A Study to use an Alternative System of Wall Bracing in Industrial Buildings

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Abstract

There are two main construction systems to be used in industrial buildings to transfer the lateral loads to supports of the structures. The systems may consist of roof with wall bracing or roof bracing with concrete walls. For any structural project the design should take into account all the particular requirements for the specific building. In some industrial buildings, the wall bracing in the steel structures may not be able to locate it in the outer bays similar to the roof bracing, due to architectural requirements of doors and windows. This study demonstrates an alternative method for resisting the lateral loads if the wall bracing cannot be used in the outer bays of the structure. The proposed system can be used to transfer the lateral forces from the roof bracing to the base of the main columns without the need for wall bracing in the external bays of the structure.

Keywords - *lateral load, industrial building, steel portal frame, concrete walls, wall bracing, dampers.*

I. INTRODUCTION

Wind load is the main load effect in the design of industrial buildings, even in low wind areas. It is therefore important to carefully evaluate wind loads. Usually, the end spans are the critical area of wind design. This is because the end spans not only have higher bending moments and higher deflections for a given uniform loads, but also higher loads because external suctions including load pressure effects are highest at the windward end under longitudinal winds. The traditional model for wind truss analysis involves applying the lateral loads as forces at the truss nodal points and calculating the reactions to be resisted by the wall bracing at the ends of the roof truss.

Generally, portal frames resist cross wind forces by in-plane flexure, but longitudinal wind forces acting on the end walls must be transferred via roof bracing to the side walls and thence to the foundations as shown in figure 1.

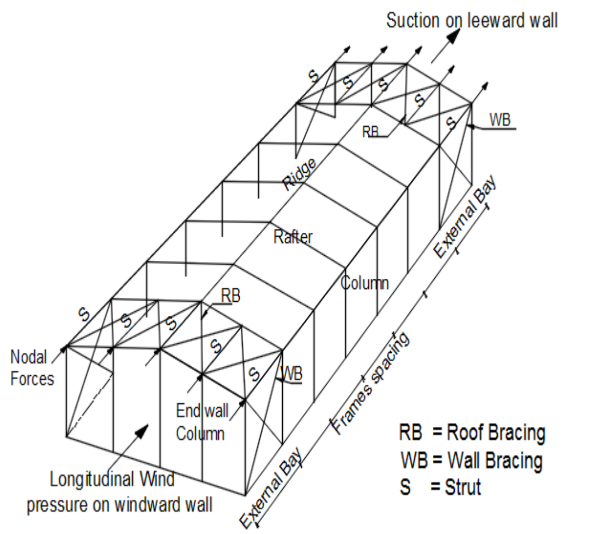


Fig. 1: Roof and wall bracing of steel portal frames

The primary function of a triangulated roof and wall bracing system is to withstand longitudinalwind forces. By means of the bracing system, the forces on the upper half of the end walls, and the frictional drag forces on the roof and side walls, are transferred to the side wall bracing and thence to the foundation.

II. BRACING SYSTEMS

The choice of the roof and wall bracing layout for a building would appear at first thought to be a simple decision. To resist end wall wind loads, the most typical layout is with each end bay braced (option I, figure 2.1). However, there can be detailing difficulties connecting the bracing to the end wall rafter if it is smaller than the typical rafter, or if it is discontinuous at end wall columns. This can be overcome if the second bays from the end are braced (option III), but extra struts will be needed in the end bays to transfer the loads from the end wall columns to the braced bays unless the purlins can double as the struts.

A. Option I: Two end bays braced

This is the simplest and most direct option. Intermediate eaves and ridge struts are usually used however, purlins are sometimes sufficient to brace internal rafters so that no intermediate struts are required. Longitudinal wind loads, as a combination of pressure on the windward wall, suction on the leeward wall and friction, could be shared between braced bays if purlins have the capacity to transfer some compression load from one end to the other.However, the bracing at each end must be designed to resist loads fromexternal pressure and internal suction on the adjacent end wall (plus half of the friction drag forces if applicable).

B. Option II: Double diagonal bracing over two bays at each end

Diagonals intersect at rafters and therefore tubes can be used as diagonals without difficulty if they are not crossed. The number of diagonals is the same as for option I but more struts are required, as shown in figure 2-2.

C. Option III: Second bay from each end braced

This option can overcome any detailing difficulties associated with end bay bracing but extra struts are required to transfer the end wall wind loads to the braced bays unless the purlins can act as struts as shown in figure 2-3.

D. Option IV: One bay braced

Struts in the un-braced bays are required to transfer end wall wind loads to the braced bay which is expensive unless the purlins can act as
struts as shown in figure 2-4.

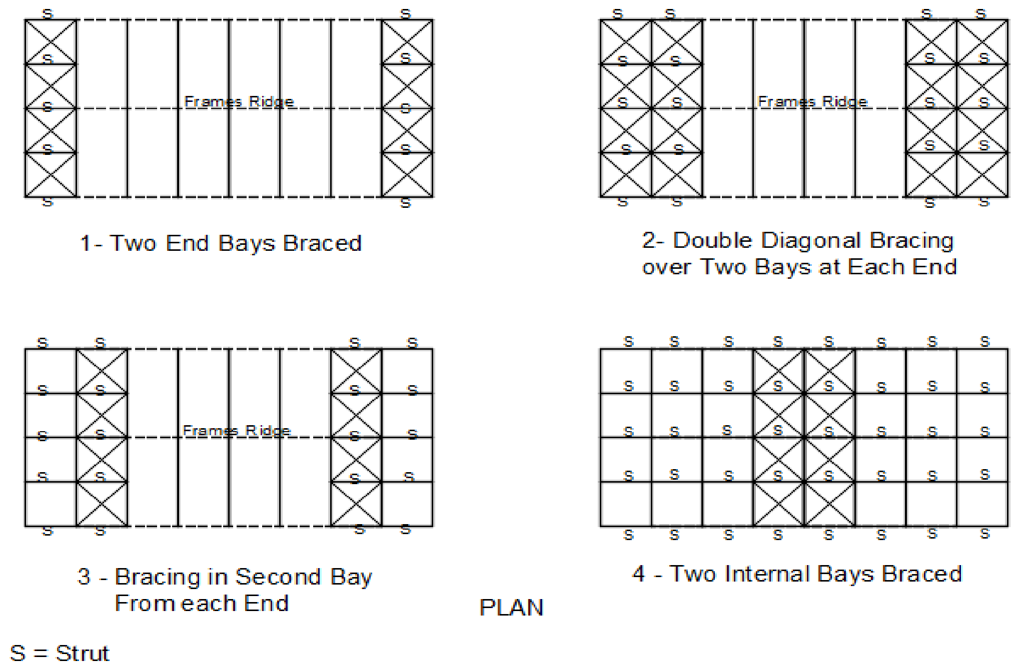


Fig. 2: roof bracing layout options with steel frames.

III. ROOF BRACING AND CONCRETE WALL PANELS

In industrial buildings, using the wall panels as loadbearing elements generally reduces the overall cost due to a reduction in the amount of structural framing required. Although more roof bracing is required, eliminating the columns provides greater saving. In single storeybuildings, the rafter spacing will usually determine the joints coinciding with rafter locations as shown in figure 3.Rafter spacing should be chosen to optimize the design of the roof, purlins, roof sheeting, etc.The concrete panels may be used either as cladding panels or load-bearing panels, i.e. to form part of the building structure.

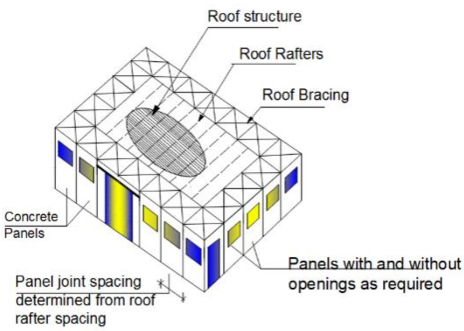


Fig. 3: relationship between roof structure, rafter spacing and panel joint spacing

When panels form part of the building structure, carrying the vertical and lateral loading, the wall must provide a sufficient force-resisting mechanism to carry the applied lateral actions, as shown in figure 4.

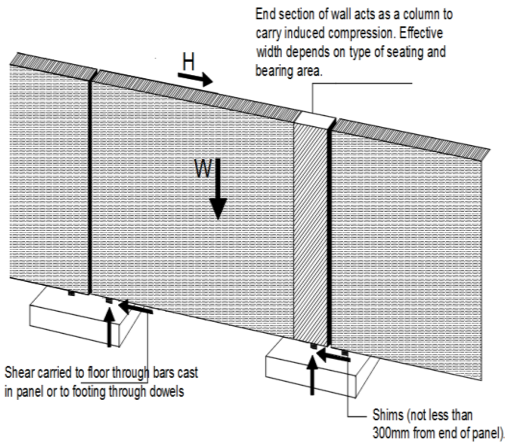
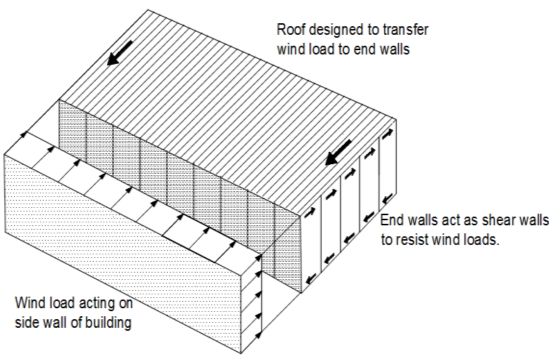


Fig. 4: lateral- force resistance mechanism

Generally, the roof is designed to function as a diaphragm to carry the lateral actions applied onone set of walls to those as right angles. The latter act as shear walls to resist the applied actions as shown in figure 5.Each bay must be braced in order to transfer the lateral loads on the walls to the supporting cross walls as shown in figure 6.

Fig. 5: typical use of concrete panels to resist



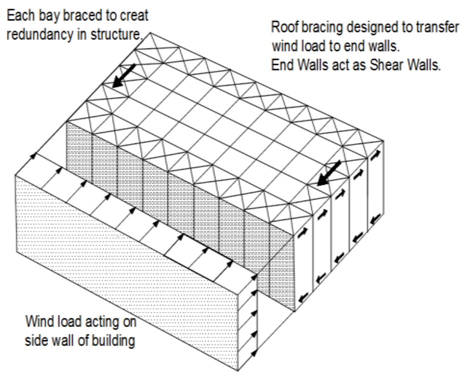


Fig. 6: brace each bay to transfer lateral loads

The traditional model for wind truss analysis involves applying the lateral loads as forces at the truss nodal points and calculating the reactions to be resisted by the shear walls at the ends of the roof truss as shown in figure 7.

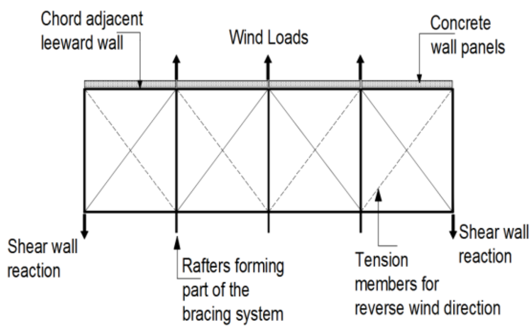


Fig. 7: traditional model for wind truss analysis

In this type of analysis, tension and compression loads in the chord members are generally largest at the centre of the truss. For large panels, the joints between panels are generally located at rafter centres so that the roof framing (or portal frames when the panels are used as cladding only) laterally supports both panels as shown in figure 8.

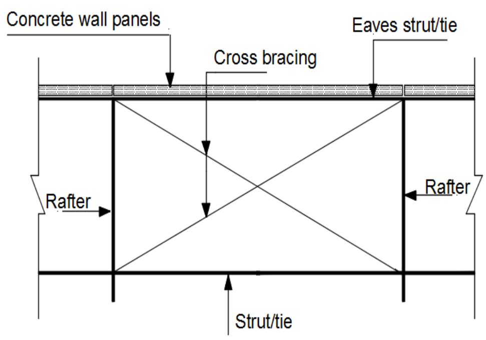


Fig. 8: panel joints located at rafter centre

IV. WALL BRACING OF STEEL FRAMED INDUSTRIAL BUILDINGS

For the roof bracing layout, the bracing at each end should be designed for the longitudinal wind acting on the adjacent end wall due to external pressure and internal suction as shown in figure 9. Half of the total longitudinal drags on the roof and the upper half of the side walls.

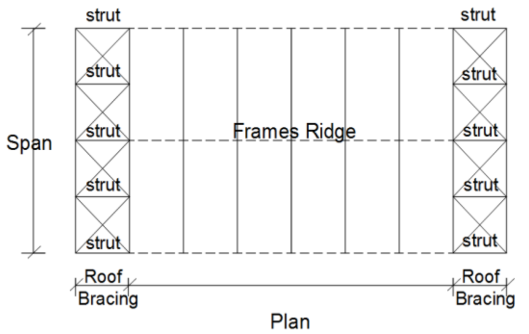


Fig. 9: traditional model for wind truss analysis to the leeward end.

The longitudinal wind forces on both end walls could be shared equally between the two end bracing systems. This would require some of the purlins adjacent to each end wall column to have sufficient capacity in compression to balance any internal suction forces on the end walls, and to transfer some of the force at the more highly loaded windwardend to the leeward end.
Relying on purlins to carry compressive forces from primary loads such as end wall wind loads is not as inherently sound as using a roof bracingsystem which is independent of the roof sheeting.

V. THE MECHANISM OF TRANSLATING THE ROOF BRACING TO THE COLUMN

The longitudinal wind load applied on the structure at the connected joints of the roof as shown in figure 10. The wind loads applied on the lower half transferred to the ground.

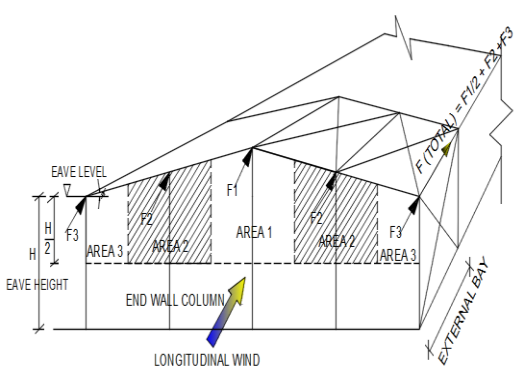


Fig. 10: roof bracing layout in steel framed industrial building.

The affected area can be calculated from the upper mid height of the structure. The sum of each side applied at the eave and transferred to the base of the columns through the wall bracing at the external bay of the building.

The load in the strut at eave of the structure can be calculated as shown in the figure 10. The cross-wind load on the building affected at the connected points of end wall columns to the roof. For example, the affected wind force F_1 = Area A_1 x wind force (including external and internal wind coefficient). In case of the force F_1 affected at the top of the structure, each side of the structure is divided to $F_1/2$. The total force affected in the strut at the eave level is equal to F = $F_1/2+F_2+F_3$.

The roof wind load "F total" at the eave of the structure is transferred to base of the wall bracing at the external bay through the wall bracing at the external bay of the building as shown in figure 11. The load in the tension rod of the wall bracing is equal to F/cos θ, whereθ is the angle of the tension rod on the horizontal.

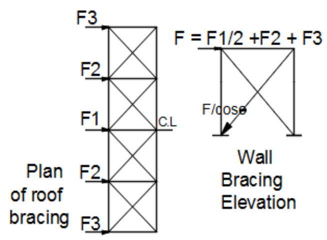


Fig. 11: forces in the wall bracing in the outer bay of the structure.

VI. ARCHITECTED REQUIREMENTS IN INDUSTRIAL BUILDINGS

Fig. 12: internal view of an industrial building

Figure 12, shows an industrial building with architectural requirements of windows and doors at the external bays of the building. In this case, installing wall bracing in these bays is not allowed and another system must be used. It will be difficult to use traditional wall bracing at outer bays because of the existing doors and windows. Moving the wall bracing to the inner bays may be costly because additional struts are required to transfer the loads to the braced bays as shown in figure 13.

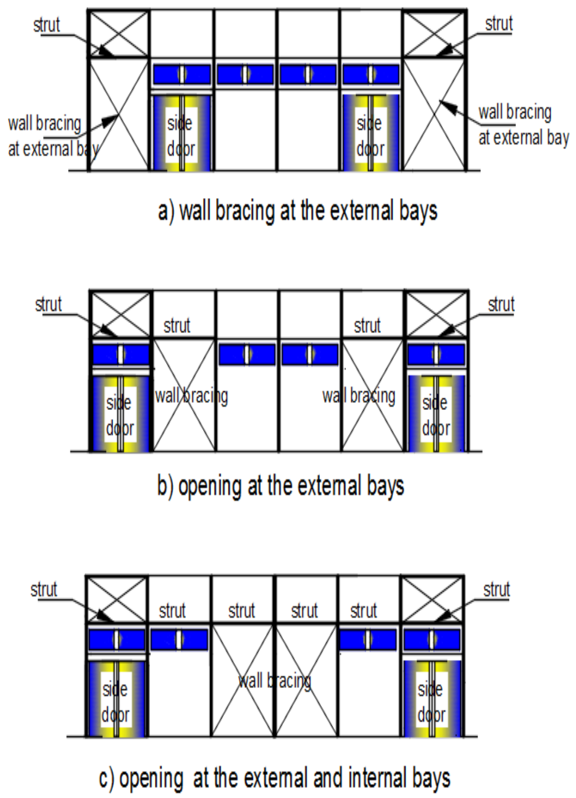


Fig. 13: different locations of opening and wall bracing with struts.

As shown in figure 13, the installation of the wall bracing depends on the location of doors and windows in the side walls of the building. Figure13-a, shows the traditional location of wall bracing in the outer bays. Figure 13-b, shows the movement of the wall bracing to second inner bays and uses the additional struts to move the horizontal forces to the inner wall bracings. More struts should be used to turn the wall bracing into internal bays to avoid existing doors and windows as shown in figure 13-c. Moving the wall bracing location responds to some other requirements for additional struts which is expensive. The solution can be accepted if there are limited opening numbers and there are no doors and windows in the inner bays.

VII. PROPOSEDSYSTEM

An alternative wall bracing system should be used to resist lateral forces applied to the structure.

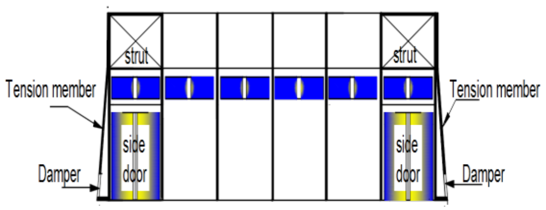


Fig. 14: tension member and damper system

The proposed system that can be used is the "damper-tension system". The system can be placed outside the structure as shown in figure 14. When lateral forces are applied to the roof of the building, the roof bracing system transfers the forces to the struts at the eave level of the structure. The force in the tension member is damped and resisted by the damper fixed at the base of the system.

VIII. EVALUATION THE EFFECT OF THE PROPOSED SYSTEM IN RESISTING THE LATERAL FORCES

The lateral deflection limits are expressed in terms of the column height "*h"* as well as column spacing "*b",* figure 15. For industrial buildings of steel sheeted wall, no ceiling, nointernal partitions against external walls or columns the limits of deflections are h/150 and b/200

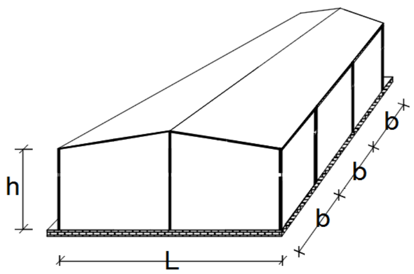


Fig. 15: parameters for deflection limits

For a structure of h =7.5 m eave and column height, with bay spacing b = 9.0m, the lateral force in the strut at the eave height is equal to 73 kN. As shown in figure 16, the deflection limit is the smallest value of h/150 = 50 mm or b/200 = 45 mm. In this case the deflection of the structure must not exceed 45 mm. By using the limit of the structure is equal to 45mm; the force in the tension member and damper can be evaluated using software structural analysis such as ETABS [8] or SAP2000.

By analysing the structural system, it was found the displacement at the eave level was equal to 45mm and the force in the tension member was 380 kN as shown in figure 16. Hence, the force in damper is known and its size can be evaluated.

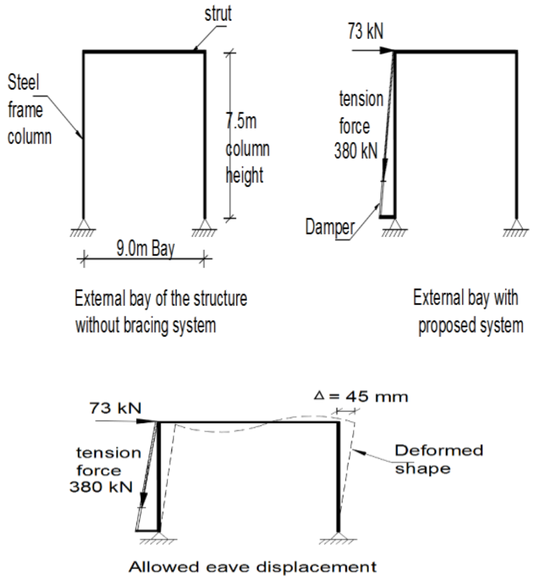


Fig. 16: external bay with and without proposed system and deformed shape of the structure

IX. SIZE OF THE DAMPER

The proposed system used to replace the wall bracing in the case of the existing architectural opening in the side walls of the industrialbuildings consists of two main components. A steel hollow section member is used to transfer the tension forces to the damper to overcome the tension force and thus reduce the lateral displacement of the structure to the permissible values. The damper device which that can be used is called a ring springs damper. The device is a passive energy dissipater based on half-cantering friction mechanism. Ring springs are frictional devices consisting of inner and outer rings that have tapered mating surfaces. As the spring column is loaded in compression or tension, the axial displacement is accompanied by sliding of the rings on the conical friction surfaces. The outer rings are subjected to circumferential tension and the inner rings experience compression as shown in figure 17-a and b.

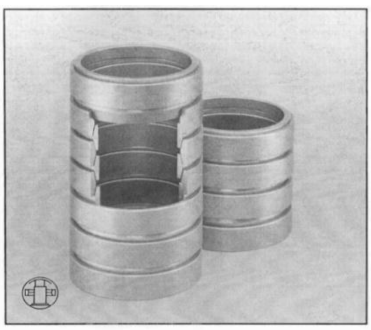


Fig 17-a: prototype ring springs [9]

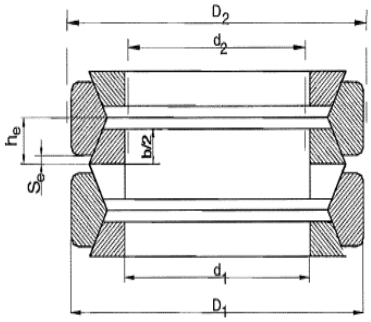


Fig 17-b:details of ring springs

F = spring end forces

Se = spring travel for one element

w_e= energy absorption (work for one element)

h_e = element height

D_1 ,d_1 = outer and inner diameter of the rings

D_2 , d_2= outer and inner diameter of the guide components

b/2= half width of ring

G_e= element weight`

Table1: details of ring springs forces and dimension [9]

F (kN)	s_e (mm)	W_e (j)	h_e (mm)	D_1 (mm)	d_1 (mm)	$b/2$ (mm)	D_2 (mm)	d_2 (mm)	G_e (kg)
350	3.7	648.0	20.0	166.0	134.0	16.0	170.0	130.0	0.822
510	3.9	995.0	22.4	198.0	162.0	18.5	203.0	157.0	1.515

From table 1, it can be observed that the use of external diameter D_2 =170mm and applied load capacity of 350 kN can be used to obtain an accepted displacement. A larger size of D_2= 203mm and a load of 510 kN can be used for a more conservative solution.

X. ARCHITECTURAL REQUIREMENTS FOR THE PROPOSED SYSTEM:

Installing the proposal system outside the industrial building, some architectural requirement may be needed to cover the location of the system. The new system may require about 1.2m width to be fixed outside the structure as shown in figure 18. The architectural cladding is used next to the external bays and on the same line of the building frame columns. Figure 18, shows the elevation and plan view of the structure and the location of the proposed system outside the building.

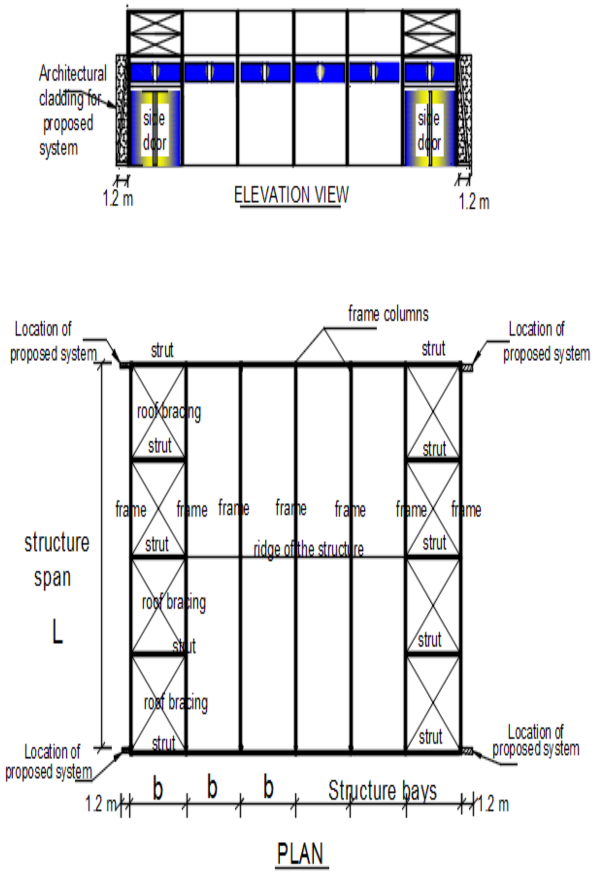


Fig 18: View the location of the proposed system

CONCLUSION

In industrial buildings, the architectural requirements of windows and doors opening of the structure do not allow the structural designer for the placement of the wall bracing in the outer bays to resist the lateral forces applied to the structure. This study shows an alternative system that can be used to transfer the lateral forces from the roof bracing to the base of the main columns of the structure. The traditional methods of transferring the lateral loads to the braced bays of the structure are expensive because the additional struts are required to transfer the loads to the braced bays. The proposed system can beattached outside the external bays and the lateral forces can be moving from the roof bracing at the roof of the structure to the base of the main columns of the structure using the proposed system. The system can be located outside the structure with the width not exceeding 1.2m as described in this study.

REFERENCES

[1] Woolcook S.T, Kitipornchaf S., Bradford M.A, “Design of portal frame buildings”, Australian Steel Institute 2003.
[2] Cement Concrete & Aggregates Australia, “Guide to tilt-up Design and Construction”, Concrete Institute of Australia 2005.
[3] Australia / New Zealand Standards Wind Actions AS/NZS 1170.2: 2002.
[4] Saudi Wind Code – Loads & Forces Requirements, SBC301-2007.
[5] Nagui W. Bishay, Athol J. Carr, “Ring Spring Dampers: Passive Control System for Seismic Protection of Structures”, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 47, No 3, September 2014.
[6] John Holms, Andrew King, A Guide to ASNZS 1170.2:2002 Wind Actions 2005.
[7] MacRac Gregory, Clifton Charles,” Rocking structure design considerations”, Steel Innovations 2013 Workshop, Christchurch, New Zealand.
[8] ETABS Software, 2016, 16.0.03 Enhancements, CSI Computers and Structures. INC.
[9] RINGFEDER GmbH, Friction Springs, Ringfeder in Mech. Eng., Report R60E, Ringfeder Corporation, Germany.
[10] Justin D. Marshall, Finley A. Charney, “A hybrid control device for steel structures” Journal of Constructional Steel Research 66 (2010) 1287-1294.
[11] Amedeo Benavent- Climent, “A brace-type seismic damper based on yielding the walls of hollow structural sections”, Engineering Structures 32 (2010) 1113-1122.
[12] H.-L. Hsu, H. Halim, “Improving seismic performance of framed structures with steel curved dampers”, Engineering Structures 130 (2017) 99-111.
[13] Sang-Hoon Oh, Youn-Ju Kim, Hong-Sik Ryu, “Seismic performance of steel structures with slit dampers”, Engineering Structures 31 (2009) 1997-2008.
[14] VajreshwariUmachagi, Katta Venkataramana, G. R. Reddy, “Application of dampers for vibration control of structures”, International Journal of Research in Engineering and Technology 2319-1163.
[15] Chandrashekhar B Adin, Praveen J. V, “Dynamic Analysis of Industrial Steel Structures by using Bracing and Dampers Under Wind and Earthquake Load”, International Journal of Engineering Research & Technology, Vol 5. Issue 07, July - 2016.
[16] S.M Gledhill, G.K.Sidwell,” The Damage Avoidance Design o Tall Steel Frame Building”, New Zealand Society for Earthquake Engineering, 2008 NZSEE Conference.