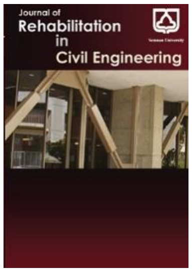
Journal of Rehabilitation in Civil Engineering 11-2 (2023) 94-112



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Numerical Analysis of J-Hook Connectors' Effect on the Performance of Steel-Concrete Composite Shear Walls

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ARTICLE INFO

Article history:
Received: 30 March 2021
Revised: 16 July 2022
Accepted: 21 September 2022

Keywords:
Composite shear wall;
FEA method;
J-hook connector;
Numerical analysis;
Push-over analysis.

ABSTRACT

The use of shear walls is one of the diverse approaches to deal with lateral forces, and composite shear walls are among the different types of these walls. Composite walls consist of two steel sheets and a concrete core between them joined by shear connectors. In this system, the concrete cover can also participate in the load-bearing of the section. Shear connectors are used for bonding concrete to the steel sheet in the wall. Due to the necessity of creating a composite functionality, these connectors play an important role in the behavior of the system. Moreover, the effect of J-hook connectors on steel-concrete composite shear walls is investigated. For this aim, an experimental model is simulated and validated in the ABAQUS software. After verifying the accuracy of the model, a parametric analysis is defined and further studies are performed by using a nonlinear in-crescent static method (pushover method). The results of this study show that the J-Hook connector positively affects increasing load capacity and reducing the out-of-plane displacement of the composite shear wall. Additionally, the number and location of the connectors have a great impact on the both load and buckling capacity of the steel plate. Above all, adding concrete to the steel shear wall which consists of two steel sheets, not only rise the wall's bearing capacity by 14 percent, but improve the performance of the interaction between materials by about 17%.

How to cite this article:
Farnam, S. M., & Khorshidi, M. (2023). Numerical Analysis of J-Hook Connectors' Effect on the Performance of Steel-Concrete Composite Shear Walls. Journal of Rehabilitation in Civil Engineering, 11(2), 94-112.
https://doi.org/10.22075/JRCE.2022.23015.1494

1. Introduction

Composite structures that are used in civil engineering are often a composition of concrete and steel. Concrete is a material that has not only efficient resistance under pressure but also lower production costs than steel. In contrast, steel has several times tensile strength, stiffness, and strength of concrete. Therefore, a proper combination of these materials can make the design more economical than conventional steel or reinforced concrete systems. The noble performance of these materials have led to the increasing use of composite systems; consequently, a great deal of study on these systems is being done in research centers.

The use of composite beams and slabs has been prevalent for over half a century, and in the last two decades, the use of lateral load-resistant composite systems has been the focus of attention by structural designers. These systems include concrete-filled steel tubes (CFT), steel-reinforced concrete columns with embedded steel sections (SRC), composite braces, and various systems, including steel-reinforced concrete walls (S-RCW) and concrete steel plate walls (C-SPW). Moreover, common use of composite systems which is favored by designers is in the field of structural retrofitting, and in many cases, the use of composite systems has considerably helped to reinforce existing structures.

Shear joints are used to bond steel members of the composite shear wall to the concrete. In addition, cast concrete in situ, shear plates and studs are usually used. Bolts can be used to attach the prefabricated concrete walls in many projects. Some Researches have shown that connectors in this type of wall are not only exposed to shear loads but are sometimes subjected to considerable tensile stresses due to the local buckling of the steel sheet. In this study, J-hook connectors were used, which are attached to steel sheets on both sides of the wall and buried inside the concrete. These hook-shaped connectors are completely locked together so that the two metal plates are connected and to the concrete with good integration between them.

Zhao and Astaneh-Asl at the University of California Berkeley conducted tests on composite shear walls where the behavior of a new type of wall under cyclic loads was evaluated and compared with conventional composite shear walls [1]. Observing the results of this experiment, they found that the use of notch reduces the overall resistance and stiffness, but this decrease is acceptable and is less significant than the increase in ductility and concrete damage caused by the notch.

Rahaei and Hatami released a study of the behavior of the composite shear wall with notch under cyclic loads [2]. They concluded by numerical and experimental studies that increasing the distance between the connectors would, to a certain extent, increase the system energy absorption and decrease off-plane displacement and maximum normal shear stress. Furthermore, the results also showed that greater distances do not have much effect on the amount of stress. It was also shown in this study that the rigidity of the mid-beam and the joint of the beam to the column have no significant effect on the behavior of the composite shear wall.

Xiaowei et al. analyzed a nonlinear composite shear wall using the FEA method [3]. The results of this investigation showed that the maximum slip in the shear wall occurs in the lower tensile field of the plate where the concrete is cracked. These results seem to be very useful for the design of high-rise structures.

Huang and Liew studied the structural behavior of concrete and steel composite walls under external pressure and bending moment [4]. In this paper, a new form of a connector, called J-hook, is introduced and used in a composite shear wall. In this study, the axial force interaction diagram vs. the bending moment for the wall is plotted.

In an experimental study, Arabzadeh et al. investigated the behavior of a composite shear wall under shear cyclic loading [5]. The results show that the samples with strong columns have better ductility and energy damping. The study has shown that in addition to the influence of columns, the behavior of the panel also depends on the ratio of the distance of the bolt to steel sheet thickness.

Nie et al. investigated the effective stiffness of a steel plate-filled composite shear wall. This project focuses on the effective stiffness of composite shear walls [6]. The formula for calculating the shear stiffness of the composite shear wall can be obtained based on this model. The overall effective stiffness is obtained by combining the shear and the effective bending stiffness in which the bending stiffness is calculated by the fiber model. The effective stiffness predictions are consistent with the results obtained from laboratory tests.

Dey and Bhowmick investigated the seismic performance of a composite shear wall and the nonlinear seismic response of a 4 and 6-story composite shear wall was studied [7]. The results showed that using composite shear walls in areas with high seismic risk improves structural performance. The results indicate that the design of side columns based on axial load and bending moment is in good agreement with the results obtained from FEA. A series of composite shear walls with different geometries were designed and analyzed for evaluating periodic formulas in building codes. The results displayed that the periods provided by the codes are much shorter than the periods calculated by the FEA method.

Hatami and Sehri studied the variations of steel sheet thickness on the behavior of composite shear walls and the comparison of bearing capacity and ductility behavior of composite steel sheets with the FEA method [8]. They concluded that the variation of the connectors' distance causes the structure ductility and energy absorption.

Farzam and Hoseinzadeh studied the in-plane shear behavior of composite steel-concrete shear walls by numerical analysis of different models [9]. They concluded that increasing the spacing between shear studs doesn't have a major effect on the slope of the force-deformation curve in the elastic region, while the slope of the curve in the post-cracking region is significantly reduced.

Kheyroddin and Hajforoush studied the behavior of CSRCWs with different types of steel and concrete materials numerically [10]. Based on the numerical results, increasing the compressive strength of concrete materials and the yield stress of steel materials, the composite shear walls were rehabilitated better.

Meghdadian and Ghalehnovi investigated the effects of the opening on the behavior of composite steel plate shear wall (CSPSW) [11]. Undoubtedly, incorporation of an opening in CSPSW reduces stiffness and energy dissipation capacity of the system. Consequently, the displacements will also increase. The degree of these variations depends on the opted remedial approaches to decrease the negative influences of openings on the performance of the system.

The system investigated in this research is a special system of composite shear walls called C-SPW. Shear connectors are used to bond steel members of the composite shear wall to the concrete. Steel-concrete composite shear walls are studied with the numerical analysis via 8 different steel-concrete composite shear walls in which J-hook connectors' effect is the most important research innovation in this research. Additionally, validation of numerical result is done with experimental test. For cast concrete in situ, shear plates and studs are usually used. Finally, bolt can be used for connecting precast concrete walls. Research has shown that connectors in this type of walls are not only exposed to shear force but are sometimes subjected to considerable tensile stresses due to the local buckling of the steel sheet. As a result, in this paper, we consider J-shaped connectors that are attached to steel sheets on both sides of the wall and buried inside the concrete. These connectors are locked together as hooks, connecting the two metal plates and forming a proper integration (Figure 1).

In addition, there are some investigations about composite materials which are included steel-concrete joints beacause of the important of this issue. For example, Mhalhal et al describe the behavior of steel–concrete-steel sandwich beams with new configuration of shear connector [12]. Moreover, Jahangir and Esfahani present an experimental investigation concerning SRG bond behavior applied to steel reinforced grout composites and masonry substrates [13]. Additionally, Zhou et al propose a new interlocked angle connector (IAC) for sandwich structures to enhance the steel-concrete interfacial bonding performances, and its tensile behaviors were also experimentally and analytically studied in detail [14].

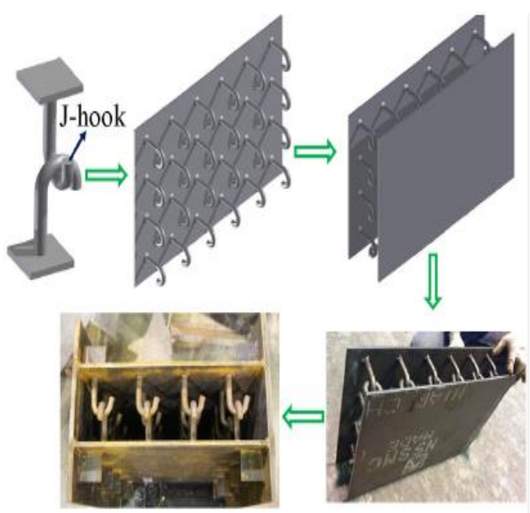


Fig 1. J-hook connector and steel sheets of physical model of the composite shear wall [4].

2. Numerical analysis

2.1. Model Validation

For modeling validation, a composite shear wall sample developed by Arabzadeh et al. [5], has been selected and modeled with the FEA model in the Abaqus software. In the experimental study of Arabzadeh et al., three-story composite shear walls were fabricated and loaded where No. CS1-1 with the following specifications is modeled and validated (Table 1).

Table 1. Dimensions and sections of the shear wall assemblies.

NO.	Member	Size
1	Column	2IPE100+2PL100×5
2	Bottom beam	2IPE100
3	Top beam	2IPE100
4	Steel sheet thickness (mm)	2
5	Connector number	4
6	Connector diameter (mm)	6
7	Diameter of the reinforcement (mm)	3
8	Concrete wall thickness (mm)	30

The mechanical properties of the elements used in the sample CS1-1, which contains the yield stress, final stress, the modulus of elasticity of the steel used in the sample, as well as the modulus of elasticity and the characteristic strength of the used concrete are listed in Table 2.

Table 2. Materials specifications for different components.

Member	Modulus of elasticity, (GPa)	Yield stress, fy (MPa)	Ultimate stress, fu (MPa)	Compressive strength, fc (MPa)
Wall sheets	200	268	415	-
IPE100 beam flange	200	308	479	-
IPE100 beam web	200	285	446	-
Wall concrete	30	-	-	72.5

In the Figure 2, the experimental sample of Arabzadeh et al. is shown, whose dimensions and sizes are identified in the figure. The joints of this specimen, including the beam-to-column joint at the top and bottom, are considered to be completely rigid. It should be mentioned that the bottom of the wall is restrained for any transitional movement.

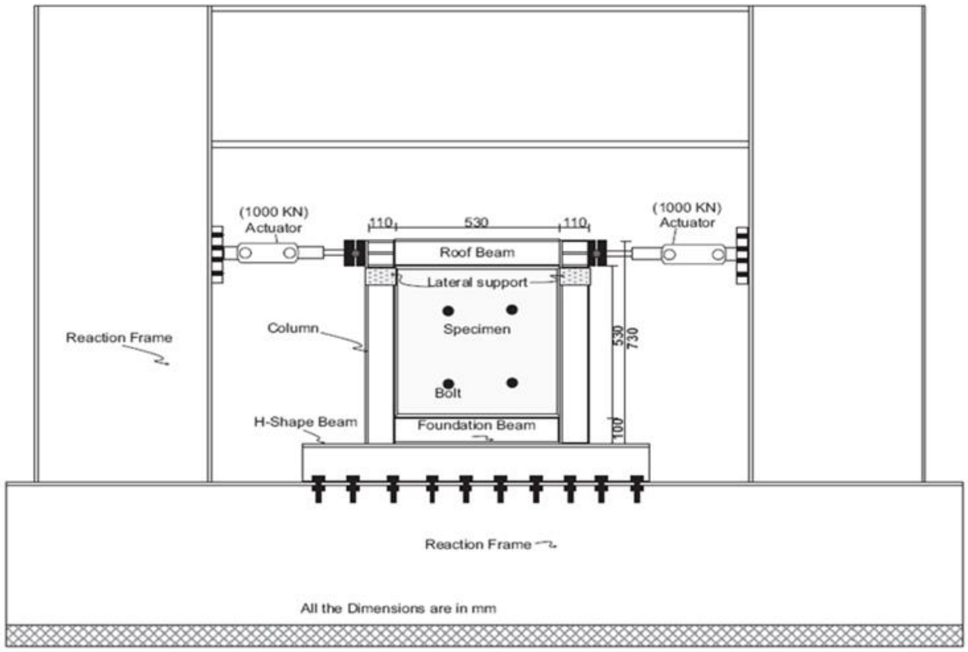


Fig 2. The experimental sample of Arabzadeh et al [5].

After the analysis and comparison of experimental samples with the FEA model, the Hysteresis graph, in Figure 3, and a comparison of the main parameters is shown in Table 3.

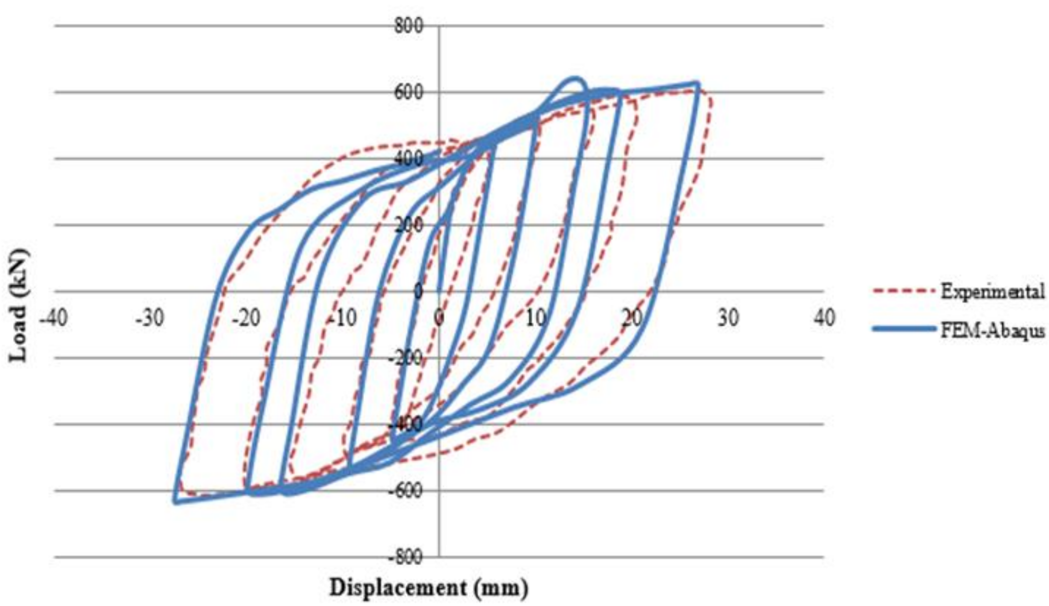


Fig 3. Hysteresis comparison diagram of the FEA model and the experimental sample.

Table 3. Comparison of analytical parameters of the laboratory sample and the FEA model.

Parameter	Percentage of difference	FEA model	Laboratory sample
Yielding force (kN)	4.2	397	380
Yielding displacement (mm)	3.1	4.75	4.9
The initial stiffness (kN/mm)	3.3	83.6	80.9
Maximum force (kN)	4.8	625	595
Maximum displacement (mm)	4.3	25.88	27
Ductility	1.1	5.44	5.5
Damped energy (kN-m)	3.6	241.3	250

In table 3, the analytical parameters of the FEA and experimental models are listed. According to this table, the difference between the FEA model parameters with the Arabzadeh et al. model is less than 5% error. Given this slight difference between the two samples, it can be concluded that the FEA model constructed in this study is acceptable and can be invoked as a software model for further research.

2.2. Numerical Modeling

2.2.1. Modeling in ABAQUS Software

In this paper, constitutive stress-strain models, which used to model materials based on ABAQUS modules options. The boundary conditions in this modeling are similar to the validated model. Loading is applied as a pushover load to the structure (according to Figure 4 based on ATC24). In this model, the displacement control method is used so that the load is applied to the upper surface of the wall. Above all, the supports load forces and the displacement of the load location are extracted. Finally, by eliminating the time parameter from the two diagrams, the displacement graph is plotted in terms of the support force. Moreover, the amount of displacement similar to the laboratory sample is applied to the upper part of the columns and the support reaction is read in the location of the shear wall base. The lower beam transition has also been restrained to apply the sample boundary conditions.

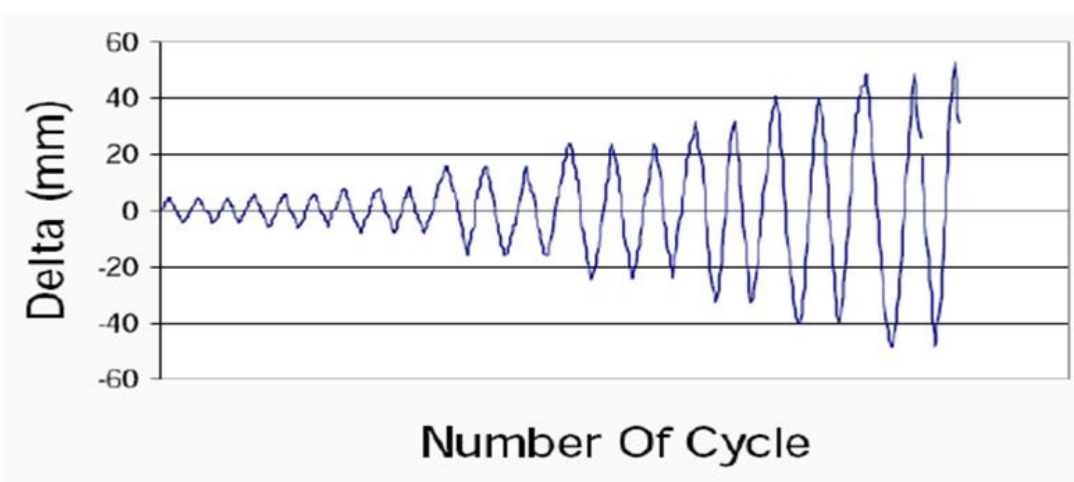


Fig 4. Loading protocol based on ATC24.

The surface-to-surface contact interaction was used to define the interaction between the concrete wall, and the steel sections (beams, columns and shear wall sheets). In this tangential interaction, the coefficient of friction equal to 0.6 between the components is defined which is a good simulation of the interaction between the concrete and the steel due to the surface roughness. It is based on the adhesion intensity variable, introduced by Frémond, which is a surface damage variable and its values vary between zero (no adhesion) and 1 (perfect adhesion) [15]. As a result, the best compatible friction coefficient is selected 0.6 in verified model via experimental results. In the normal contact, the hard contact interaction is defined that the components are not allowed to dip into each other and become separated when traction occurs. A constraint couple is used for connecting the connector to the steel plate that is a proper simulation of the welding behavior at the connector attached to the steel plate (Figure 5).

In this model, both horizontal and vertical steel bars are modeled by link elements which have been embedded in concrete [16].

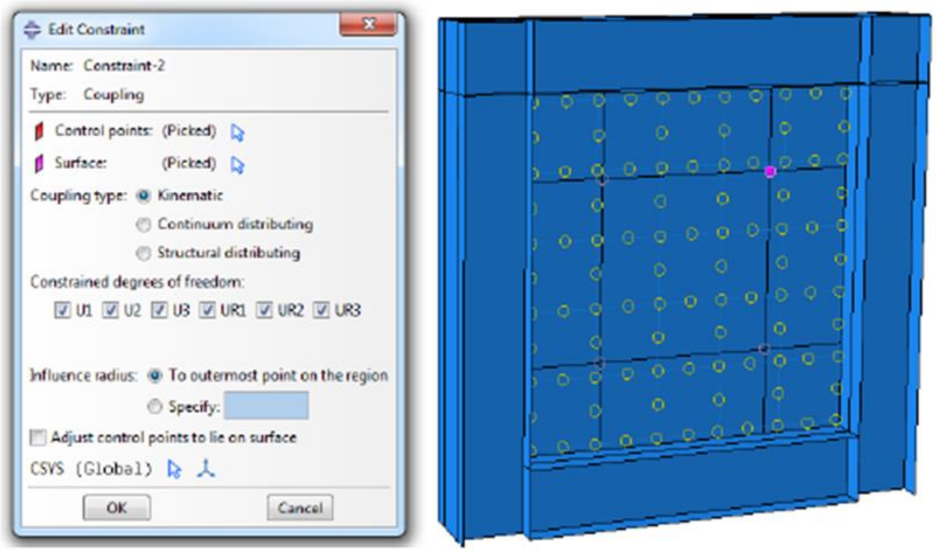


Fig 5. "Couple" constraint to define the attachment of the bolt to the steel plate.

All members are modeled by shell elements. The intended element for meshing the model is S4R that is a four-node shell element with reduced integrals. For meshing concrete wall, C3D8R element is used which is an eight-node three-dimensional cubic element with reduced hardness. Reinforcement bars are modeled by using the T3D2 element that represents a three-dimensional two-node truss element.

Generally, the elements used in the numerical models of this research are presented in Table 4 which the mechanical properties of the elements used are listed in Table 5. These values are chosen to be as close as possible to the real process of concrete and steel production.

Table 4. Dimensions and sections of shear wall assemblies.

Member	Size
column	Box200×200×10
Floor beam	PG200×6×200×8
Steel sheet thickness (mm)	5
Number of connectors (mm)	variable
Connector diameter (mm)	6
Concrete wall thickness (mm)	200

Table 5. Materials specifications for different components of the shear wall.

Member	Used steels	Wall concrete
Compressive strength, fc (MPa)	-	25
Ultimate stress, fu (MPa)	370	-
Yield stress, fy (MPa)	240	-
Modulus of elasticity, (GPa)	200	25

2.2.2. Defined Numerical Models in ABAQUS

In this research, a shear wall in a steel frame with dimensions of 3×3 meters is both modeled and analyzed in 8 different modes listed below (Table 6), so In the following, different types of numerical models are discussed.

In Table 6, the last column represents the veriety of J-hooks connectors jointing to the plate. For instance, in row 3, all nine connectors were modeled and placed in whole zones of steel plates or in row 4, composite shear wall means that no connector are placed on steel plates.

Table 6. Numerical Models Defined in the ABAQUS Software (length units are mm).

Row	Model	Steel thickness	Concrete thickness	Connector number	Form of connectors
1	Frame	-	-	-	-
2	SP	5	-	-	-
3	SP-C	5	-	9	Whole connectors
4	SC-R	5	200	-	Composite shear wall
5	SC-1C	5	200	1	Center only
6	SC-3D	5	200	3	Diagonal
7	SC-5C	5	200	5	X-shape
8	SC-9F	5	200	9	Whole connectors

Frame model

In this model, a steel frame without a steel sheet, concrete wall, and connector is investigated on the effect of shear walls on frames.

SP model

The purpose of this model is to investigate the difference between the steel shear wall and the composite shear wall with two steel sheets as a steel shear wall. The mode of failure is shear as shown as in Fig.7.

SP-C model

The steel frame is modeled with two sheets as a steel shear wall, and to control the buckling of the wall sheets, nine J-hook connectors are attached in three triple rows at equal intervals are used to define the SP-C model. The aim of the model is to discussed the impact of shears on the buckling of steel sheets and the wall bearing capacity.

SC-R model

The metod is used to simulate in the present model is a steel frame with two sheets as a steel shear wall and concrete between them, noting that no connector has been used to attach the concrete wall to the sheets. This simulation is studied the capacity of shear wall composite with and without connectors.

SC-1C model

The steel frame with two sheets is modeled as a steel shear wall and concrete between them for modeling SC-1C model with a J-hook connector in the middle of the wall that is used to attach the concrete wall to the sheets.

As can be seen in Fig.10, the mode of failure is tension failure.

SC-3D model

Three diagonal J-hook connectors are used here to attach the concrete wall to the sheets to introduce the steel frame with two sheets as a steel shear wall and concrete between them. In Fig.11, the mode of failure is shear-moment failure.

SC-5C model

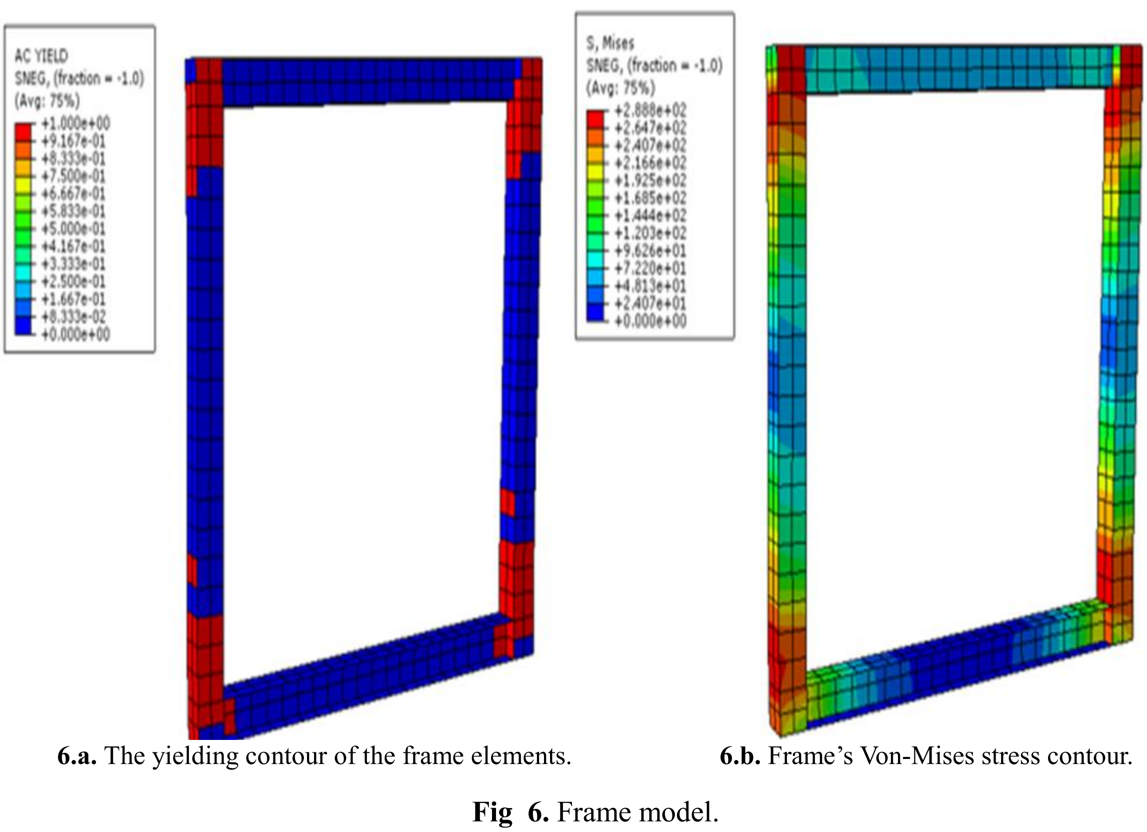
The SC-5C simulation, the steel frame with two sheets is modeled as a steel shear wall and concrete between them. In addition, a J-hook connector in the middle of the wall and four connectors in the four corners of the wall were used to attach the concrete wall to the sheets.

SC-9F model

The whole conectors model, the steel frame with two sheets is modeled as a steel shear wall and concrete between them with nine J-hook connectors which are used in three triple rows with equal intervals.

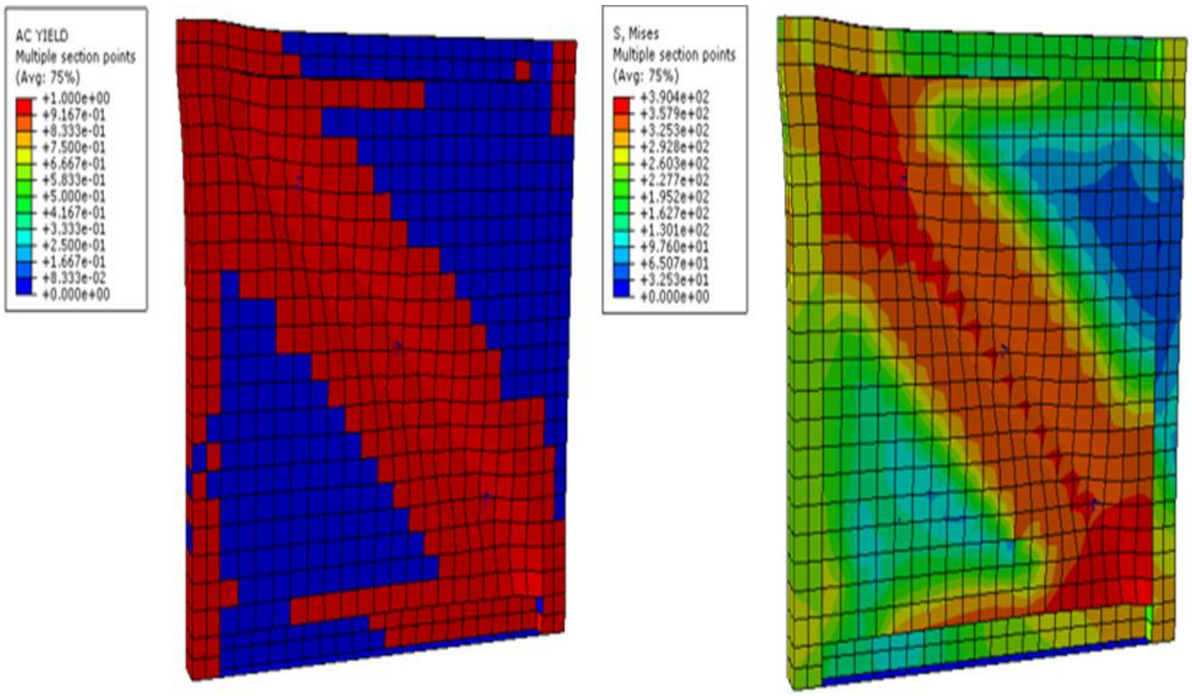
3. Results

The results of the force-displacement diagram, the yielding contour of the frame elements and Frame's Von-Mises stress contour of all models are shown in Figures 6 to 11 respectively except the whole model that is explained in the next part.



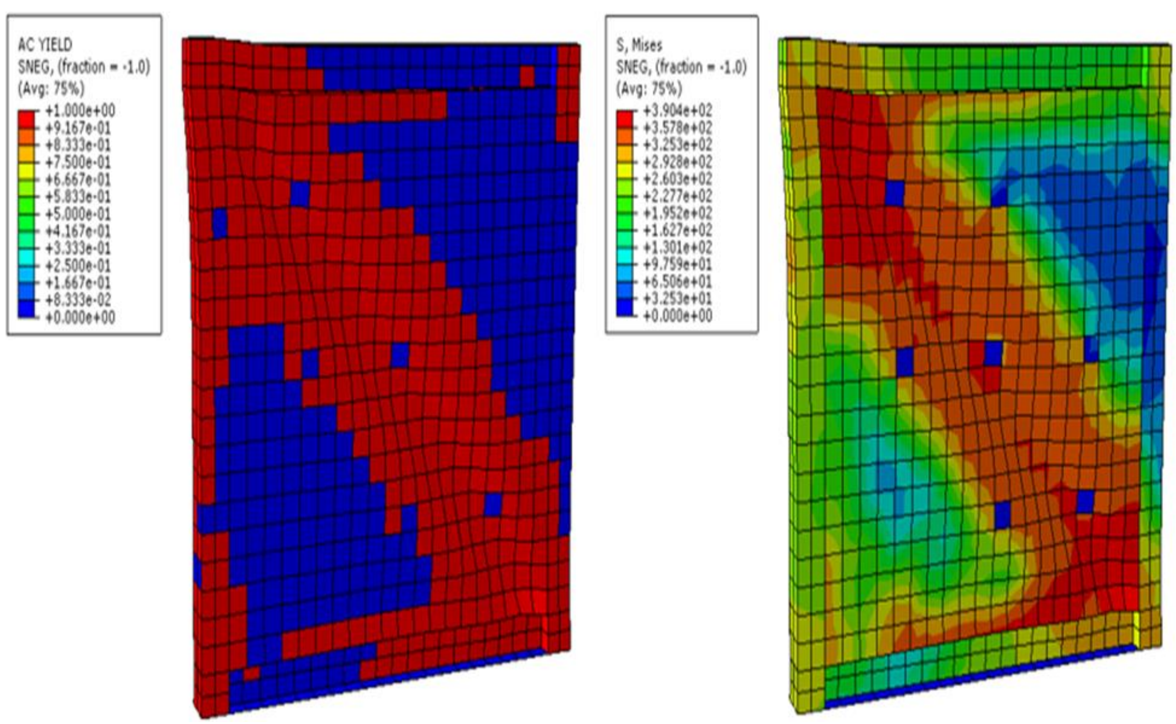
6.a. The yielding contour of the frame elements. **6.b.** Frame's Von-Mises stress contour.

Fig 6. Frame model.



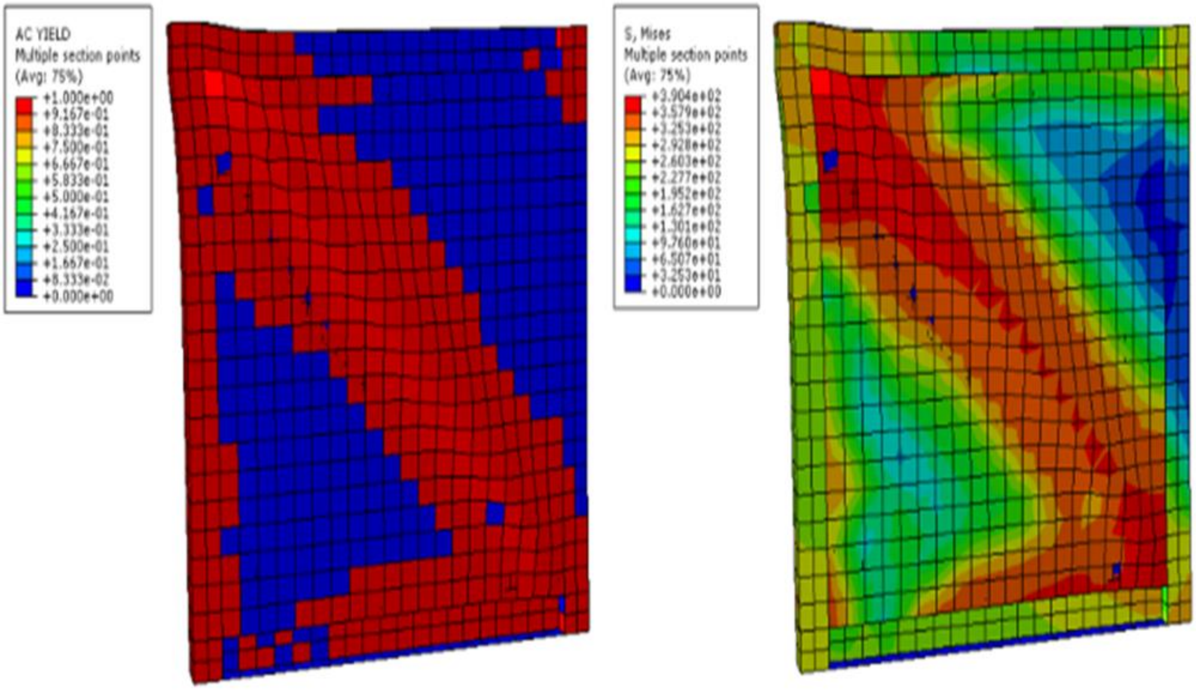
7.a. The yielding contour of the frame elements. **7.b.** Frame's Von-Mises stress contour.

Fig 7. SP model.



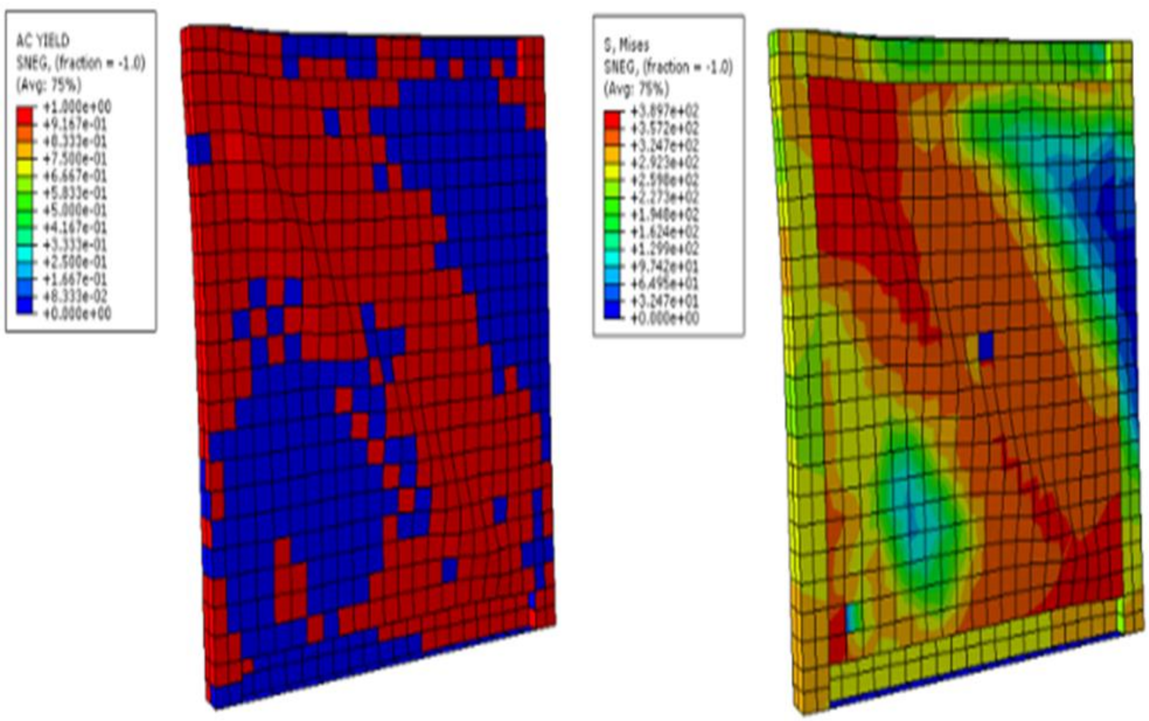
8.a. The yielding frame elements contour. **8.b.** Frame's Von-Mises stress contour.

Fig 8. SP-C model.

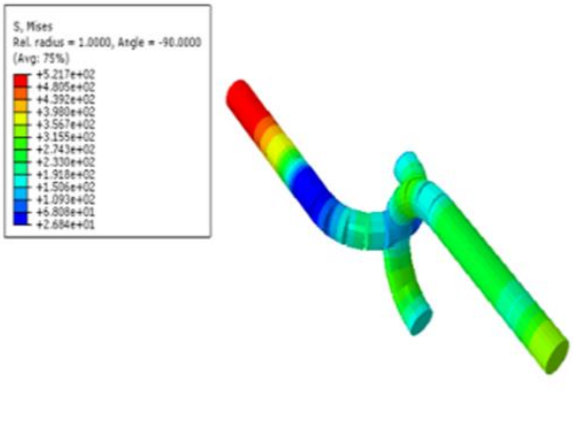


9.a. The yielding frame elements contour. **9.b.** Frame’s Von-Mises stress contour.

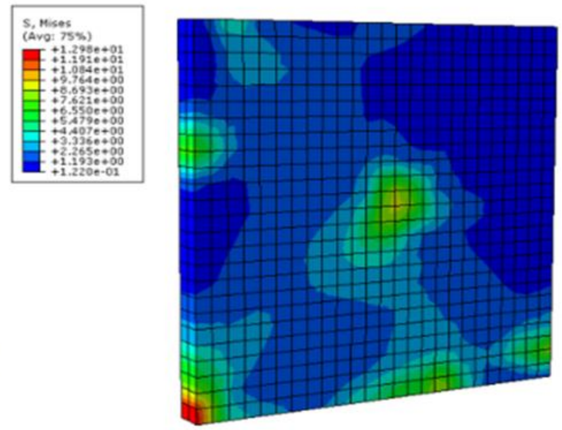
Fig 9. SC-R model.



10.a. The yielding frame elements contour. **10.b.** Frame’s Von-Mises stress contour.

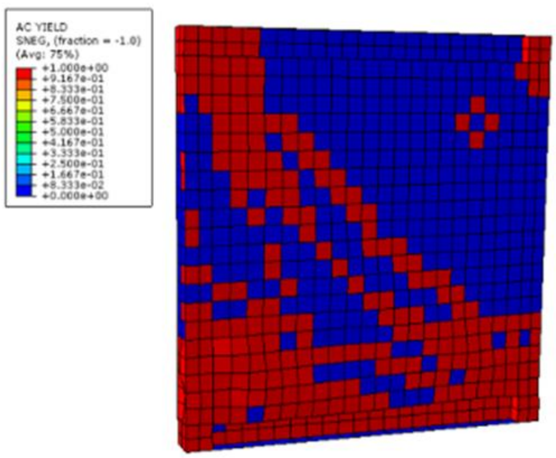


10.c. J-hook connector Von-Mises stress contour.

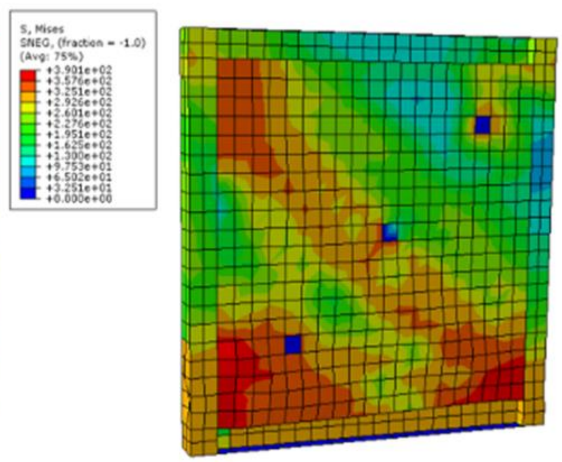


10.d. Concrete wall's Von-Mises stress contour.

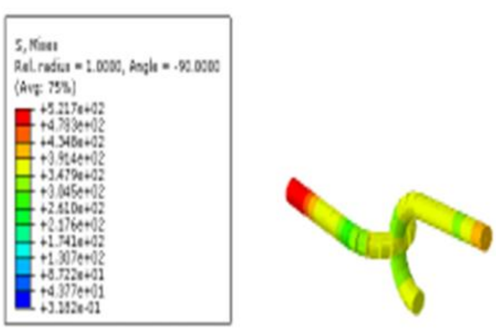
Fig 10. SC-1C model.



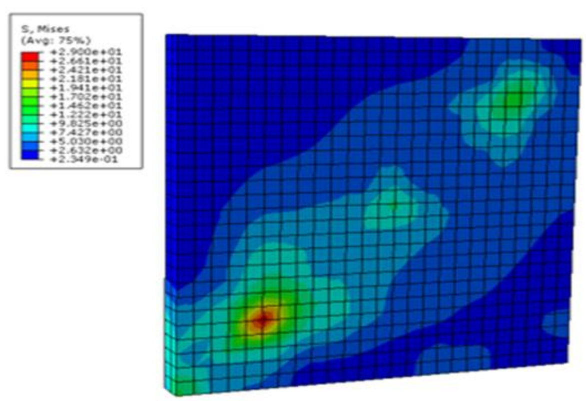
11.a. The yielding frame elements contour.



11.b. Frame's Von-Mises stress contour.



11.c. J-hook connector Von-Mises stress contour.



11.d. Concrete wall's Von-Mises stress contour.

Fig 11. SC-3D model.

Moreover, the force-displacement diagram of all models except SC-5C model are illustrated in the single digram simeltanusely acording to Fig. 12.

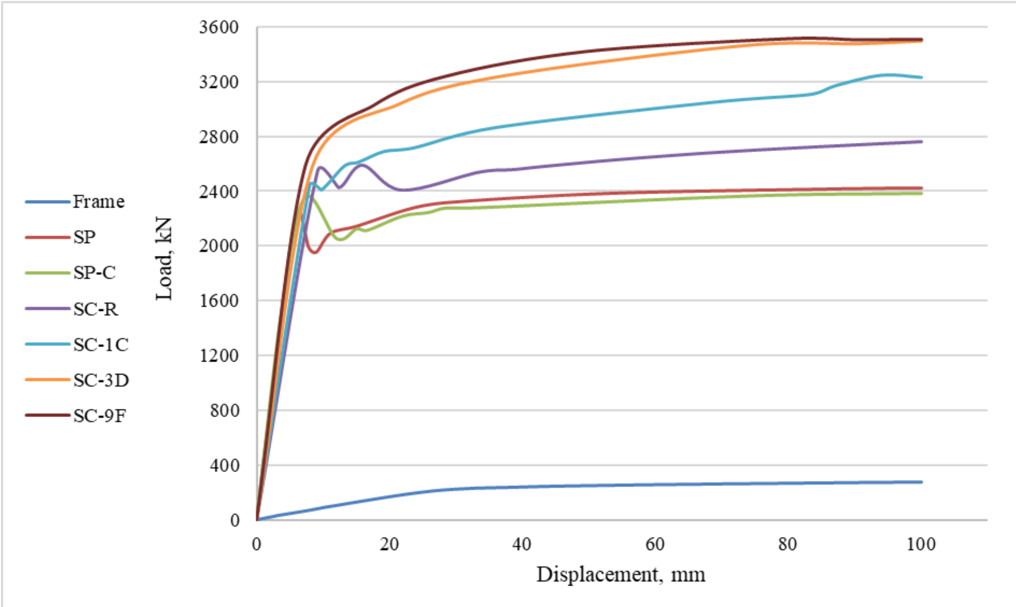


Fig 12. Force-displacement diagram of all models except the SC-5C model.

3.1. SC-5C Model

In Fig.13, the stress contour on steel sheets are shown. This figure shows that most of the stress in the steel sheets lies near the beam and column of the frame, due to the maximum shear stress in those areas. In Fig.14, also the yielding contour of the steel sheet is displayed. In areas where stress reaches its maximum value in the stress contour, the steel plates are yielded. This figure illustrates that yielding does not occur in the areas around the connectors, which can be concluded that the connectors do a great deal to prevent distortion and buckling of the steel sheets.

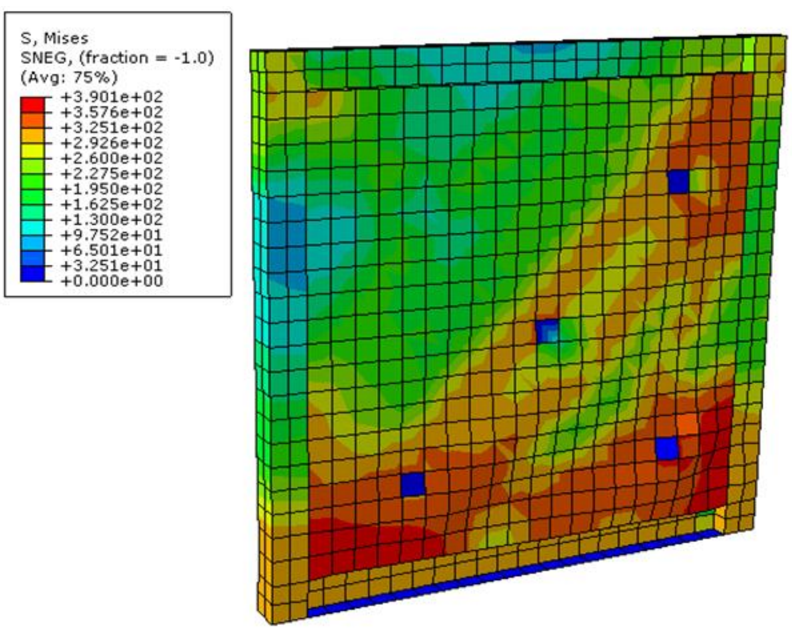


Fig 13. Von-Mises contour of the steel sheets.

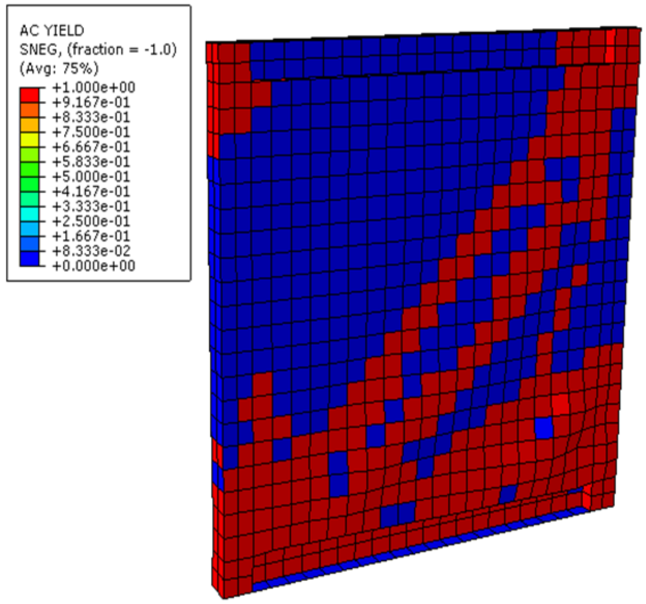


Fig 14. The yielding contour of the steel sheets.

Fig.15 demonstrates the stress contour on the concrete wall. As shown in the figure, the stress on the concrete wall is not extrimely high and is less than the characteristic strength of the concrete which causes the concrete to be undamaged. As a result, the wall failure firstly starts with the steel sheets. The controlling parameter of the model is steel sheet failure. Moreover, Fig.16 expresses the tension in the connectors. As shown in the figure, the stress value reaches its maximum value at the end of the connector and at the connector's attached area to the steel sheet which can be comprehended that by buckling of the steel sheets, these connectors are affected by tensile stresses.

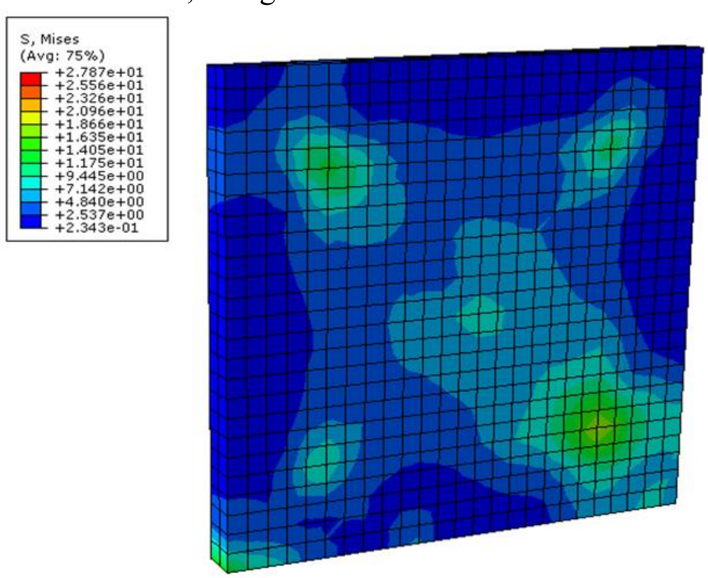


Fig 15. Concrete wall's stress contour.

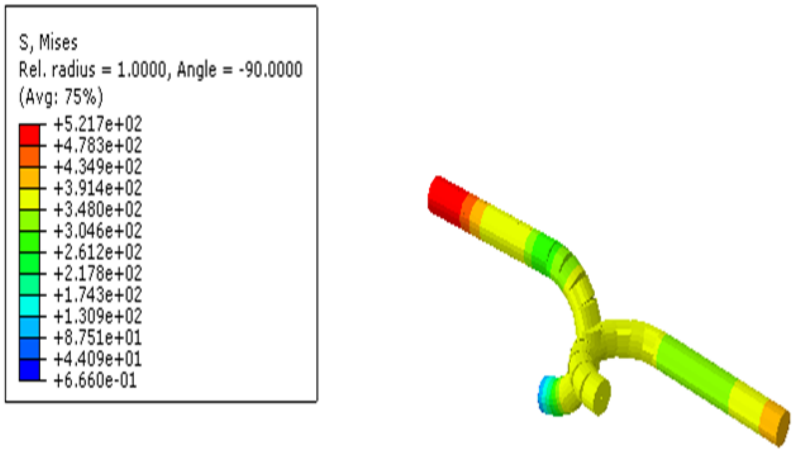


Fig 16. J-hook connector stress contour.

Finally, similar to the other models in this study, the force-displacement diagram of the original model is plotted in Fig.17. The comparison of these graphs and the capacity of these models are presented in Fig.18.

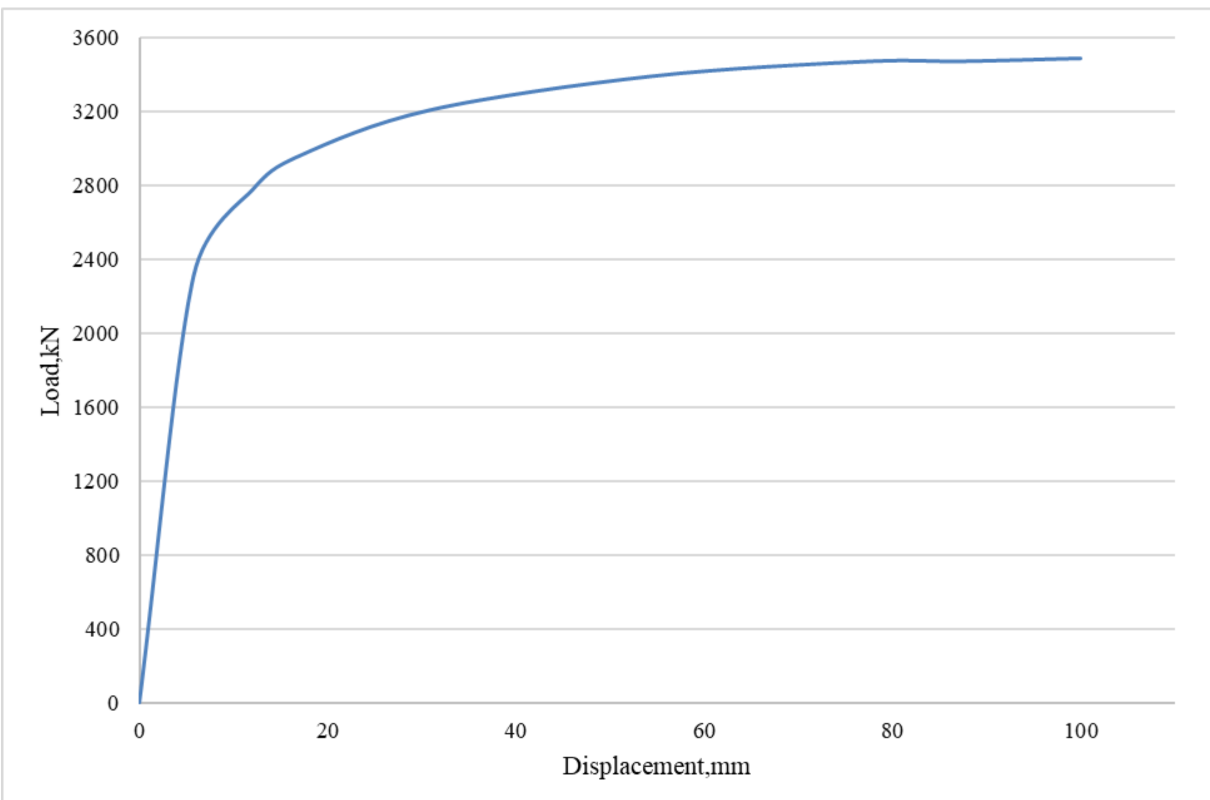


Fig 17. SC-5C model force-displacement diagram.

Fig.18 and Table 7 displays the diagram of the shear wall bearing capacity of the various models. This bar chart confirms the differences in the wall bearing capacities of different models and shows that among the models in this research, models with five and nine connectors have the highest bearing capacity.

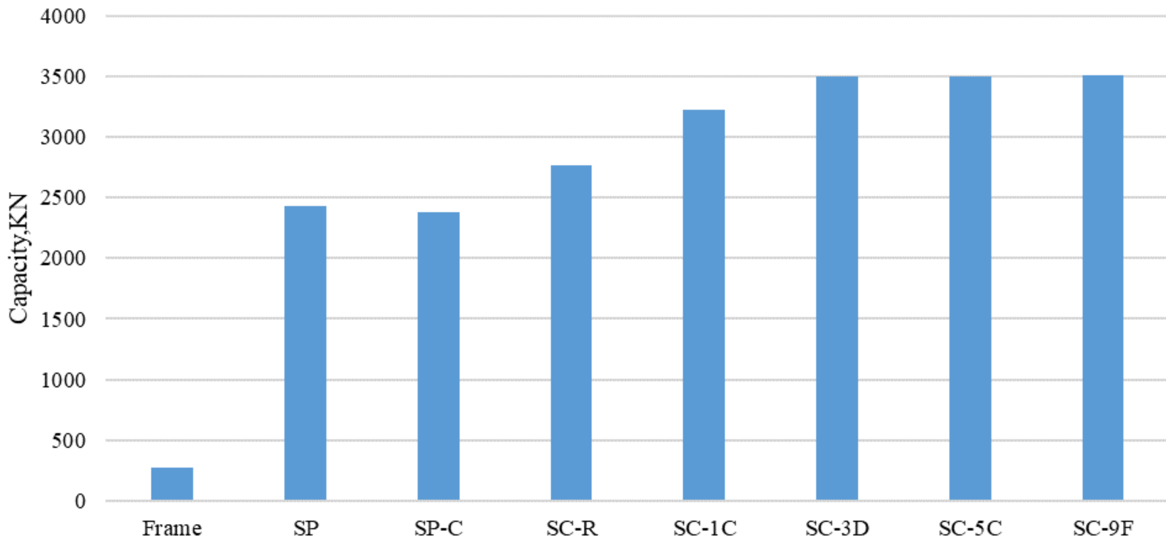


Fig 18. The bearing capacity of the numerical models.

Table 7. The bearing capacity of the numerical models.

Model	Capacity(KN)
Frame	278
SP	2426
SP-C	2383
SC-R	2763
SC-1C	3227
SC-3D	3496
SC-5C	3496
SC-9F	3508

By comparing the Frame model and SP model, it can be concluded that adding a steel shear wall to the non-wall moment frame, will increase the stiffness of the frame which dramatically increases the bearing capacity of the moment frame.

Interestingly,the comparison between the SP-C model and the SC-R model, it can be determined that the addition of a concrete wall to the model, without connectors, will increase its bearing capacity up to 14 percent. This increase in the bearing capacity is due to the buckling and distortion control of the steel sheets and has nothing to do with the performance of the steel and concrete composite in the wall. From this comparison, it can also be concluded that using concrete in shear walls is very desirable to control the buckling of the shear wall steel sheets.

Moreover, when it is compared the SC-R model with the SC-1C model, it turns out that even with a single connector in the middle of the wall, the shear wall composite performance is somewhat formed and the wall's bearing capacity has increased by about 14%. However, there is a defect in the corners of the wall where the buckling of the steel sheets is the highest and this weakness cannot be solved by a single connector.

With comparing the SC-1C model and the SC-3D model, it is clear that the distortion and the buckling of the steel sheets have been controlled by adding connectors at the corners of the wall. Nevertheless, due to the freedom of the other side of the wall, if the applied load is deflected, the wall defects still exist and the bearing capacity is reduced. The results show that the use of whether 5 or 9 connectors that restrain the corners of the wall is sufficient to utilize the maximum wall bearing capacity.

4. Conclusion

In this paper, the results show that the steel shear wall has a higher bearing capacity than the moment frames. Moreover, no significant increase in the bearing capacity of the steel wall with and/or without connectors is/are understood. However, adding concrete to the steel shear wall which consists of two steel sheets, rises the wall's bearing capacity by 14 percent.

On the other hand, the main focus of this research is on the effect of J-hook connectors on the performance of composite walls. As can be realized from the results of the analysis, when the concrete is added to the steel shear wall, it increases the wall capacity although the full composite performance has not yet formed between the steel and the concrete. Adding connectors to the composite are improved the performance of the interaction between concrete and steel so that the outputs show that the bearing capacity of the wall has augmented by about 17% after the connectors have been replaced. The analysis demonstrates that the bearing capacity of the composite shear walls is subjected to the yield of the steel sheets and less damage is developed in the concrete during loading. The yielding of steel sheets also follows when distortion and buckling occur. Another use of J-hook connectors is to control the aforementioned distortion and buckling, which increases the wall capacity. The results show that most of the distortion and buckling has occurred at the corners of the wall. The steel sheet buckling can, therefore, be controlled by placing the connectors in the corners. In these models with three, five , and nine connectors, the connectors are positioned in the corners and because of this kind of positioning, the bearing capacity of the wall has increased. Nonetheless, in the model with three connectors, there is still one free corner and it can not be said that the wall shows its maximum bearing capacity. Additionally, models with five, and nine connectors provide maximum wall bearing capacity. However, due to the slight difference in bearing capacity between the two models, it can be understood that the model that has five connectors has an acceptable bearing capacity with better economic justification.

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