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Lateral bracing and steel shear wall integration in steel high-rises

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Abstract. In high-rise building structures, the designers tend to utilize tube systems, or their combination with other structural frame systems, as an efficient lateral resisting structural system. The drawbacks of tube systems include the effect of shear lag and the architectural problems arising from the closely spaced columns. These drawbacks can be remedied by using exterior braces (concentric bracing system), which provide high shear stiffness in combination with the tubes. Since in high-rise building structures control of bending drift is so difficult and complicated, utlizing the exterior braces is regarded as a practical method due to its high shear and bending stiffenesss. In so doing, in this paper, an innovative concept was investigated in which steel plate shear walls are utilized at the two extreme bays of a frame, and giant exterior braces are used between the shear walls. These two walls act as strong moment arms against the overturning moment and, because of their high stiffness, absorb most of the produced shear; consequently, the shear lag effect diminishes. The obtained results indicate that in the proposed system, the lateral displacement is diminished by around 2.13 times; consequently, the axial forces and bending moments in columns are reduced considerably by about 30 % and 50 %, respectively, demonstrating this system's high effectiveness.

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1. Introduction

Human has always been fascinated by tall buildings and high-rise towers. In ancient civilizations, tall structures were used for defensive purposes and symbolic and ceremonial applications. The growth and development of new high-rise structures with residential and commercial applications began in the 1880s [1]. Till now, several approaches have emerged to control the drift of the tall building, including dampers (passive, active, or semi-active) (Fisco and Adeli [2, 3]);(Nie et al [4]);(Farzampour et al [5]); (Zhao et al [6]); Qiu [7]);(Meghdadaian and Ghalehnovi [8]); (Al-Rumaithi et al [9]);(Ghamari et al. [10–12]);(AL-Shamaa et al [13])(Arshadi et al [14]);(Maddahi et al [15]); the TMDs (Smith et al. [16]);(Jin and Doloi [17]); (Lin et al., [18]);(Farghaly and Salem Ahmed [19]); (Kim [20]);(Giaralis and Petrini [21]);(Khaleel and AL-Shamaa [22]);(Zhang [23]); which improve a structure's behavior and performance against wind and explosion, are typically considered as semi-active or passive control mechanisms (Moon et al. [24]); (Shayanfar et al.[25]). Fig. 1 illustrates some of the systems for steel structures. (Taranath [26]), stated that the use of tube systems has been deemed effective for tall buildings of the following order.

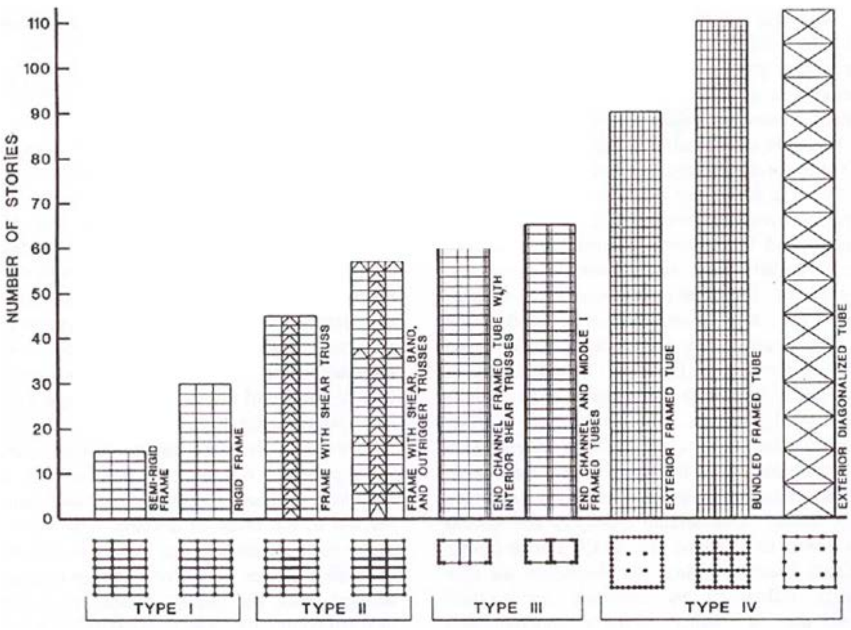


Figure 1. Systems tall buildings [26].

Tube systems are the developed versions of traditional rigid frames. The goal of using this structural form is to transfer most of the lateral load-resisting material to the structure's perimeter to maximize the flexural rigidity of the structure. The tube systems' strength and lateral stiffness are provided by using deep beams and closed columns. Practically, to achieve the optimum geometry and structural system in steel structures, the distance between the columns is set between 1.5 and 3.00 m, and the depth of the spandrels is considered between 0.9 and 1.5 m [26]. Although the tube systems withstand the entire lateral load, the gravity loads are divided among the peripheral and interior columns. When the structure is subjected to a lateral load, the peripheral frames in the direction of loading act as a web, and the frames normal to the loading direction act as a flange [26].

The main drawback of the tubes in controlling the lateral displacement stems from the flexibility of the beams of this system. This flexibility of the beams causes a non-uniform distribution of the axial stresses of the first-story columns according to Fig. 2, which is known as the “shear lag effect”. Another shortcoming of this system is related to the architectural aspects, i.e., the closely spaced peripheral columns. In addition, the problem associated with the closely-spaced columns becomes more apparent in tall buildings where the first stories have parking and commercial applications. These problems are remedied to a large extent by using giant exterior braces.

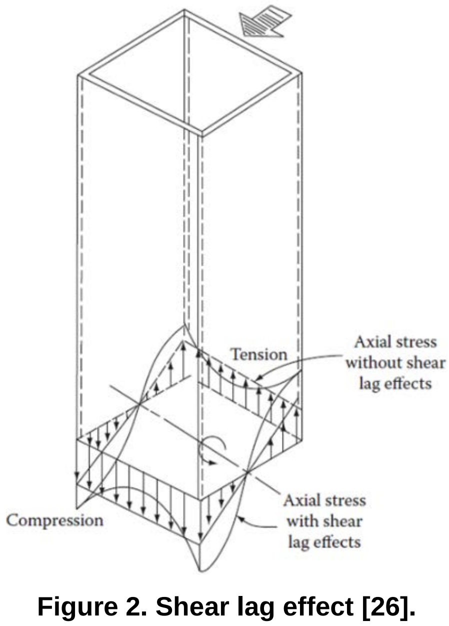


Figure 2. Shear lag effect [26].

By adding the giant exterior braces to the tubes, as shown in Fig. 3, the stiffness and rigidity of the system increase considerably.

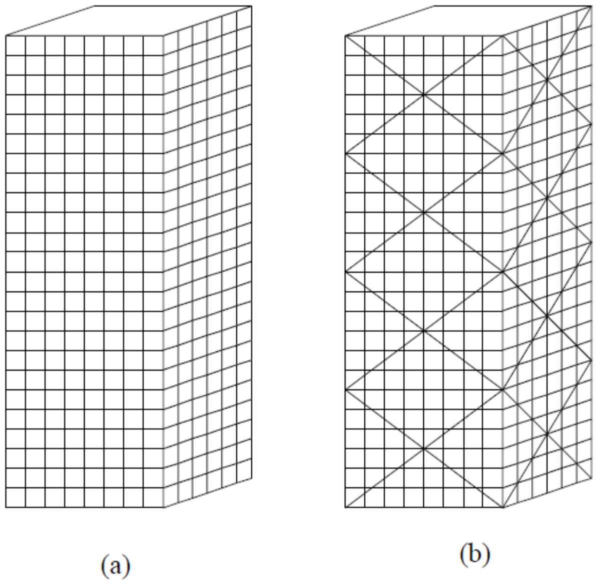


Figure 3. a) Structural framed tube b) exterior diagonalized tube.

By employing diagonal members, more considerable distances between the columns and smaller sections for beams and columns can be used [26]. Also, in the exterior diagonalized tubes, a significant portion of shear is withstood by the diagonal members, which makes the axial stresses of the first story more uniform and therefore reduces the shear lag. Employing these diagonal elements, the system behaves more like a pure cantilever system. This behavior is displayed in Fig. 4. Given the above statements, adding giant exterior braces improves the behavior of a tube system. In this regard, by taking advantage of the benefits of tubes and exterior braces in combination with steel plate shear walls, an innovative system will be introduced and analyzed, improving the structure's behavior.

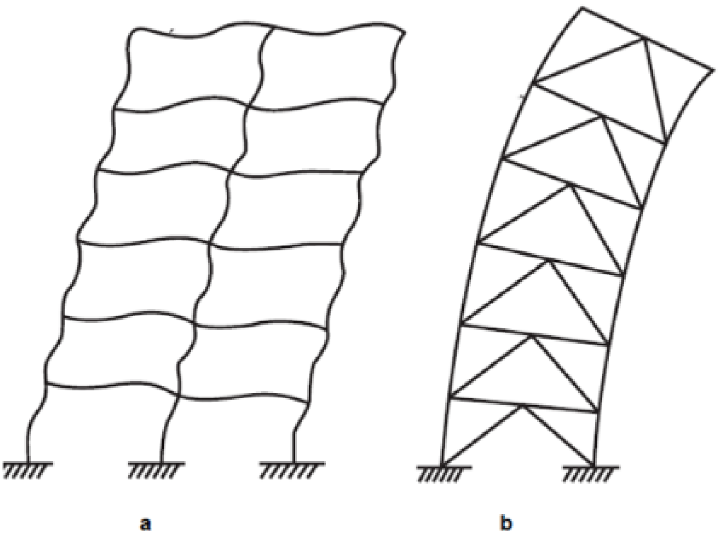


Figure 4. Shear and bending deformations [26].

To achieve a higher stiffness for a structure, usually concentric braces are used, because concentric diagonal braces have higher shear stiffness relative to other types of braces. However, researchers have demonstrated that a steel plate shear wall has greater shear stiffness and lateral strength relative to concentric braces. Considering the same amount and type of material, the stiffness of a shear wall can be 1.7 to 2 times and its ultimate strength can be 1.5 to 2 times that of a concentric brace [26]. The braces are connected to the exterior frame through gusset plates. The failure and rupture of welds in gusset plates resulting from stress concentration or faulty welding prevents the braces from participating in lateral load bearing. However, in steel plate shear walls, the steel plate is continuously attached to the peripheral frame, which reduces the amount of stress concentration. Therefore, the probability of eliminating the steel plate from participation in the carrying of lateral load is much less than the gusset plate weld failure; thus, the use of steel plate shear walls is safer. The higher stiffness and strength of a steel plate shear wall relative to a brace is due to its load-carrying mechanism.

The post-buckled state of a slender plate loaded in pure shear is stable due to the development of tension-field action. In other words, following the deformation of a plate due to shear buckling, some secondary stresses are produced which tend to restore the plate to an equilibrium state. As a result of these stresses, the sudden buckling that normally occurs in column and beam elements is not observed in plates; and after shear buckling, plates exhibit a large loading capacity [27]. Although the occurrence of buckling in beam elements considerably reduces their load-carrying capacity, a Steel Plate Shear Wall (SPSW) withstands under applied loads in tension and compression in both diagonal directions.

By referring to Fig. 5, it can be realized that by the addition of steel SPSW at the two extreme bays of the frame, shear lag may diminish significantly. Also, the compressive stresses developed in slender SPSWs are generally very small in comparison to the tensile stresses and are typically ignored in analysis and design, thus, the SPSW web acts as a tensile brace. Due to the connection of the steel plate to the boundary elements, the produced stresses are distributed to it, and the beams and columns bordering the steel plate have a greater participation in carrying the lateral load. In addition, the presence of the wall at the two extreme bays of the frame, as illustrated in Fig. 6, produces a strong resistive moment arm against the overturning moment. The existence of the braces between the two walls causes the walls to act in tandem; and with the elimination of these braces, because of the flexibility of the beams, the axial forces will not be transferred properly. Besides the above advantages, SPSW has enhanced the redundancy of the structure, which is effective in reducing lateral deflections. In addition to the above advantages, the overall material weight of the structure will decrease, but, we know that the material weight, although important, is not the only factor that impacts cost. It is a reasonable metric to use for assessing cost, but it is not the only one. The numerical models in the subsequent sections will demonstrate the behavior of the proposed system. The main disadvantage of the proposed system, from an architectural point of view, is the usage of the shear walls at the corner spans. Of course, one should bear in mind that this is just a theoretical study and could have practical applications where certain circumstances demand.

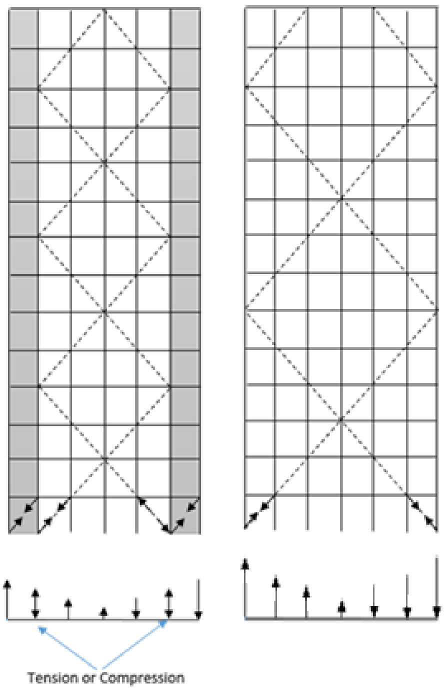


Figure 5. The SPSW impact on the reduction of shear lag.

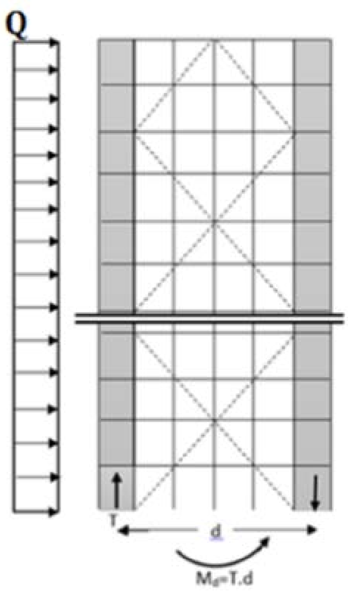


Figure 6. Moment arm resisting the overturning moment.

2. Methods

2.1. Numerical study

Since the analysis of high-rise buildings is time-consuming, to introduce the proposed system and to evaluate its behavior, the intended analyses will be performed in two series. In the first series of analyses, the proposed system will be analyzed as a 2D system subjected to a uniformly distributed load (simplified wind load) to highlight its effectiveness and superiority relative to the other systems. Also, since the analyses are preliminary, some simplifying assumptions were considered. Also, the unit less load is selected for the magnitude of lateral load which the value of the uniform load is equal to Q. In these analyses, the sections selected in all the models are such that the same amount of material is obtained for each system. In the second series of analyses, a real structure is designed and analyzed three-dimensionally under real wind and seismic loads to achieve more realistic results.

Parametric studies were performed for all of the numerical models. For this purpose, the uniform lateral load selected as Q would have the same dimension as axial load, shear force, etc. The parameter Q and the displacement in the said figures are normalized and hence the displacement of each structural system at the same story level is obtained relative to one another concerning the induced uniform lateral load Q. Moreover, the lateral displacement is a function of the dimension of the modulus of elasticity, the moment of inertia of beams and columns, and beam and column lengths.

2.2. Numerical models of Series 1

In the first series of analyses and investigations, four types of 2D structural systems with 40, 60, 80, and 100 stories have been investigated to evaluate and compare their behaviors with each other. In these system designations, M indicates the sole moment frame, M-B shows the combination of flexural frame and exterior braces, M-S specifies the combination of moment frame and steel plate shear wall, and M-B-S indicates the combination of moment frame and exterior braces with steel plate shear walls. By considering the rigidities of the beams and columns, the shear stiffness of the frame at each story is determined approximately from Eq. (1). It should be mentioned that the K values used in Eq. (1) ignore beam shear deformations, joint rigid offsets, and joint deformation [28].

$$K_i = \frac{24E}{h_i^2\left[\frac{2}{\sum k_c} + \frac{1}{\sum k_{bb}} + \frac{1}{\sum k_{bt}}\right]}. \tag{1}$$

In this equation, E is the modulus of elasticity, h is the story height, $\sum K_c$ is the sum of the stiffness values of columns, and $\sum K_{bt}$ and $\sum K_{bb}$ are the sums of the stiffness values of beams at the upper and lower stories, respectively. By considering the moments of inertia of the beams and columns based on Fig. 7 as well as an equal height for the stories, the stiffness of each story in terms of the elasticity modulus (E) is obtained as $K_i = 0.06E$. Also, the shear stiffness of the braces at each story is calculated from Eq. (2), where A is the section area, L is the brace length, n is the number of diagonal braces and α is the brace angle.

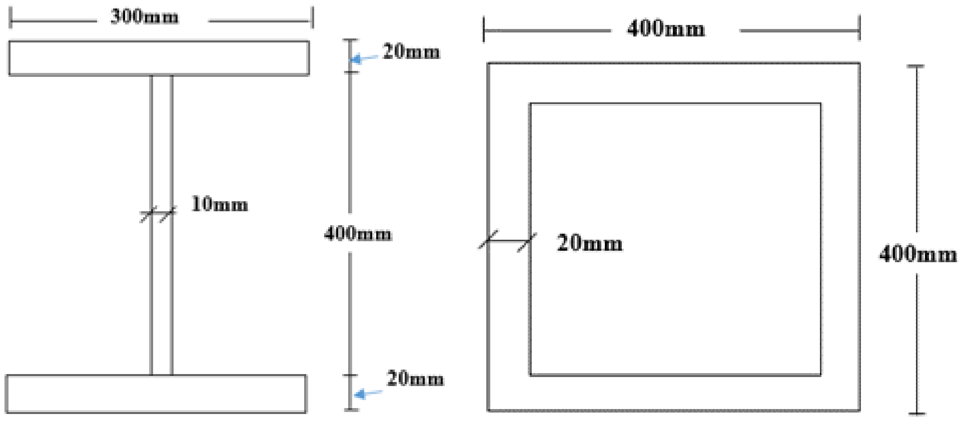


Figure 7. Beam and column geometries.

$$K_{brace} = n\frac{EA}{L}\cos^2\alpha, \tag{2}$$

where, E is the modulus of elasticity n is the number of the diagonal brace elements, α is the angle of the brace with the horizon, and L is the length of the brace.

By equating the stiffness values of the braces and the moment frame at each story, the cross-sectional area of the braces will be equal to $n\frac{E.A}{L}\cos^2 0.79 = 0.06E$. Accordingly, the cross of $A = 45.48$ cm² was obtained. Therefore, the section of 2UNP100 was selected. Knowing that the stiffness of SPSW is determined as:

$$K_w = \frac{Ebt}{4h}, \tag{3}$$

where, t is the thickness of the infill plate, E is the modulus of elasticity, h is the height of SPSW, and b is the length of the infill plate. By equivalent the stiffness of the SPSW and brace, the thickness of the SPSW is obtained as 3 mm. Substituting the $h,\ b,\ t$ and E to Eq. (3) gives the $K_w = 0.08E$.

Thus, with the same amount and type of material, the shear stiffness of a SPSW is 1.33 times $\left(\frac{K_w}{K_{brace}} == \frac{0.08E}{0.06E} = 1.33\right)$ greater than that of braces. So the shear stiffness of the proposed system is expected to be higher than that of the other examined systems; while, with the increase in structure height, bending stiffness dominates. To prove this claim, the analysis results of the models, obtained by the finite element method are discussed.

2.3. Numerical Models of Series 2

In the second series of analyses, a 3D numerical models of a 40-story executive office building has been examined. This structure has been designed and constructed for a wind load according to Fig. 8, and for a seismic load with a base acceleration of 0.35 g. In this structure, the dead load equals 5.5 kN/m^2, the live load for general use spaces equals 2.0 kN/m^2, and the live load for floors equals 5.0 kN/m^2 have all been considered. The building is 40x40 m with the center-to-center distance between the peripheral columns is 4 m, Fig. 8. So, the distance between the C2 columns are 16 m.

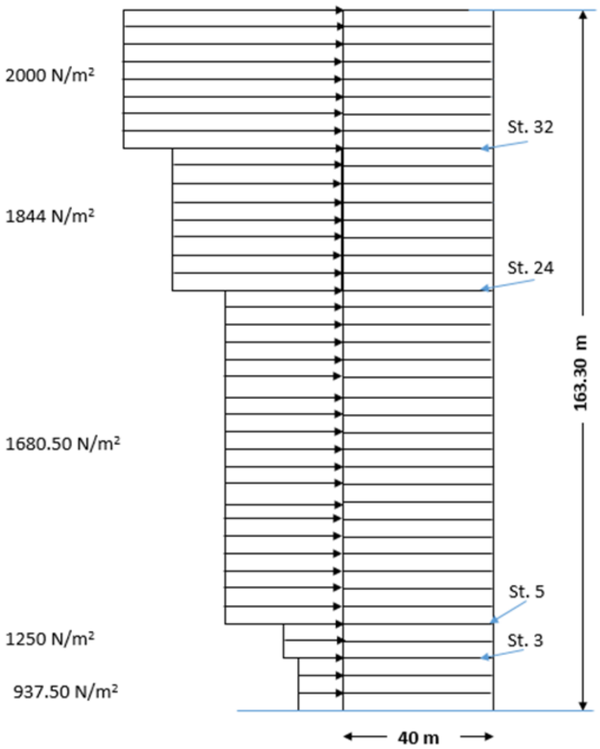


Figure 8. Distribution of wind force at various heights.

The slable thickness of the structure was 170mm. Also, during the design of the structure for seismic loading, the structure was designed under acceleration of 0.8g m/s^2. Also, the spectrum analysis was used to design of the structures. In the design computations of this structure, it has been demonstrated that the wind load governs the design Therefore, this structure will be analyzed for wind load by using the proposed system. The specifications of the structure have been presented in Fig. 9. It should be noted that there are practical issues with using box sections in SPSW construction. The box sections generally need to be filled with concrete or have large internal stiffeners to adequately transfer the web plate force to the column.

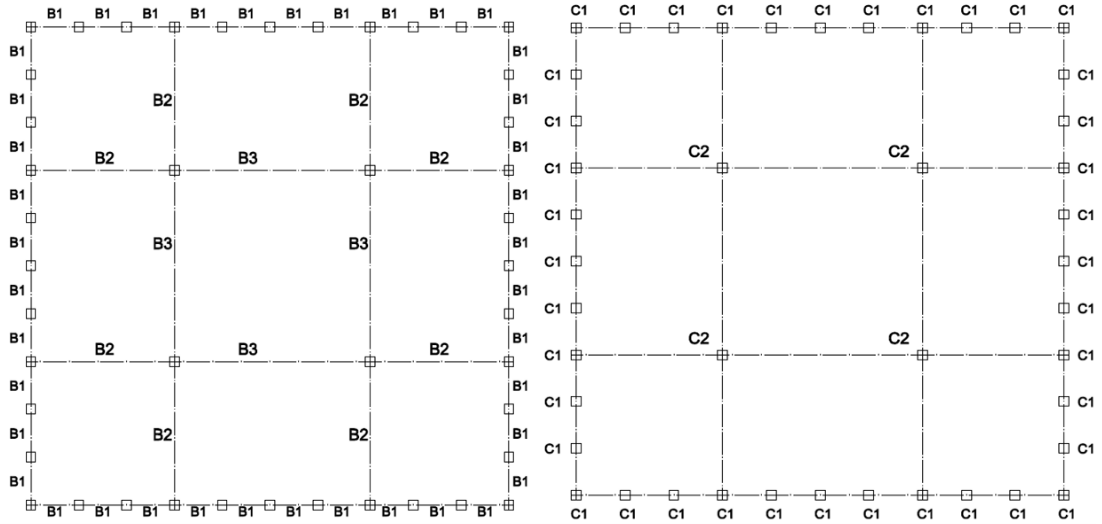


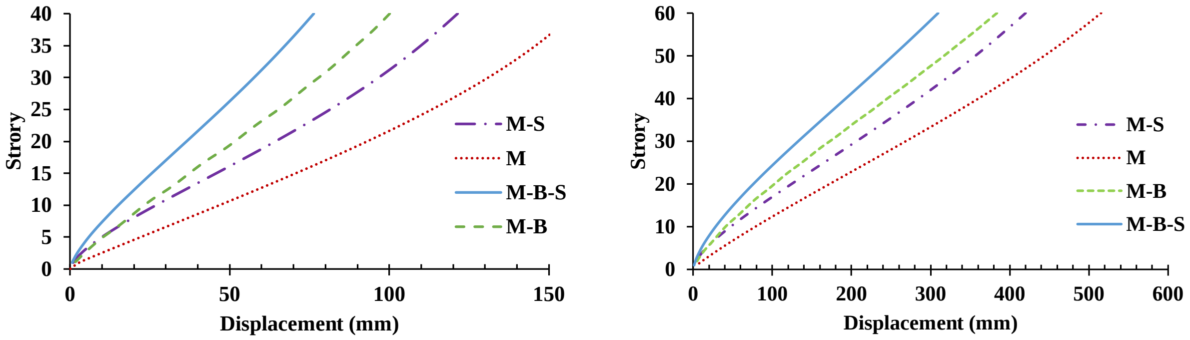
Figure 9. Plan views of beam and column layouts.

3. Results and Disscusion

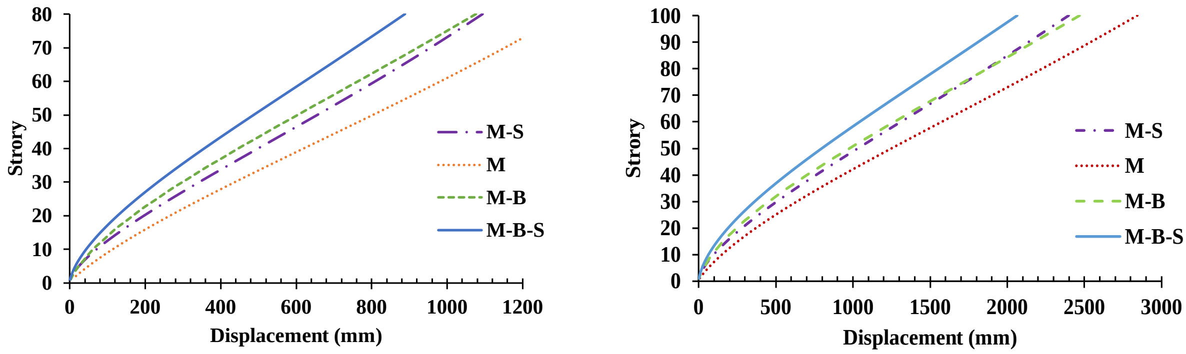
3.1. Discussion on results of Series 1

3.1.1 Displacement and the drift ratio

As was mentioned before, the lateral displacement control of tall structures is one of the most important factors in determining the right system for a particular high-rise structure. The lateral displacements of various frame systems versus the number of stories have been plotted in Fig. 10.



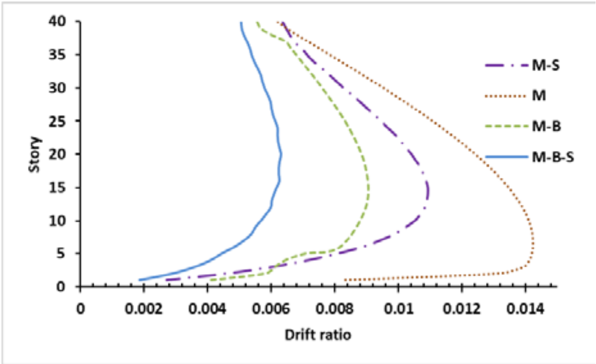
a .40- multistorey model frames **b. 60- multistorey model frames**



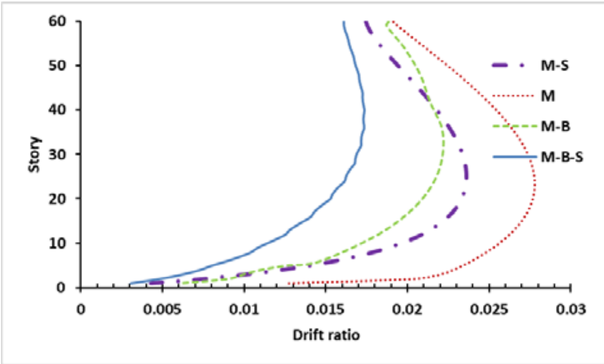
c. 80- multistorey model frames **d. 100- multistorey model frames**

Figure 10. Relative displacements of the multi story model frames.

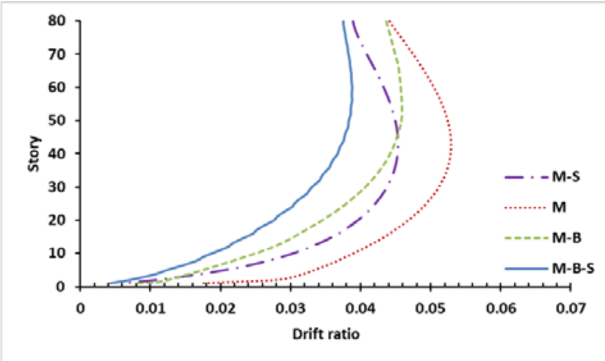
It is observed that the proposed system has the least amount of lateral displacement. Among all the considered systems, the least amount of lateral displacement belongs to the M-B-S system (the proposed system), followed by the M-B system (the moment frame with the exterior braces), the M-S system (the moment frame in conjunction with the steel plate shear wall) and finally the M system (the moment frame alone). The comparison between the drift ratios (horizontal drift to story height) of structures in Fig. 11 shows the better performance of the proposed system relative to the other systems. In comparing the 40-story structures, the M-B system has a lower drift, relative to the M-S system; but in 60 and 80 stories, the drift ratios are close to each other and at mid-stories the M-B have a lower drift, relative to the M-S system. This behavior is reversed at higher stories. In comparing the 100-story structures, the M-B system has almost similar drifts up to 30 stories, relative to the M-S; and at higher stories the M-S system has a lower drift, relative to the M-B system.



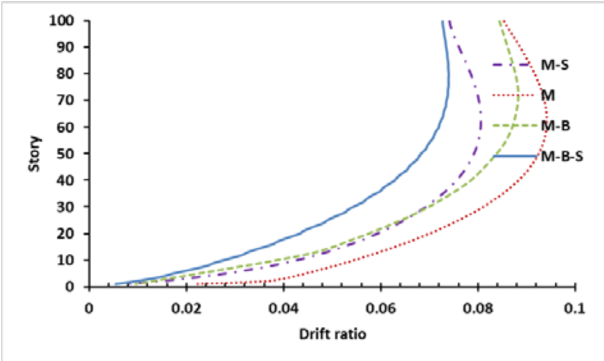
a .40- multistorey model frames



b. 60- multistorey model frames



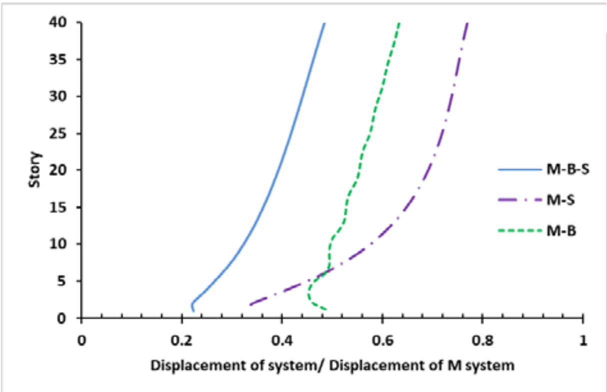
c. 80- multistorey model frames



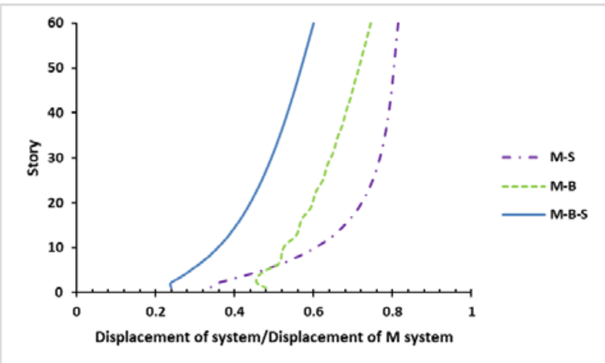
d. 100- multistorey model frames

Figure 11. Relative drift ratios of the multi story model frames.

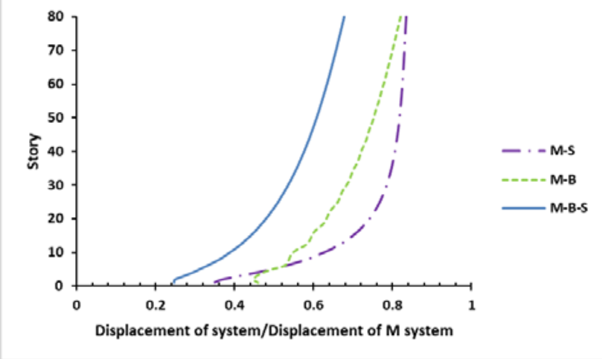
In Fig. 12, the lateral displacements of the systems have been divided by the lateral displacements of the moment frames to yield the percent reduction in the displacement of a system relative to a moment frame. These results better illustrate the effectiveness of a system. A comparison of these results shows a 20 to 50 % in 40-story, 20 to 60 % in 60-story, 20 to 67 % in 80-story and 20 to 73 % in 100-story reduction in the lateral displacement of the proposed system. With this percentage of lateral displacement reduction, a lesser amount of material will be needed to control the frame displacement and thus the system will be more economical.



a .40- multistorey model frames



b. 60- multistorey model frames



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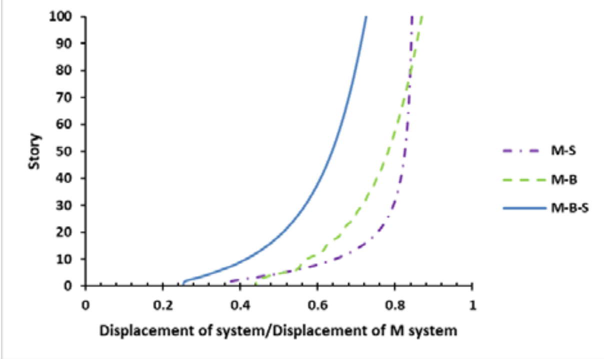
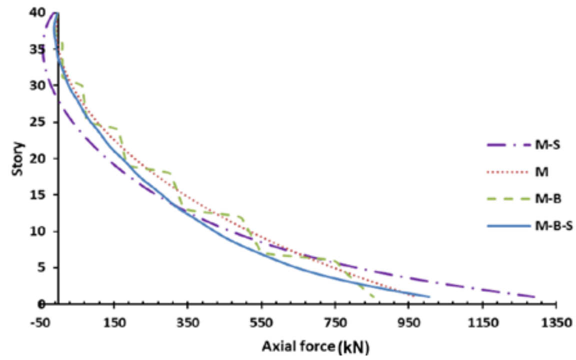


d. 100- multistorey model frames

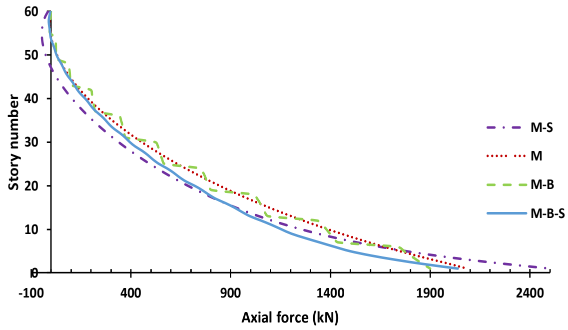
Figure 12. Comparison of displacement ratios of different systems with respect to story numbers.

3.1.2 Axial force in columns

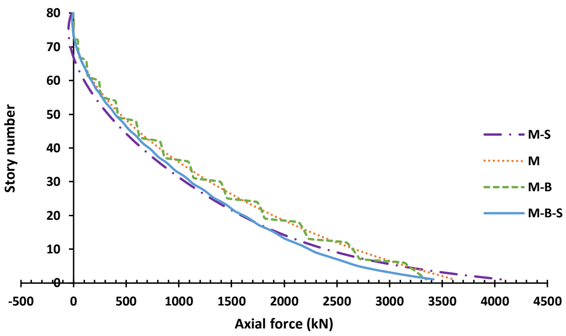
By applying the lateral wind load, axial forces and a bending moment are produced in a column. The axial forces produced in windward columns and leeward columns are tensile and compressive forces, respectively, with their absolute values being equal. Also, an axial force of almost zero magnitude is obtained for the middle column in all the models. In Fig. 13, the axial force in the windward column has been plotted versus the number of stories for all the mentioned systems. The results indicate that in almost all the structures, the largest axial force is obtained in the system containing the steel plate shear wall at the lower stories; however, at the higher stories, the structure has a better performance. In the M-B system, the sudden changes of the axial force at higher stories are clearly observed. Also, the axial force in the columns of the lower stories in the proposed system's 40-story structures is greater than that in the M-B and M systems; however, at the fourth story (almost the mid height of the structure), the said values become equal and from this level on up, the lowest amount of axial force is produced in the columns of the proposed system. Of course, at the very top stories, the forces in the windward columns are compressive forces resulting from the stresses applied by the steel plate shear wall to those columns. The same trend can be seen in the 60, 80 and 100-story models. The reduction of axial force in the proposed system, with respect to the M-S system, will definitely lead to the reduction of the base plate and foundation dimensions, which is important from the perspective of structure economics and work execution issues. In addition, by reducing the axial force in a column, smaller sections can be used.



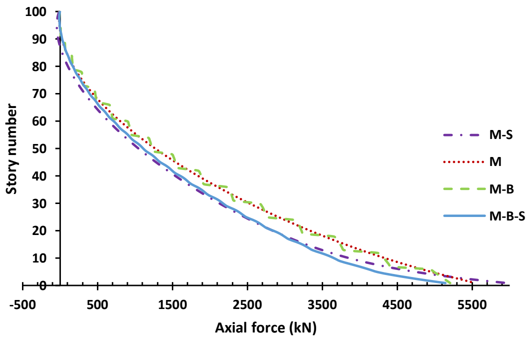
a .40- multistorey model frames



b. 60- multistorey model frames



c. 80- multistorey model frames



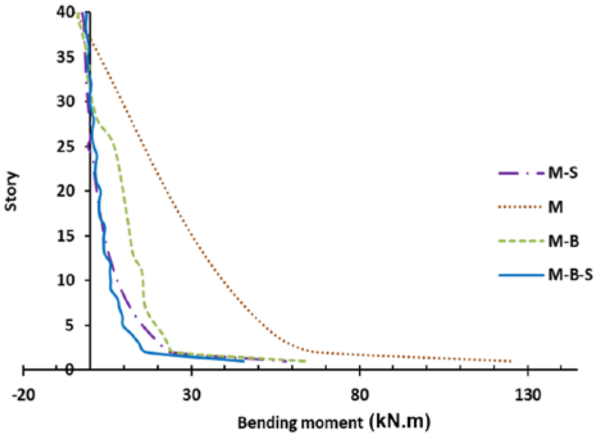
d. 100- multistorey model frames

Figure 13. Maximum axial force in each corner column.

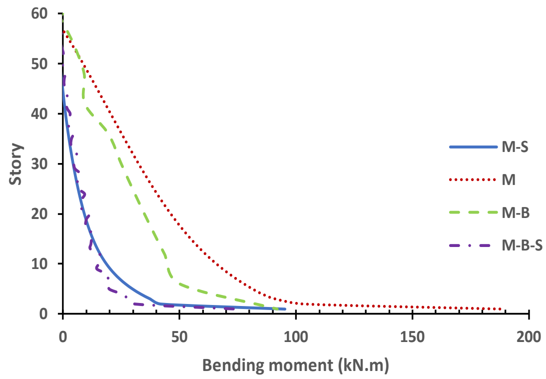
3.1.3 Bending moment and shear force in columns

Besides the axial force, the bending moment and shear force are also important in sizing the columns. The shear forces and bending moments produced in the columns of each system have been shown in Fig. 14. The results indicate that, with regards to the bending moment produced in the columns, the proposed system performs better in all the structures (40, 60, 80 and 100 stories). In system M-B, the bending moment rises and falls abruptly; and this case is also true for relative displacements. In all the diagrams, the largest bending moment is generated in the sole bending frame (system M).

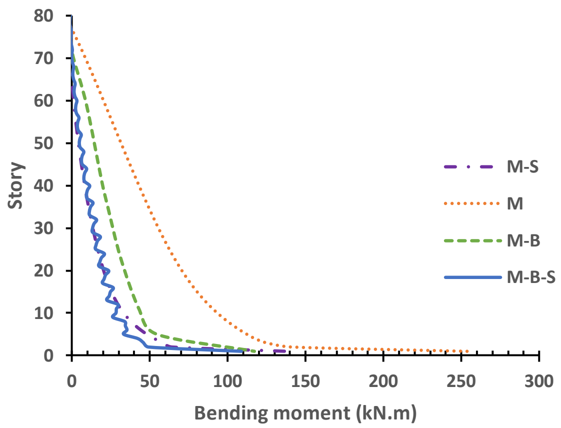
As observed in the Fig. 15, regarding the maximum shear forces produced in the corner columns, the structural performance and the obtained results are the same as those related to the maximum bending moments. So, these results indicate the superiority of the proposed system, which produces the least amount of bending moment and shear force in the columns. This is because the lateral shear is endured by the exterior brace and the shear wall; and working together, they eliminate the irregularity and the sudden fluctuations in the bending moment and shear force. Also, since the shear stiffness of the steel plate shear wall is much higher than that of the brace and frame, the wall absorbs more of the lateral shear force and improves the system behavior. Also, the performance of the diagonal tension field of the shear wall causes a more uniform distribution of stresses at the bases of the first story columns and a reduction of the shear lag, because it counteracts the axial forces in those columns.



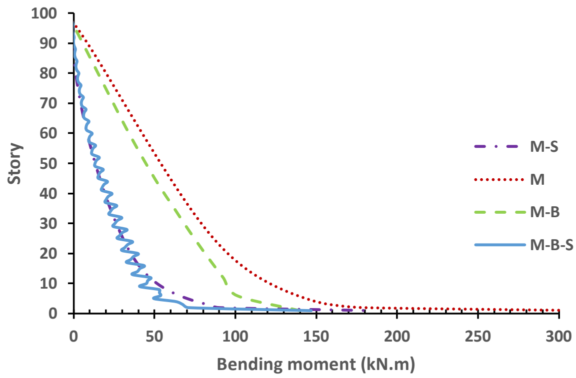
a .40- multistorey model frames



b. 60- multistorey model frames

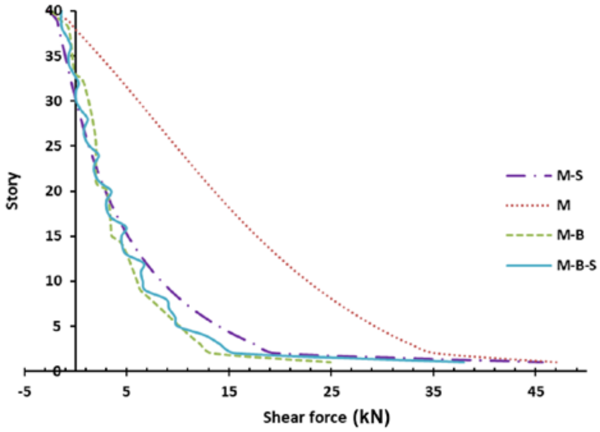


c. 80- multistorey model frames



d. 100- multistorey model frames

Figure 14. Maximum bending moment in each corner column.



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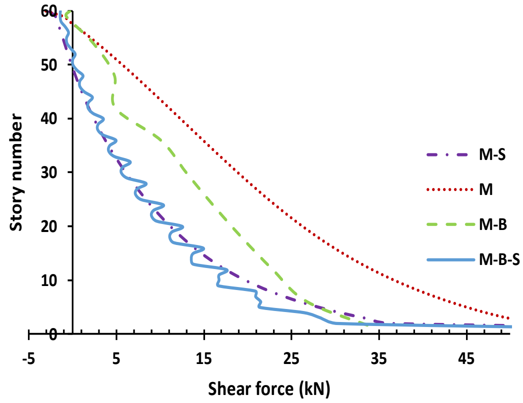
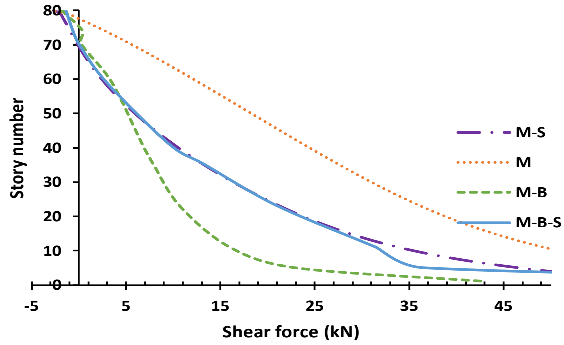
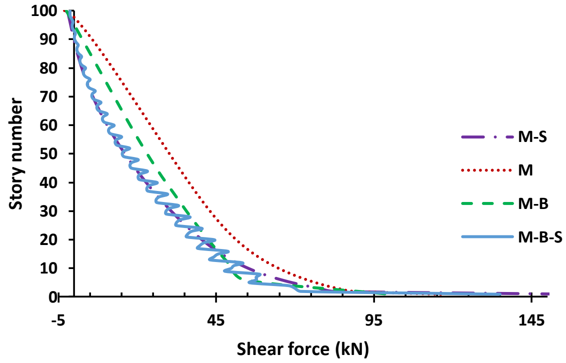


b. 60- multistorey model frames



c. 80- multistorey model frames



d. 100- multistorey model frames

Figure 15. Maximum shear force in each corner column.

3.2. Discussion on results of Series 2

After parametric study of the proposed system in the previous sections, its effectiveness is evaluated when utilized in an actual structure, as introduced in section 4-1. Fig. 16 shows that by applying the proposed concept, the lateral displacements are reduced.

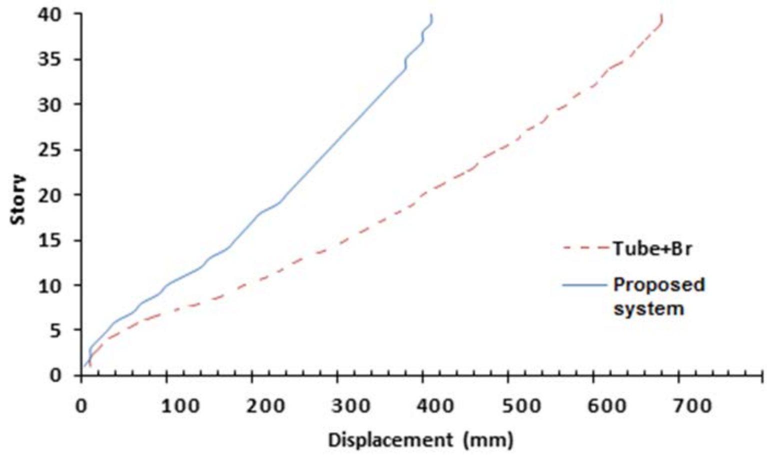


Figure 16. Comparison of displacements of the proposed system with that of the existing structure.

Moreover, as was claimed before, the proposed system reduces the shear lag effect. In Fig. 17 and 18, the axial forces in the columns of the first story have been shown for comparing the shear lag effects. This comparison indicates a more uniform distribution of forces in the proposed system. In other words, in the proposed system, the shear lag effect has diminished considerably, relative to the other two systems.

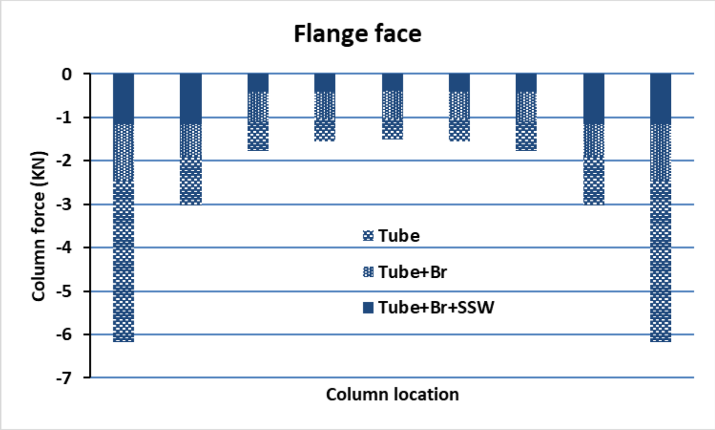


Figure 17. Comparison of shear lag effects in tube system flanges.

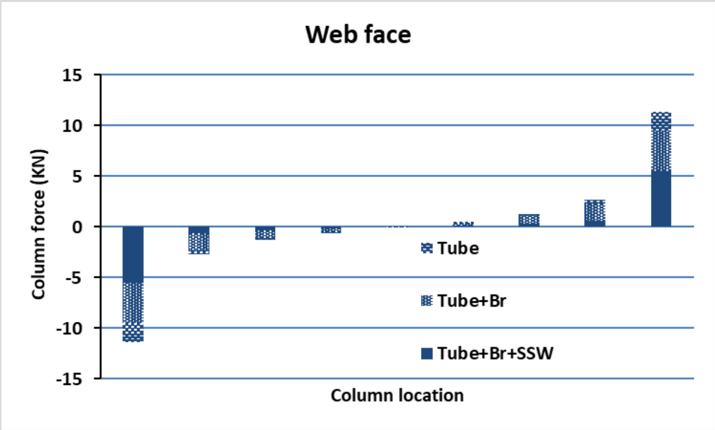


Figure 18.Comparison of shear lag effects in tube system webs.

4. Conclusions

In this paper, the behavior of steel tall buildings employing a combined lateral bracing and steel plate shear wall system was investigated. Combining the SPSW and bracing system allows advantages of both systems regarding increasing the stiffness and strength. The finding can be summarized as follows:

- The combination of tubes, giant braces, and steel plate shear walls was proposed as a new approach that exploits the advantages of all these systems.
- The numerical results indicate that by employing the proposed system, the lateral displacement and drift of the structure diminish by around 2.13 times.
- In addition, due to the high shear stiffness of the SPSW, the wall absorbs a significant portion of the lateral shear; thus, the forces produced in the beams and columns diminish substantially.
- By using the combined system, the maximum bending moment in the columns was reduced by around 50 %.
- Results indicated that by using the combined system, the shear force in the columns was reduced by around 30 %. The effect of the system on structures with lower heights is greater than that on the taller buildings.
- Moreover, in this system, the shear lag may diminish considerably. Therefore, the obtained results indicate that the proposed system can be used as an economical, effective, and safe system.

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