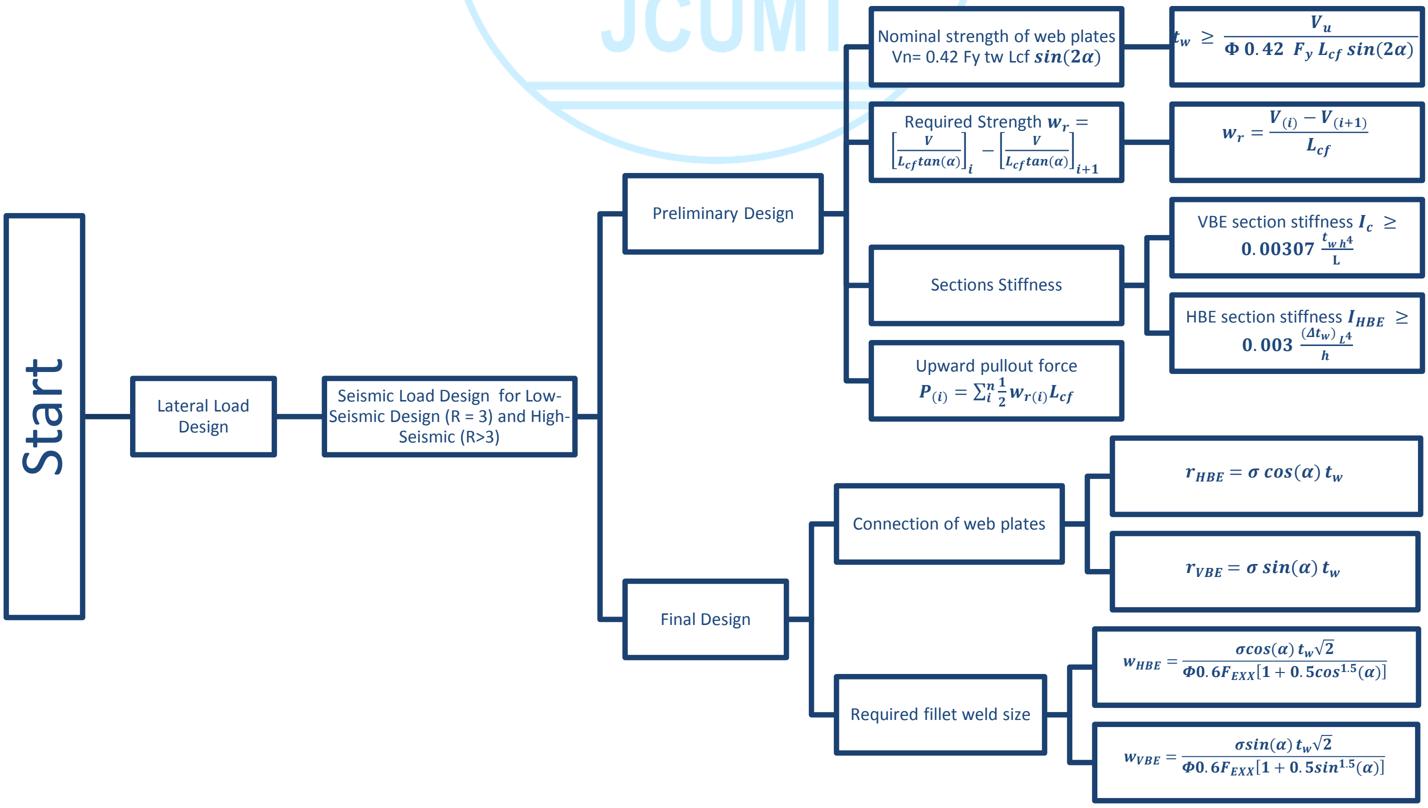


Fig. 7 SPSW design flow chart

4. Finite Element Analysis of Steel Plate Shear Walls

This section discusses the design of a steel plate shear wall system, utilizing analytical methods to drive design forces and estimate displacement in line with building code requirements.

4.1 Model Description

Steel plate shear wall structures consist of edge beams, edge columns, infill panel, beam-to-column connections, and fish plates. The fish plate can be neglected in finite element models, avoiding shear locking. H-shaped frames and infill panels are modeled in ANSYS Workbench R18.1 with a shell element to avoid shear locking. Initial defects are imposed on panels to simulate plate buckling. Residual stress is not considered in finite element modeling. The loading processes is horizontal loads on edge beams to simulate seismic and wind loading.

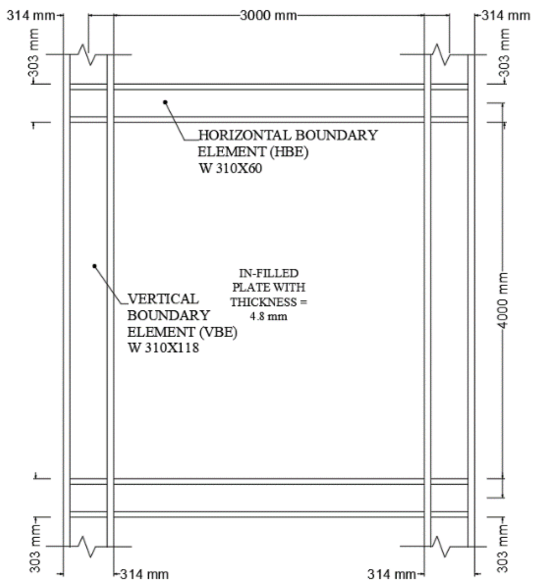
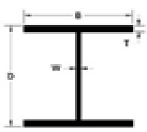
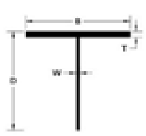


Fig. 8 Specimen details of the studied models

4.2 Material Modeling

The stress-strain curve of steel under cyclic loading differs significantly from that under monotonic loading, making the traditional material constitutive model difficult to calculate accurately. Therefore, a cyclic constitutive model is recommended to accurately simulate cyclic hardening, buckling, cumulative damage, and degradation phenomena in structures subjected to cyclic loading patterns, which is parameterized in ANSYS Workbench R18.1.

4.2.1 *Boundary elements dimensions and properties*

Section Properties					D	B	T	W	BT	HW	Mass	Ar-ea	IX	SX	RX	ZX	IY	SY	RY	ZY
Dim. of Boundary Elements	ary Elements	W 310X60	Length= 4000		303	203	13.1	7.5	7.75	36.9	60	7530	128	842	130	933	18.3	180	49.3	275
	Vertical Boundary Elements (V.B.E.)	W 310X118	Length= 3000		314	307	18.7	11.9	8.21	23.2	118	15000	275	1750	136	1950	90.2	588	77.6	893
Dim. of Stiffener	Vertical and Horizontal stiffeners				100	100	10	10												
					mm	mm	mm	mm			Kg/m	mm^2	mm□	mm^3	mm	mm^3	mm□	mm^3	mm	mm^3

4.2.2 *The Parameter illustration of steel plate shear wall structures with different structural constructions.*

#	Model ID	SPSW Type	No. of Levels	With/ without opening	Opening Position	Lateral load acting
1	U - 01	Un- Stiffened	3	Without	NA	Wind Load
2	U - 02	Un- Stiffened	5	Without	NA	Wind Load
3	U - 03	Un- Stiffened	3	With	Window (1.00x 2.00) Middle	Wind Load
4	U - 04	Un- Stiffened	5	With	Window (1.00x 2.00) Middle	Wind Load
5	U - 05	Un- Stiffened	3	With	Door (1.00x 3.00) Middle	Wind Load
6	U - 06	Un- Stiffened	5	With	Door (1.00x 3.00) Middle	Wind Load
7	U - 07	Un- Stiffened	3	Without	NA	Seismic Load
8	U - 08	Un- Stiffened	5	Without	NA	Seismic Load
9	U - 09	Un- Stiffened	3	With	Window (1.00x 2.00) Middle	Seismic Load
10	U - 10	Un- Stiffened	5	With	Window (1.00x 2.00) Middle	Seismic Load
11	U - 11	Un- Stiffened	3	With	Door (1.00x 3.00) Middle	Seismic Load
12	U - 12	Un- Stiffened	5	With	Door (1.00x 3.00) Middle	Seismic Load
13	U - 14	Un- Stiffened	3	With	Window (1.00x 2.00) Right	Wind Load
14	U - 16	Un- Stiffened	5	With	Window (1.00x 2.00) Right	Wind Load
15	U - 17	Un- Stiffened	3	With	Door (1.00x 3.00) Left	Wind Load
16	U - 18	Un- Stiffened	5	With	Door (1.00x 3.00) Left	Wind Load
17	U - 20	Un- Stiffened	3	With	Window (1.00x 2.00) Right	Seismic Load
18	U - 22	Un- Stiffened	5	With	Window (1.00x 2.00) Right	Seismic Load
19	U - 23	Un- Stiffened	3	With	Door (1.00x 3.00) Left	Seismic Load
20	U - 24	Un- Stiffened	5	With	Door (1.00x 3.00) Left	Seismic Load
21	S - 01	Stiffened	3	Without	NA	Wind Load
22	S - 02	Stiffened	5	Without	NA	Wind Load
23	S - 03	Stiffened	3	With	Window (1.00x 2.00) Middle	Wind Load
24	S - 04	Stiffened	5	With	Window (1.00x 2.00) Middle	Wind Load
25	S - 05	Stiffened	3	With	Door (1.00x 3.00) Middle	Wind Load
26	S - 06	Stiffened	5	With	Door (1.00x 3.00) Middle	Wind Load
27	S - 07	Stiffened	3	Without	NA	Seismic Load
28	S - 08	Stiffened	5	Without	NA	Seismic Load
29	S - 09	Stiffened	3	With	Window (1.00x 2.00) Middle	Seismic Load
30	S - 10	Stiffened	5	With	Window (1.00x 2.00) Middle	Seismic Load
31	S - 11	Stiffened	3	With	Door (1.00x 3.00) Middle	Seismic Load
32	S - 12	Stiffened	5	With	Door (1.00x 3.00) Middle	Seismic Load
33	S - 14	Stiffened	3	With	Window (1.00x 2.00) Right	Wind Load
34	S - 16	Stiffened	5	With	Window (1.00x 2.00) Right	Wind Load
35	S - 17	Stiffened	3	With	Door (1.00x 3.00) Left	Wind Load
36	S - 18	Stiffened	5	With	Door (1.00x 3.00) Left	Wind Load
37	S - 20	Stiffened	3	With	Window (1.00x 2.00) Right	Seismic Load
38	S - 22	Stiffened	5	With	Window (1.00x 2.00) Right	Seismic Load
39	S - 23	Stiffened	3	With	Door (1.00x 3.00) Left	Seismic Load
40	S - 24	Stiffened	5	With	Door (1.00x 3.00) Left	Seismic Load

4.2.3 *Material characteristics and constraint condition*

The edge frame of a building has a yield strength of 380 MPa, while the infill panel has a yield strength of 240 MPa. The specimens are fixed, and connections are restrained to prevent instability. The initial defect value is 1/500 height of the steel plate. The axial compression stress to strength ratio is maintained at 0.2. The inter-story drift angle is defined as $\theta = \Delta/H$, with a maximum displacement of 60 mm to investigate earthquake behavior.

Type	$\sigma\|_0$	Q_∞	b_{iso}	$C_{kin,1}$	γ_1	$C_{kin,2}$	γ_2	$C_{kin,1}$	γ_3	$C_{kin,4}$	γ_4
	(MPa)	(MPa)		(MPa)		(MPa)		(MPa)		(MPa)	
Column and Beam (HBE and VBE)	380	16	1.1	4924	154	3101	120	2730	31	1450	26
Wall (Plate)	240	21	1.2	4924	154	3101	120	2730	31	1450	26

4.3 Computing Platform

The study focuses on the thin steel plate shear wall under cyclic loadings, aiming to simulate strong nonlinear behaviors such as apparent buckling, out-of-plane deformation, and tension strip mutations. ANSYS Workbench R18.1 is used for analyses, which treat the static problem as a dynamic process and uses the central difference method for gradual integration of structural motion equations. The structure density is required, and a loading speed of 0.5 is selected every step. The loading rate is relatively slower for static tests, so it doesn't significantly affect the calculation results.

5. Verification of Numerical Models

5.1 Jin-yu Lu, Lu-nan Yan, Yi Tang, and Heng-hua Wang

The study analyzes the impact of slit parameters on steel slit wall behavior, proposing an equation for lateral bearing capacity considering edge stiffener effects. A simplified analytical model, the "wall-frame analytical model," is presented, allowing nonlinear dynamic and static analysis of structures with steel slit walls. The model's validity is demonstrated using two specimens, and it accurately predicts mutual effects of the bearing wall and frame.

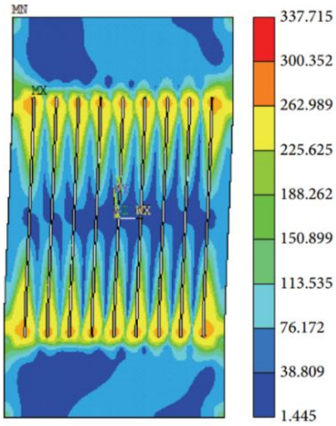


Fig. 9 Von Mises Stresses by Jin-yu Lu, Lu-nan Yan, Yi Tang, and Heng-hua Wang

5.2 Nima Paslar and Alireza Farzampour

This study investigates the impact of infill plate connection with boundary elements on steel plate shear walls' structural performance. Over 21 computational models were established, examining four connection types with different connectivity ratios. Results show column-only connected infill plate shear walls reduce structural loading resisting capacity more than beam-only systems. Systems with partial infill plate connections have similar structural performance, potentially enabling lateral resisting systems.

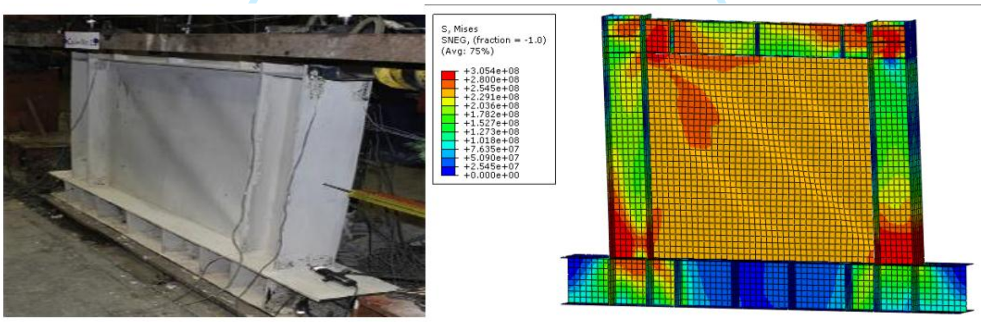


Fig. 10 The verification of Von-Mises stresses with laboratory test by Nima Paslar and Alireza Farzampour

5.3 M.A. Amer, S.S. Safar and B.E Machaly

This study analyzed CR-SPSWs using the finite element method and verified the numerical model with literature results. The results showed that column restrainers reduced column in-ward deflections, reduced base shear, and reduced rigidity requirements for full yielding of in-fill plates. They also accelerated full yielding of in-fill plates, increased in-fill plate thickness and strength without increasing base shear, and slightly increased diagonal tension forces. A mathematical expression for ultimate shear strength was established, and column restrainers should support axial force from horizontal diagonal tension forces on columns.

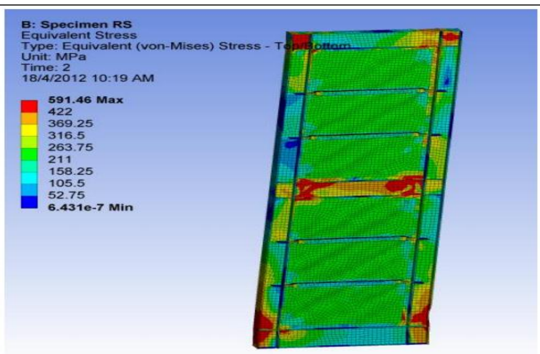


Fig. 11 Von-Mises stresses (MPa) at limit load, Specimen RS by M.A. Amer, S.S. Safar and B.E Machaly

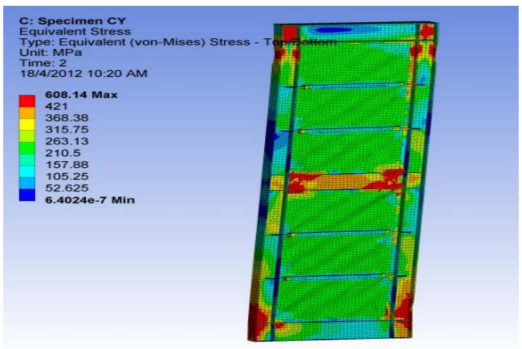


Fig. 12 Von-Mises stresses (MPa) at limit load, Specimen CY by M.A. Amer, S.S. Safar and B.E Machaly

5.4 Serra ZerrinKorrkmaza

The study examines the use of Structural Strengthening Systems (SPSW) on reinforced concrete frames. It found that SPSW systems significantly increase horizontal load-bearing capacity and stiffness, with the highest strength increase in S-Inner and S-Full samples. The study also found that SPSW systems can alter the structure's dynamic properties, avoiding weight increase, and ensuring sufficient shear capacity.

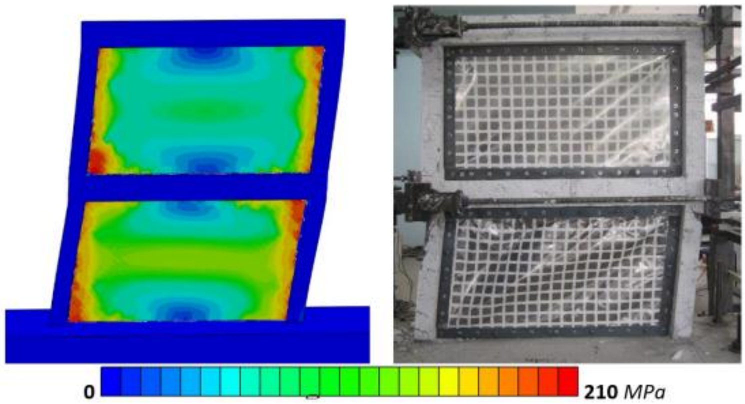


Fig. 13 The Von Mises stresses distribution and deformed shape of SPSW element of the "S-Inner" specimen By Serra ZerrinKorrkmaza

6. Results and Discussion

The dynamic analysis must include the dynamic properties of the structure, and the foundations and the soil bearing it, or in other words, the impact on the building with dynamic forces like a group of earthquakes or wind loads that affect the building in a specific location during a certain period.

6.1 Behavior of different types and Systems of SPSW subjected to seismic load

To study the behavior of seismic load it should depends on taking the impact of the earthquake on the structure as transverse static forces that affect the slab level of each floor and determine the values of these forces using the dynamic properties of the structure.

Response spectrum curve It is a curve that describes the change in the maximum response of buildings or structural elements (displacement, rotation), with the change in the value of its natural frequency because of a specific earthquake or the average for a group of selected earthquakes.

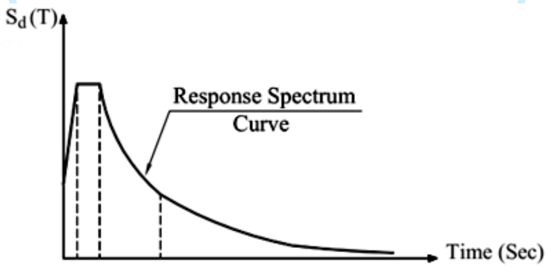


Fig. 14 Response spectrum curve

6.1.1 *Comparison between Total Deformations, Equivalent Stress (Von-Mises), Normal Elastic Strain and Shear Elastic strain*

#	Model Name	Max Total Deformation	Max Equivalent Stress (Von-Mises)	Max Normal Elastic Strain	Max Shear Elastic Strain
	Units	mm	MPa	mm/mm	mm/mm
1	Un- Stiffened SPSW 3 stories without opening	47.234	687.6	0.0024512	0.003647
2	Un- Stiffened SPSW 5 stories without opening	18.396	310.89	0.00074625	0.0010334
3	Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle	56.854	817.17	0.0023728	0.004258
4	Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle	21.781	237.79	0.0007434	0.0012686
5	Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle	71.79	757.11	0.0016226	0.003388
6	Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle	26.636	322.86	0.0010469	0.001676
7	Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right	62.194	796.62	0.0028864	0.0041479
8	Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right	21.618	229.38	0.00076221	0.001231

9	Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left	110.38	874.87	0.0026368	0.004586
10	Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left	30.778	406.26	0.0014079	0.0019969
11	Stiffened SPSW 3 stories without opening	21.695	224.23	0.0010521	0.0074286
12	Stiffened SPSW 5 stories without opening	29.84	229.82	0.0010817	0.0077533
13	Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle	143.89	191.17	0.0099278	0.0088723
14	Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle	106.85	90.892	0.0045374	0.0049081
15	Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle	65.044	87.031	0.0038114	0.0033105
16	Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle	82.957	128.17	0.0032411	0.006652
17	Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right	107.8	171.62	0.0034703	0.0054998
18	Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right	128.68	147.00	0.0057397	0.004842
19	Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left	107.00	81.384	0.0029398	0.0039535
20	Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left	25.304	224.61	0.00065765	0.001741

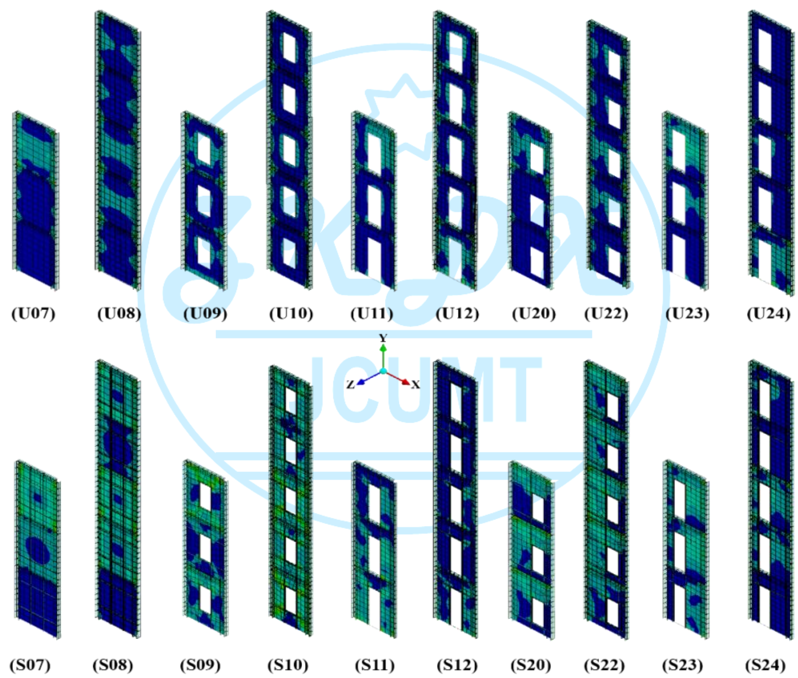


Fig. 15 Equivalent stresses Von-Mises for unstiffened and stiffened SPSWs subjected to seismic load

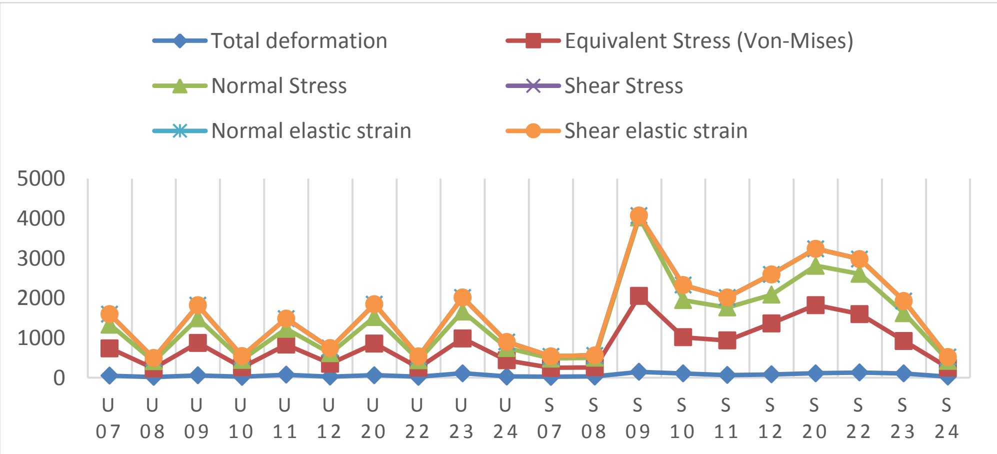


Fig. 16 Comparison graph between Total Deformations, Equivalent Stress (Von-Mises), Normal Elastic Strain and Shear Elastic strain subjected to seismic load.

Based on the response spectrum analysis of both stiffened and unstiffened steel plate shear walls (SPSW), it was found that the unstiffened SPSW had higher stresses without opening than the stiffened SPSW. This indicates that the stiffeners' resistance reduces the Von-Mises equivalent stresses from 687.6 MPa to 224.23 MPa. In addition, the stresses in the unstiffened SPSW five stories without an opening are higher than those in the stiffened SPSW five stories without an opening. This indicates that the stiffeners' absence reduces the Von-Mises equivalent stresses from 310.89 MPa to 229.82 MPa.

Regarding the Un-Stiffened SPSW 3 story models with a window opening (1.00 x 2.00) @middle and Un-Stiffened SPSW 3 story models with a window opening (1.00 x 2.00) @right, the location of the window opening is the only difference between the previous 2 models, and the 2.5% difference in stresses between these 2 models is found. furthermore, concerning the models Un-Stiffened SPSW 5 stories with a window opening (1.00 x 2.00) @middle and Un-Stiffened SPSW 5 stories with a window opening (1.00 x 2.00) Correct, the location of the window opening is the only difference between these two models, and there is a 3.5% difference in stresses between them. and keeping in mind the @middle, Stiffened SPSW 3 stories with a window opening (1.00 x 2.00) The following are the locations of the Un-Stiffened SPSW 5 stories with window openings (1.00 x 2.00) at the @right and @middle. @right, the Von-Mises stresses of the stiffened and unstiffened models are 191.17, 171.62, 90.892, and 147.00, respectively, less than those of the unstiffened models.

Concerning the SPSW with door opening the location of the door opening is the only difference between the two previous models (Un-Stiffened SPSW 3 stories with a door opening (1.00 x 3.00) @middle and Un-Stiffened SPSW 3 stories with a door opening (1.00 x 3.00) @left. The stride difference between these two models is found to be 13.4%. Regarding the Un-Stiffened SPSW 5 story models with a door opening (1.00 x 3.00) @middle and Un-Stiffened SPSW 5 story models with a door opening (1.00 x 3.00) @left, the door opening's location is the only difference between the two previous models, and the difference in stride between these two models is found to be 20.52%. The Von-Misses stresses of the stiffened models are less than those of the un-stiffened models, which equal 87.031, 128.17, 81.384, and 224.61, respectively, after accounting for the following: the stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle, the stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left, the un-stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle, and the un-stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left.

6.1.2 Comparison of time vs. displacement graphs

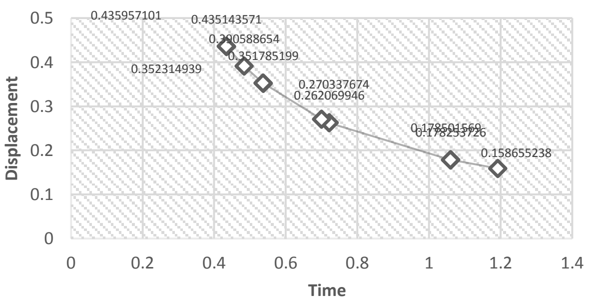


Fig. 17 Un- Stiffened SPSW 3 stories without opening subjected to seismic load

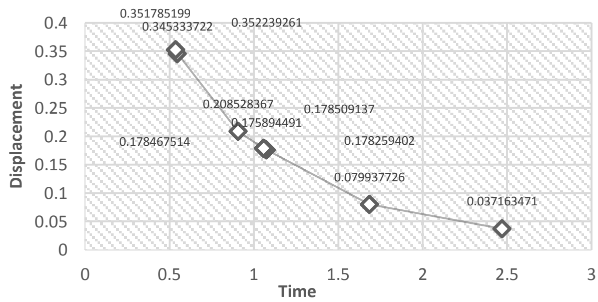


Fig. 18 Un- Stiffened SPSW 5 stories without opening subjected to seismic load

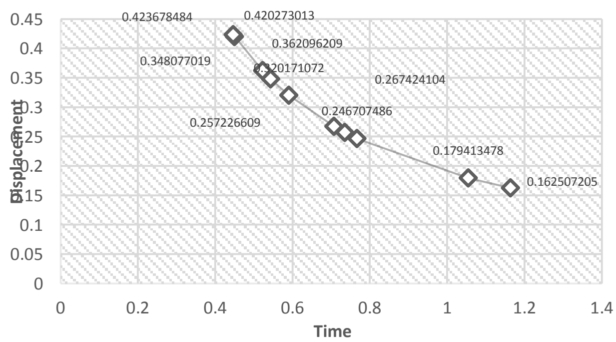


Fig. 19 Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle subjected to seismic load

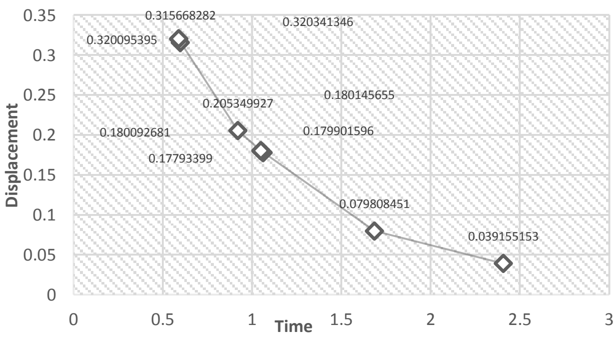


Fig. 20 Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle subjected to seismic load

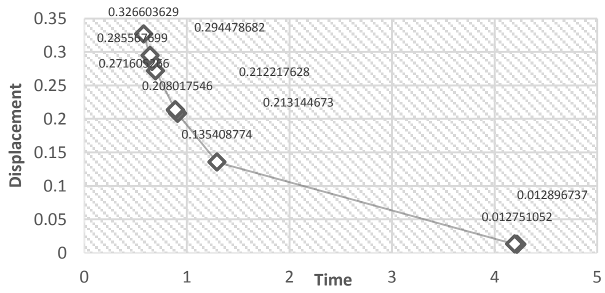


Fig. 21 Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle subjected to seismic load

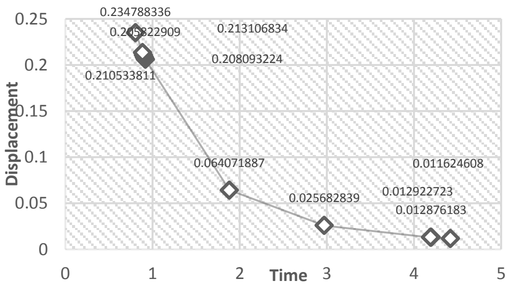


Fig. 22 Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle subjected to seismic load

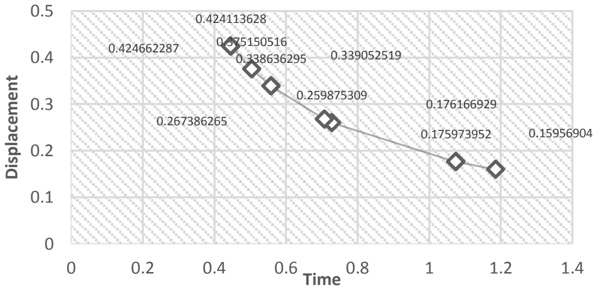


Fig. 23 Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right subjected to seismic load

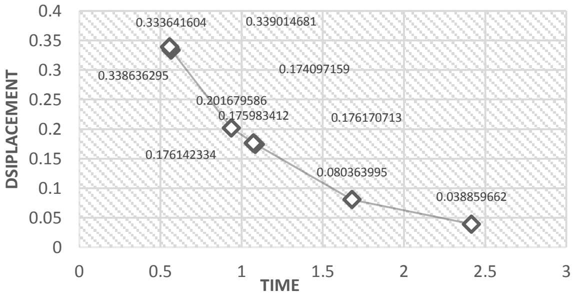


Fig. 24 Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right subjected to seismic load

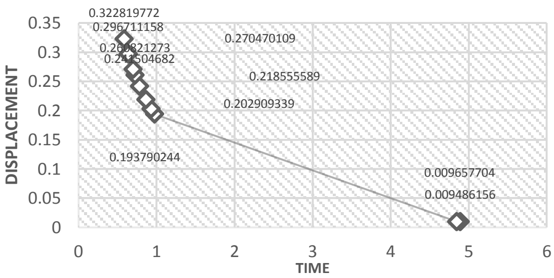


Fig. 25 Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left subjected to seismic load

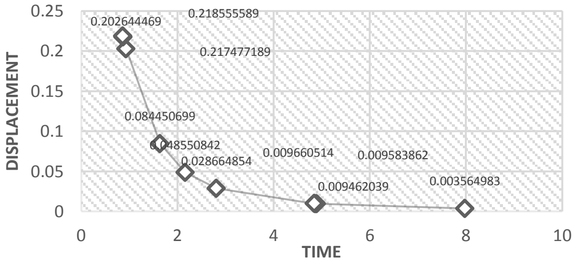


Fig. 26 Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left subjected to seismic load

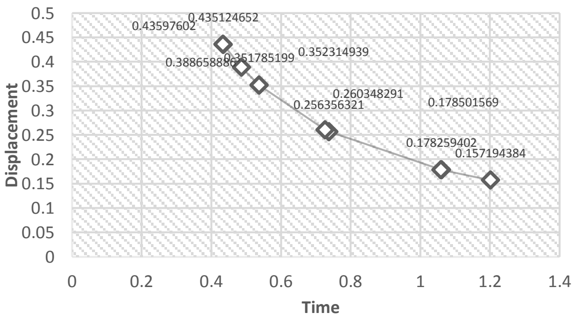


Fig. 27 Stiffened SPSW 3 stories without opening subjected to seismic load

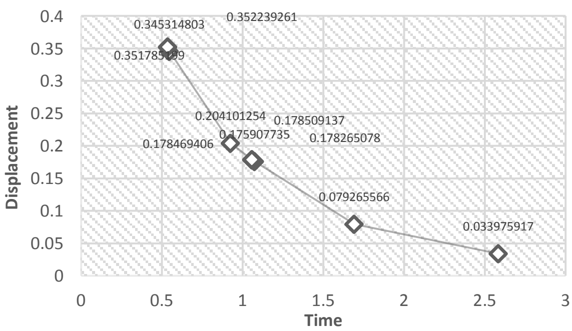


Fig. 28 Stiffened SPSW 5 stories without opening subjected to seismic load

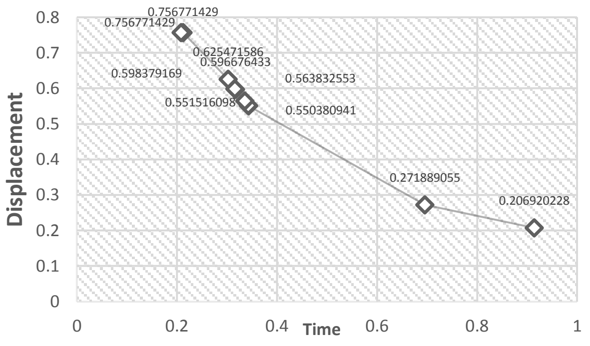


Fig. 29 Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle subjected to seismic load

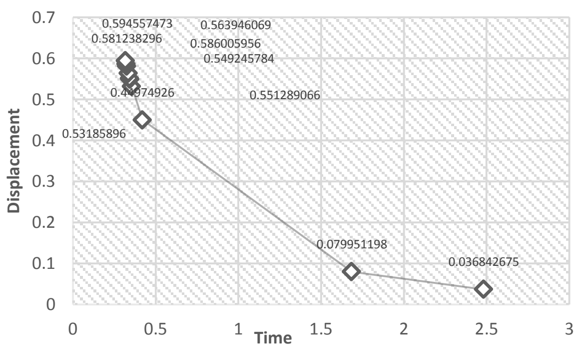


Fig. 30 Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle subjected to seismic load

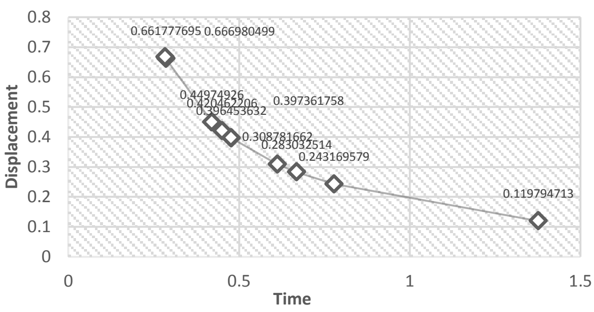


Fig. 31 Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle subjected to seismic load

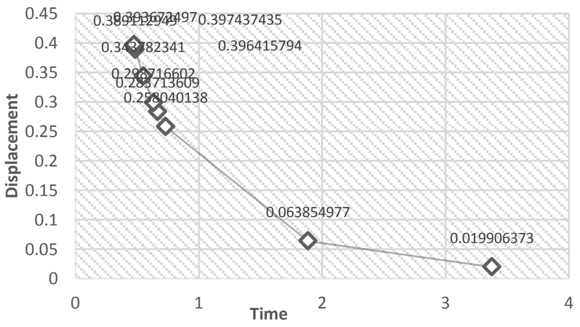


Fig. 32 Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle subjected to seismic load

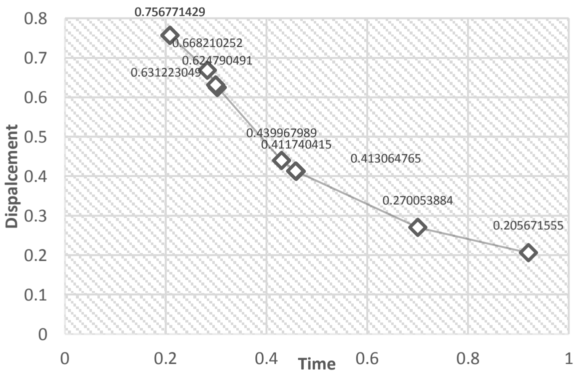


Fig. 33 Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right subjected to seismic load

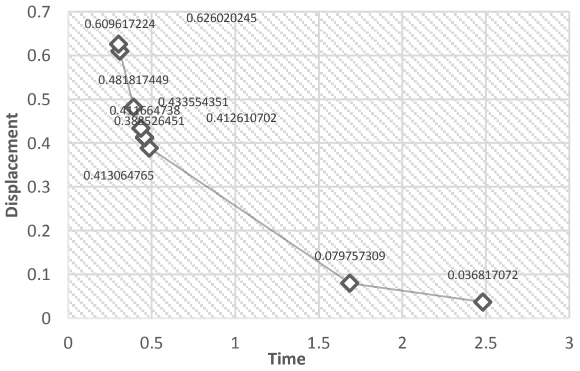


Fig. 34 Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right subjected to seismic load

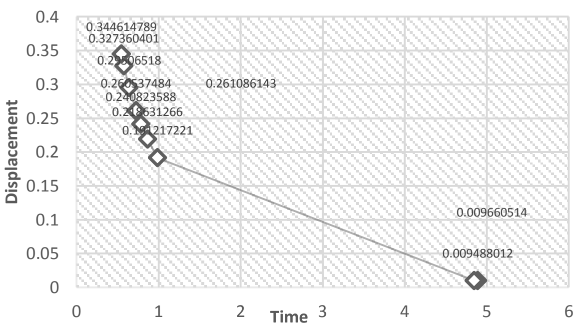


Fig. 35 Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left subjected to seismic load

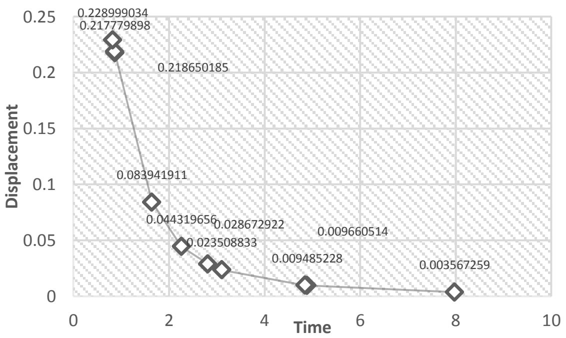


Fig. 36 Stiffened SPSW 5 stories with a door opening(1.00x 3.00) @left subjected to seismic load

6.2 Behavior of different types and Systems of SPSW subjected to wind load

6.2.1 Comparison between Total Deformations, Equivalent Stress (Von-Mises), Normal Elastic Strain and Shear Elastic strain

#	Model ID	Max Total Deformation	Max Equivalent Stress (Von-Mises)	Max Normal elastic strain	Max Shear elastic strain
	Units	mm	MPa	mm/mm	mm/mm
1	Un- Stiffened SPSW 3 stories without opening	78.204	346.56	0.0012566	0.0018512
2	Un- Stiffened SPSW 5 stories without opening	88.086	402.23	0.0012373	0.0015475
3	Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle	62.425	408.01	0.0014191	0.001829
4	Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle	89.402	462.12	0.0013923	0.0016704
5	Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle	22.272	194.54	0.0008	0.00075325
6	Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle	34.582	222.4	0.00071821	0.0010424
7	Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right	39.273	256.18	0.00097413	0.0011368
8	Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right	91.978	456.18	0.0013844	0.0018322
9	Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left	22.651	269.73	0.00054854	0.00090863
10	Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left	41.148	222.14	0.00064876	0.0010065
11	Stiffened SPSW 3 stories without opening	65.364	319.38	0.0010395	0.0014895
12	Stiffened SPSW 5 stories without opening	88.086	403.22	0.0012374	0.0015486
13	Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle	23.56	166.63	0.00069145	0.000747
14	Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle	40.683	204.29	0.00072866	0.00080656
15	Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle	10.144	89.236	0.00038457	0.00035714
16	Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle	32.576	218.49	0.00075593	0.00091037
17	Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right	30.315	220.39	0.00085337	0.001415
18	Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right	30.696	164.48	0.00063113	0.00099383
19	Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left	36.097	245.02	0.00093693	0.0017274
20	Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left	39.133	216.59	0.00070777	0.0013465

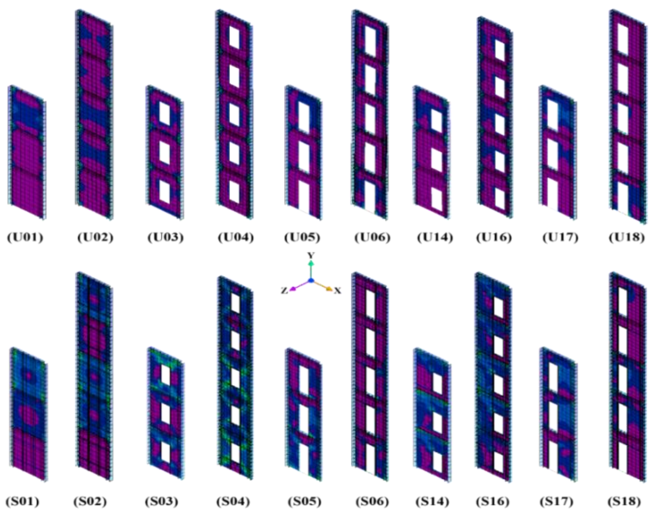


Fig. 37 Equivalent stresses Von-Mises for unstiffened and stiffened SPSWs subjected to wind load

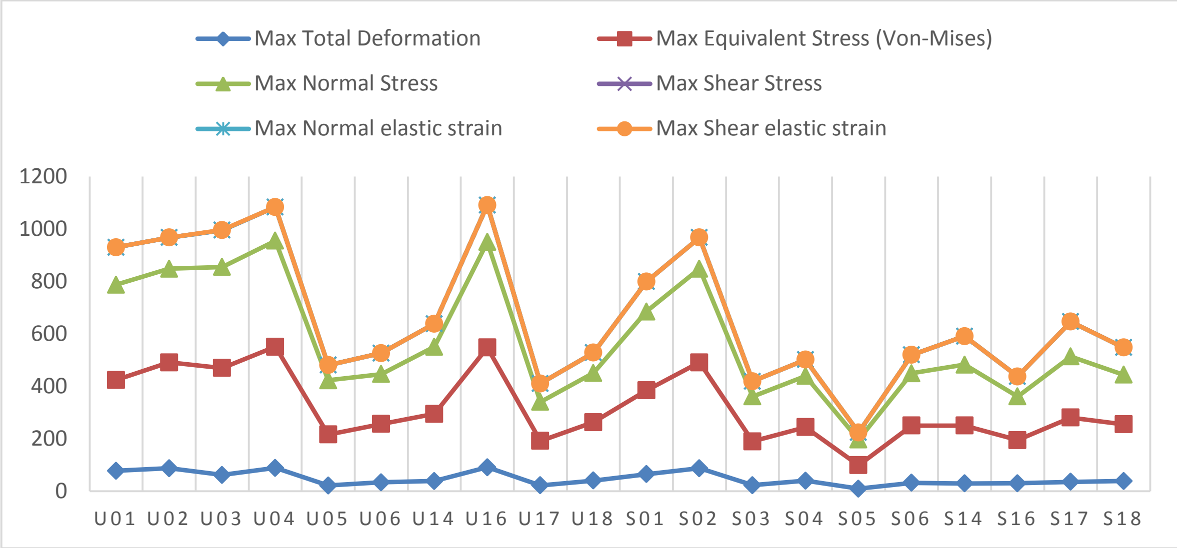


Fig. 38 Comparison graph between Total Deformations, Equivalent Stress (Von-Mises), Normal Elastic Strain and Shear Elastic strain subjected to wind load.

Based on the wind load analysis of both stiffened and unstiffened steel plate shear walls (SPSW), it was found that the unstiffened SPSW had higher stresses without opening than the stiffened SPSW. This indicates that the stiffeners' resistance reduces the Von-Mises equivalent stresses from 346.56 MPa to 319.38 MPa. In addition, the stresses in the unstiffened SPSW five stories without an opening are higher than those in the stiffened SPSW five stories without an opening. This indicates that the stiffeners' absence reduces the Von-Mises equivalent stresses from 402.23 MPa to 403.22 MPa and this a aminor difference.

Regarding the Un-Stiffened SPSW 3 story models with a window opening (1.00 x 2.00) @middle and Un-Stiffened SPSW 3 story models with a window opening (1.00 x 2.00) @right, the location of the window opening is the only difference between the previous 2 models, and the 37.21% difference in stresses between these 2 models is found. furthermore, concerning the models Un-Stiffened SPSW 5 stories with a window opening (1.00 x 2.00) @middle and Un-Stiffened SPSW 5 stories with a window opening (1.00 x 2.00) @ right, the location of the window opening is the only difference between these two models, and there is a 1.285% difference in stresses between them. and keeping in mind the @middle, Stiffened SPSW 3 stories with a window opening (1.00 x 2.00) The following are the locations of the Un-Stiffened SPSW 5 stories with window openings (1.00 x 2.00) at the @right and @middle. @right, the Von-Mises stresses of the stiffened and unstiffened models are 116.63, 204.29, 220.39, and 164.48, respectively, less than those of the unstiffened models.

Concerning the SPSW with door opening the location of the door opening is the only difference between the two following models (Un-Stiffened SPSW 3 stories with a door opening (1.00 x 3.00) @middle and Un-Stiffened SPSW 3 stories with a door opening (1.00 x 3.00) @left. The stride difference between these two models is found to be 27.87%. Regarding the Un-Stiffened SPSW 5 story models with a door opening (1.00 x 3.00) @middle and Un-Stiffened SPSW 5 story models with a door opening (1.00 x 3.00) @left, the door opening's location is the only difference between the two previous models, and the difference in stride between these two models is found to be 0.12%. The Von-Misses stresses of the stiffened models are less than those of the un-stiffened models, which equal 89.236, 218.49, 245.02, and 216.59, respectively, after accounting for the following: the stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle, the stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left, the un-stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle, and the un-stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left.

6.2.1 Comparison of frequency and amplitude

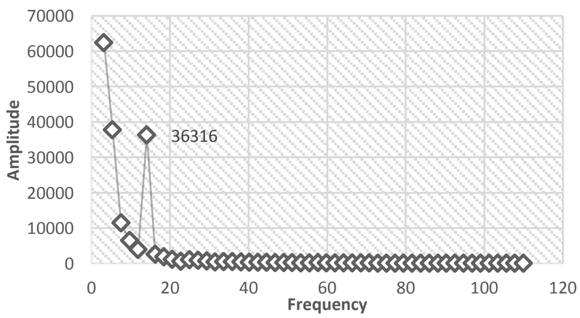


Fig. 39 Un- Stiffened SPSW 3 stories without opening subjected to wind load.

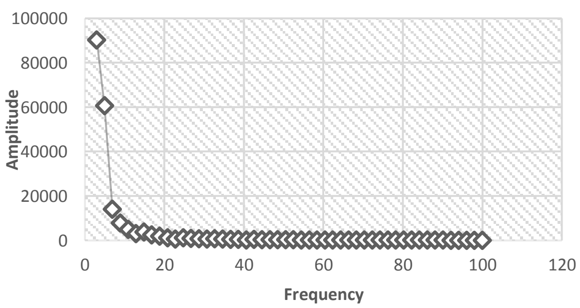


Fig. 40 Un- Stiffened SPSW 5 stories without opening subjected to wind load.

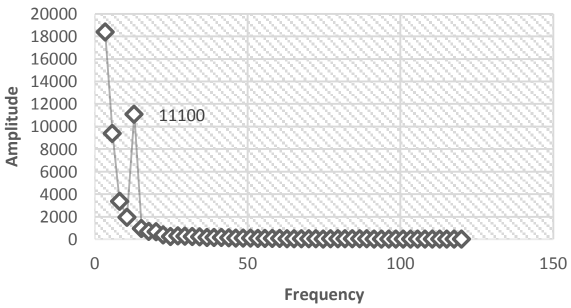


Fig. 41 Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle subjected to wind load

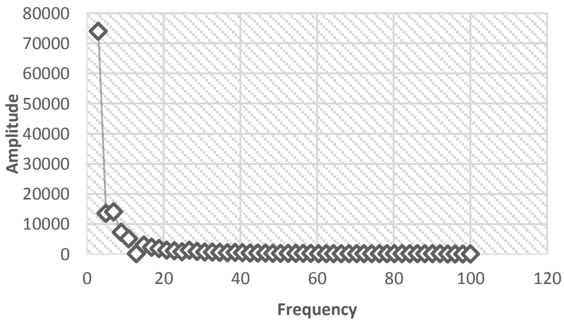


Fig. 42 Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle subjected to wind load

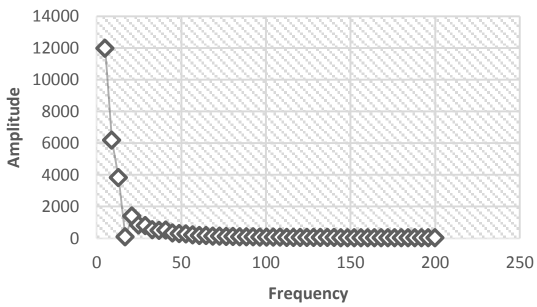


Fig. 43 Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle subjected to wind load

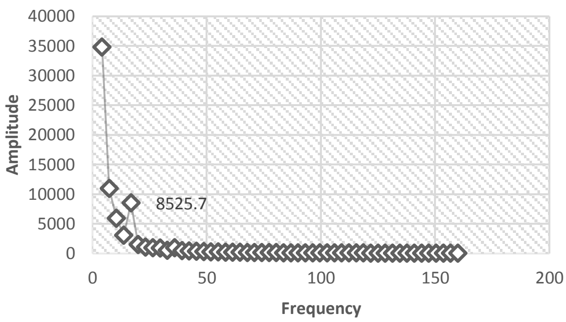


Fig. 44 Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle subjected to wind load

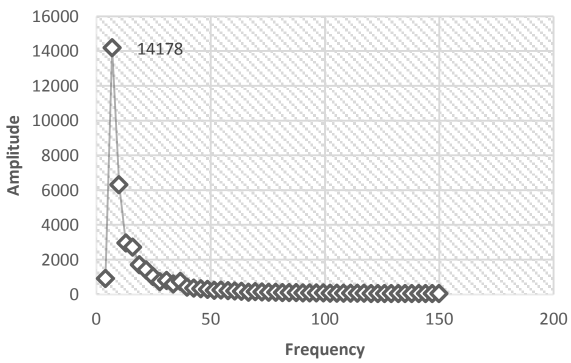


Fig. 45 Un- Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right subjected to wind load

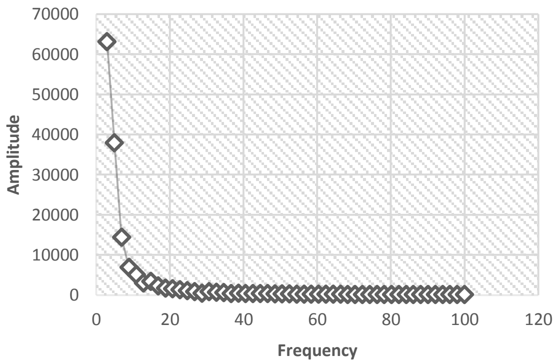


Fig. 46 Un- Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right subjected to wind load

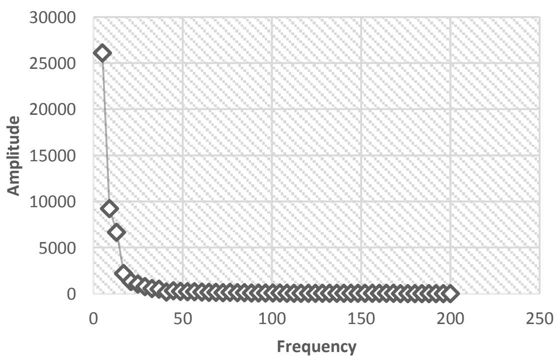


Fig. 47 Un- Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left subjected to wind load

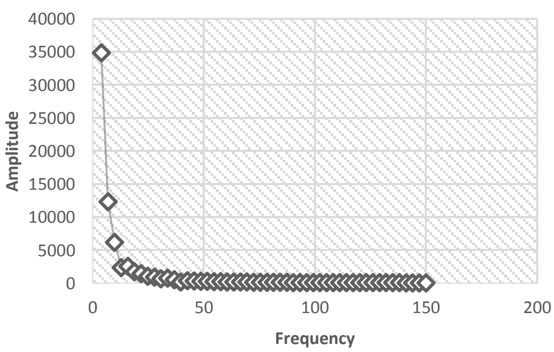


Fig. 48 Un- Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left subjected to wind load

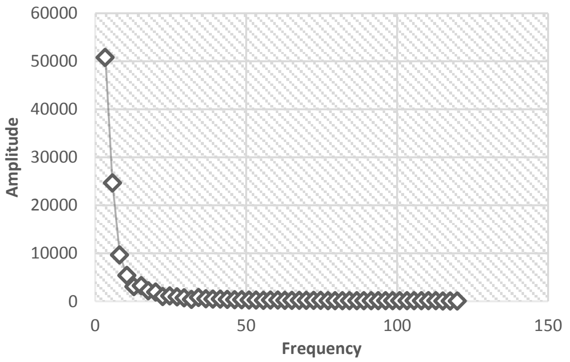


Fig. 49 Stiffened SPSW 3 stories without opening subjected to wind load

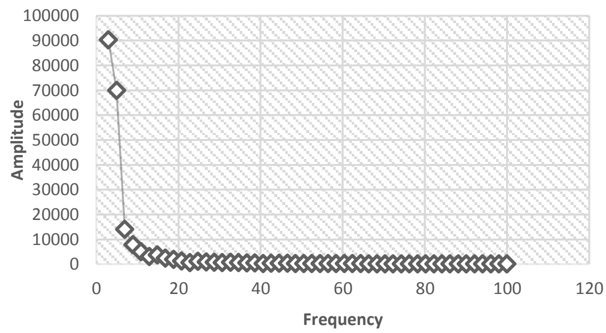


Figure 50 Stiffened SPSW 5 stories without opening subjected to wind load

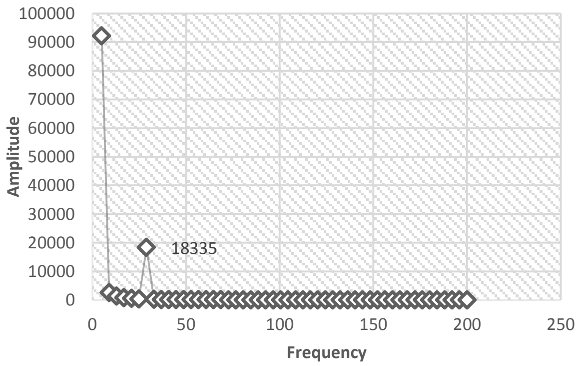


Fig. 51 Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @middle subjected to wind load

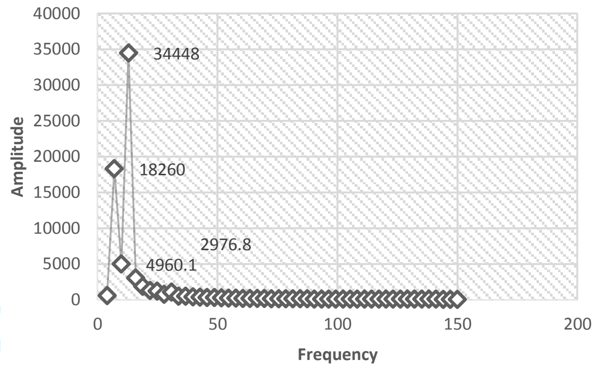


Fig. 52 Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @middle subjected to wind load

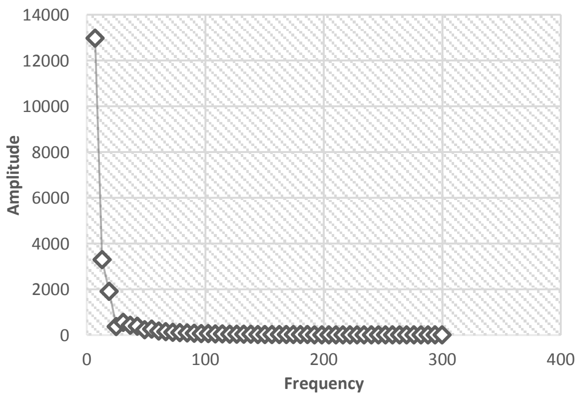


Fig. 53 Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @middle subjected to wind load

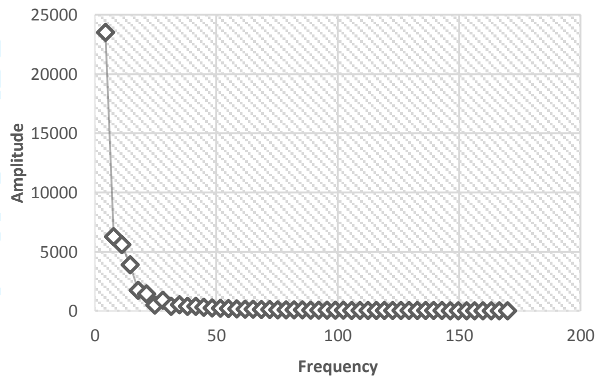


Fig. 54 Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @middle subjected to wind load

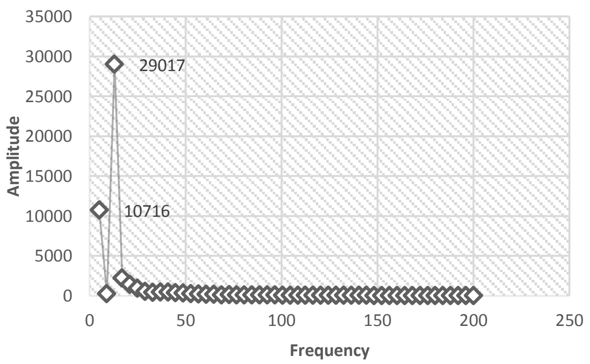


Fig. 55 Stiffened SPSW 3 stories with a window opening (1.00x 2.00) @right subjected to wind load

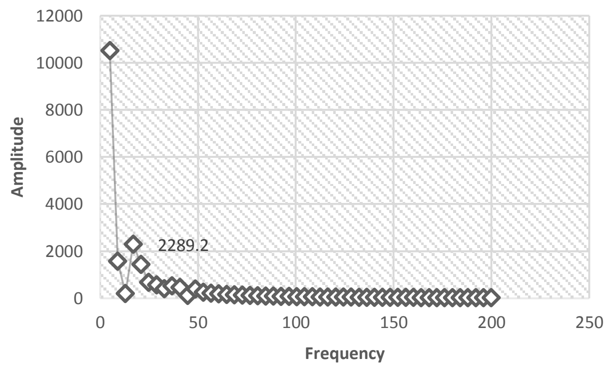


Fig. 56 Stiffened SPSW 5 stories with a window opening (1.00x 2.00) @right subjected to wind load

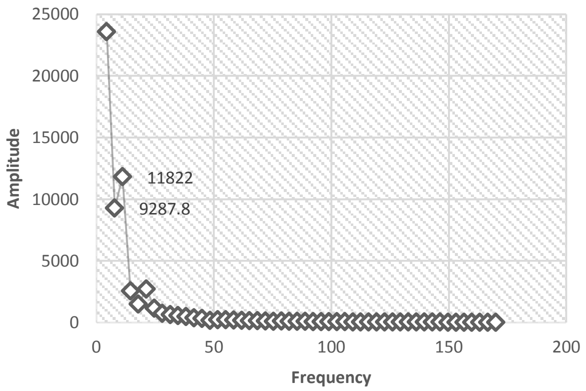


Fig. 57 Stiffened SPSW 3 stories with a door opening (1.00x 3.00) @left subjected to wind load

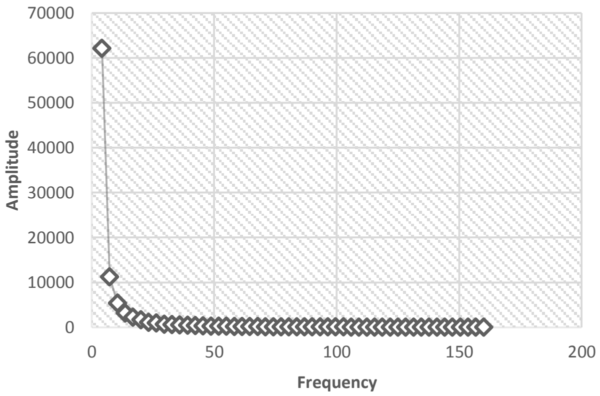


Fig. 58 Stiffened SPSW 5 stories with a door opening (1.00x 3.00) @left subjected to wind load

As can be seen from the earlier charts and figures, the SPSWs without any openings decrease in amplitude with increasing frequency until they reach zero. Regarding the SPSWs with openings (windows, doors). we can see that these openings alter the frequency amplitude charts, as shown in figures (40, 41, 47, 48, 51, 52, 53, etc.). And if we pay close attention to figure 48, we will see that the amplitude increases as the frequency increases until it reaches the value 18260. After opening, it then decreases to 4960.1, increases once more to 34448, decreases once more to 2976.8, and then decreases further to a value that is nearly constant.

7. Conclusion

Forty numerical models of various SPSW types—both stiffened and unstiffened—as well as closed and opened (window and doors) with a different location, SPSWs models with various lateral loads—both seismic and wind—are examined. The results of the numerical analysis are summed up as follows:

a) The study presents a promising lateral force-resisting structural system called the Stiffened Steel Plate Shear Wall (SPSW), which can be divided into stiffened and unstiffened SPSW based on the presence of stiffeners.
b) The finite element method proposed can accurately predict the behaviors of steel plate shear walls under lateral loads, including total deformations, normal and shear stresses, normal and shear strains, frequency versus displacements and amplitudes, and the application of initial defects.
c) Seismic loads are distributed better over the larger number of floors within the limits allowed by the code because the load is distributed over a larger area and therefore the stresses on the floors are less.
d) The presence of stiffeners that have a window openning reduces stress on SPSWs compared to SPSWs without stiifeners, and it is preferable for the window openning to be in the middle of the SPSW, as they are better in stresses distribution in seismic load and wind load in minor values.
e) The presence of stiffeners that have a door openning reduces stress on SPSWs compared to SPSWs without stiifeners, and it is preferable for the door openning to be in the middle of the SPSW, as they are better in stresses distribution in seismic load and wind load.
f) The method also demonstrates the rationality of selected element types and constitutive models, making it a valuable tool for studying the performance of steel plate shear walls. The stiffened SPSW is an efficient and economical lateral force-resisting system in high-rise steel structure buildings.
g) Using stiffeners increases the sections' ability to resist lateral loads.
h) SPSW can resist both seismic load and wind load efficiently in both directions.

References

1. Abhishek Verma and Dipti Ranjan Sahoo. Seismic behaviour of steel plate shear wall systems with staggered web configurations. Earthquake Engng Struct Dyn. 2018;47:660–677
2. Adriansyah Bagus Aryanto, Aditya Rio Prabowo*, Teguh Muttaqie, Nurul Muhayat, Tuswan Tuswan, Nurul Huda, and Quang Thang Do. Investigation of the ability of steel plate shear walls against designed cyclic loadings: Benchmarking and parametric study. Journal of the Mechanical Behavior of Materials 2023; 32: 20220301
3. Akbar Vasseghi. A simple model for nonlinear analysis of steel plate shear wall structural systems. Struct Design Tall Spec Build. 2021;30:e1828. https://doi.org/10.1002/tal.1828
4. Cao Z, Wang Z, Du P, Liu H, Fan F. Quasi-static experiments on steel plate shear walls reinforced with X-shaped restrainers. J Build Eng. 2020;31:101451.
5. Chen SJ, Jhang C. Cyclic behavior of low yield point steel shear walls. Thin-Walled Struct 2006;44(7):730–8.
6. Dr. B P Annapoorna, Mohammed Ali Boodihal. A Study on Steel Plate Shear Wall. IOP Conf. Series: Materials Science and Engineering 577 (2019) 012173.IOP Publishing. doi:10.1088/1757-899X/577/1/012173

7. H. Bakhshi, H. Khosravi and M. Ghoddusi. Evaluation of Seismic Behavior of Steel Shear Wall by Time History Analysis. Computational Engineering and Physical Modeling 2-1 (2019) 38-55
8. Hamza Basri1, Abdelouahab Ras, Karin Hamdaoui and Almoutaz Bellah Alsamawi. Numerical investigation on seismic behavior of steel frame equipped with steel plate shear wall. INTERNATIONAL JOURNAL OF ADVANCED SCIENTIFIC RESEARCH AND INNOVATION. ISSN: 2785-9541 VOLUME 5, ISSUE 1, 2022, 54 – 66
9. Jianian He, Xuhao Li, Shizhe Chen and Huasheng Xian. Topology Optimization of Stiffened Steel Plate Shear Wall Based on the Bidirectional Progressive Structural Optimization Method. Buildings 2023, 13, 831. https://doi.org/10.3390/buildings13030831
10. Jin-yu Lu, Lu-nan Yan, Yi Tang, and Heng-hua Wang. Study on Seismic Performance of a Stiffened Steel Plate Shear Wall with Slits. Hindawi Publishing Corporation Shock and Vibration Volume 2015, Article ID 689373, http://dx.doi.org/10.1155/2015/689373
11. Leila Hosseinzadeh, Denise-Penelope N. Kontoni and Behnam Babaei. Investigation of the Behavior of Steel Plate Shear Walls Considering Double Corrugated Low-Yield-Point Steel Infill Plate. International Journal of Civil Engineering (2023) 21:1631–1642 https://doi.org/10.1007/s40999-023-00855-z
12. Li F, Li H, Li ZM, Li ZJ, Chen XF, Ding L. Cyclic test of diagonally stiffened steel plate shear wall. J Xian Univ Architect Technol (Nat Sci Ed) 2009;41(1):57–62.
13. Li F. Experimental and theoretical investigation to earthquake resistant behavior of steel plate shear wall. [Doctoral dissertation] Xian, China: Xian University of Architecture and Technology; 2011.
14. Lu J, Zhang H, Yu S. Study on seismic behaviors of self-centering steel plate shear walls with slits. J Constr Steel Res. 2021;185:106878.
15. M.A. Amer, S.S. Safar, B.E Machaly. Analysis and design of steel plate shear walls with column restrainers. Proceedings of the Annual Stability Conference Structural Stability Research Council Toronto, Canada, March 25-28, 2014
16. Mark Sarkisian, Neville John Mathias, Eric Long and Zhihui Huang. Steel Plate Shear Walls, Efficient Structural Solution for slender High Rise in China. Seismic Engineering International Conference 2008.
17. Meisam Safari Gorji and J.J. Roger Cheng. Steel plate shear walls with outriggers. Part II: Seismic design and performance. Journal of Constructional Steel Research. 137 (2017) 311-324
18. Mojtaba Gorji Azandariani, Ali Mohammad Rousta, Masoud Mohammadi, Maria Rashidi, Hamid Abdolmaleki. Numerical and analytical study of ultimate capacity of steel plate shear walls with partial plate-column connection (SPSW-PC). Structures 33 (2021) 3066-3080
19. Nima Paslar , Alireza Farzampour. Investigation of the beams and columns connection with infill plate on the structural behavior of the steel plate shear walls. International Conference on Science & Engineering, University of Tokyo, Japan, October 14,15, 2019
20. Omid Haddad, N. H. Ramli Sulong, Zainah Ibrahim. Cyclic performance of stiffened steel plate shear walls with various configurations of stiffeners. JVE INTERNATIONAL LTD. JOURNAL OF VIBROENGINEERING. FEB 2018, VOL. 20, ISSUE 1. ISSN 1392-8716
21. P. Ebadi and S. Farajloomanesh. Seismic design philosophy of special steel plate shear walls. Magazine of Civil Engineering. 2020. 95(3).
22. Rafi Ramdhani Aziz and Irpan Hidayat. The effect of retrofitting high-rise buildings with steel plate shear walls. E3S Web of Conferences 426, 01028 (2023) https://doi.org/10.1051/e3sconf/202342601028 ICOBAR 2023.
23. Serra ZerrinKorrkmaz. An Analytical Study About the Use of Steel Plate Shear Walls to Improve Lateral Rigidity of Reinforced Concrete Framed Structure. Latin American Journal of Solids and Structures. ISSN 1679-7825.
24. Shi Y, Gu Y, Xu Y, Li H, Yang X. Seismic behavior of steel-sheathed cold-formed steel shear walls with reinforced end columns. J Constr Steel Res. 2022;197:107509.
25. Shi Y, Luo Z, Xu Y, Zou Y, Xu L, Ma Q. Experimental study on the seismic behavior of high-performance cold-formed steel plate shear walls. Eng Struct. 2022;251:113552.
26. V. Broujerdiana, R. Pahnabia and A. Ghamari. Semi-supported steel plate shear wall with oblique sides. Scientia Iranica A (2023) 30(3), 891{901
27. Vian D, Bruneau M, Tsai KC, Lin YC. Special perforated steel plate shear walls with reduced beam section anchor beams. I: experimental investigation. J Struct Eng 2009;135(3):211–20
28. Wang C, Qian H, Zhang Z, Yang Z, Wang Y. Experimental and numerical study on seismic performance of a novel type of coldformed steel shear wall with built-in sandwich panels. Structures. 2022;44:1105–26.
29. Wang W, Quan CC, Li Y, Zhen GK, Zhao HT. Experimental study, and numerical simulation analysis on seismic performance of corrugated steel-plate shear wall with replaceable bottom corner dampers. Soil Dyn Earthquake Eng. 2022;152:107061.
30. Xiaoming Ma, Yi Hu, Xinyuan Cheng, Liqiang Jiang, Yun Li and Hong Zheng. Seismic Performance of Flat Steel Plate Shear Walls with Atmospheric Corrosion. Buildings 2022, 12, 1881. https://doi.org/10.3390/buildings12111881

31. Xu Jian. Research on the behavior and design methods of unstiffened thin steel plate shear wall. [Doctoral dissertation] Chongqing, China: Chongqing University; 2012.
32. Yang Lv, Ling Li, Di Wu, Bo Zhong, Yu Chen and Nawawi Chouw. Experimental Investigation of Steel Plate Shear Walls under Shear-Compression Interaction. Hindawi Shock and Vibration Volume 2019, Article ID 8202780, https://doi.org/10.1155/2019/8202780
33. Yigit Ozcelik and Patricia M. Clayton. Seismic design and performance of SPSWs with beam-connected web plates. Journal of Constructional Steel Research 142 (2018) 55-67
34. Yigit Ozcelik and Patricia M. Clayton. Strip model for steel plate shear walls with beam-connected web plates. Engineering Structures. 136 (2017) 369-379
35. Yuben Zhang, Xun Zhan. Study on Seismic Behavior of Steel Frame-Steel Shear Wall with Assembled Two-Side Connections. Hindawi Mathematical Problems in Engineering Volume 2019, Article ID 3024912, https://doi.org/10.1155/2019/3024912
36. Yuqing Yang, Zaigen Mu and Boli Zhu. Study on Shear Strength of Partially Connected Steel Plate Shear Wall. Metals 2022, 12, 1060. https://doi.org/10.3390/met12071060
37. Zhang X, Zhang E, Zhang Y. Study on shear performance of cold formed thin-walled steel walls sheathed by paper straw board. Eng Struct. 2021;245:112873.
38. Zhang Z, Singh A, Derveni F, Torabian S, Peterman KD, Hutchinson TC, et al. Cyclic experiments on isolated steel sheet connections for CFS framed steel sheet sheathed shear walls with new configurations. Eng Struct. 2021;244:112805

ZKDX
JCUMT