Journal of Constructional Steel Research 148 (2018) 138–153

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Elastic and elastic-plastic threshold stiffness of stiffened steel plate walls in compression

Zhaoyu Xu [1], Genshu Tong [*,2], Lei Zhang [3]

Department of Civil Engineering, Zhejiang University, Hangzhou, Zhejiang 310058, China

ARTICLE INFO

Article history:
Received 25 October 2017
Received in revised form 14 March 2018
Accepted 10 May 2018
Available online xxxx

Keywords:
Stiffened plate
Vertical stiffeners
Threshold stiffness
Elastic-plastic stability
Post-buckling

ABSTRACT

This paper studies the minimum stiffness required to ensure elastic buckling and elastic-plastic post-buckling strength of subpanels of vertically stiffened steel plate walls (S-SPWs) under compression. In the linear elastic analysis, the threshold stiffness at which the buckling mode of S-SPWs changes from overall to subpanel local buckling is determined. Based on the understanding that the increased elastic critical buckling strength of the S-SPW is provided by the elastic buckling resistant capacity of the stiffeners, a formula is proposed to predict the elastic threshold stiffness.

The elastic-plastic threshold stiffness of vertical stiffeners, which makes the subpanels develop their full elastic-plastic post-buckling capacity, is obtained through nonlinear analysis. This paper investigated the effects of subpanel aspect ratio, subpanel width-to-thickness ratio, number of stiffeners on the elastic-plastic threshold stiffness. The effect of initial imperfection is also included. Based on the understanding that the increased capacity of the S-SPW from overall to subpanel post-buckling is provided by the elastic-plastic strength and stiffness of the stiffeners, a formula to predict the elastic-plastic threshold stiffness is proposed. The proposed formulas for both threshold stiffness are found to possess good accuracy after some minor modifications. The comparison between elastic and elastic-plastic threshold stiffness is also presented.

1. Introduction

Steel plate wall (SPW) is gaining increasing popularity as a lateral load resisting system due to its ductile behaviour, simple assembly, quick construction and floor space saving. Two types of SPWs are applied in practice, one is the unstiffened SPWs which are applied mainly in low-rise or residential buildings, the other is the stiffened SPWs (will be denoted as S-SPW) used in high-rise buildings. As reported by Astaneh-Asl [1] and Sabelli & Bruneau [2], the first building using S-SPWs was the Nippon Steel Company, a 20-storey office building completed in 1970.

In 2008, the second author designed a 46-storey office building of height 187.7 m in which S-SPWs were used in the core. The building is located at Kunming, a high intensity seismic area of China. It was put into service in 2014. The plan of the building is given in Fig. 1a, some details for the bottom of the S-SPWs are given in Fig. 1b and the elevation of S-SPWs is given in Fig. 1c. The S-SPWs were used in this building to save space and increase the width of the elevator corridor.

In the design of the SPWs for this building, it was found that great vertical compression stresses (about 40% of the yield strength f_y of Q345 steel (345 MPa)) were present in the SPWs under the gravity load combination. Such stresses could be reduced by only about 10% through postponed final fixing. The final fixing of SPWs, welding the bottom edge of SPWs onto the top flange of the beams, is normally permitted before the erection of the above 20 floors, otherwise, extra costs may be necessary due to additional erection time consuming. Therefore, the vertical stiffeners are usually used to prevent the SPWs from the compressive buckling in the service limit state.

Another finding is that the shear demand is not the dominating factor in designing such a high-rise building. Just as a steel beam is rarely controlled by shear strength of its web, S-SPWs in high-rise buildings are seldom governed by shear demand. One can imagine that a high-rise building of 50-storey with a 40m × 40m plan, the gravity load is about $40\text{m} \times 40\text{m} \times 8\text{kN/m}^2 \times 50 = 64 \times 10^4\text{kN}$ and the estimated period is $0.12 \times 50 = 6\text{sec}$. In an area of maximum ground acceleration of 0.2 g (according to the return period 475 years), the design base shear is about $0.032 \times 64 \times 10^4 = 20480\text{kN}$. Assuming that at least three 15 m long S-SPWs with thickness of 16 mm are used, the average shear stress in the S-SPWs is only 28.44 MPa, far less than the shear yield strength of material (199 MPa for Q345 steel).

Compared with braces, the authors of present paper found that if an S-SPW is allowed to carry vertical loads and overturning moments, the

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* Corresponding author.
E-mail address: tonggs@zju.edu.cn (G. Tong).
[1] PhD Student in Structural Engineering.
[2] Professor in Structural Engineering.
[3] Associate Professor in Structural Engineering.

https://doi.org/10.1016/j.jcsr.2018.05.014

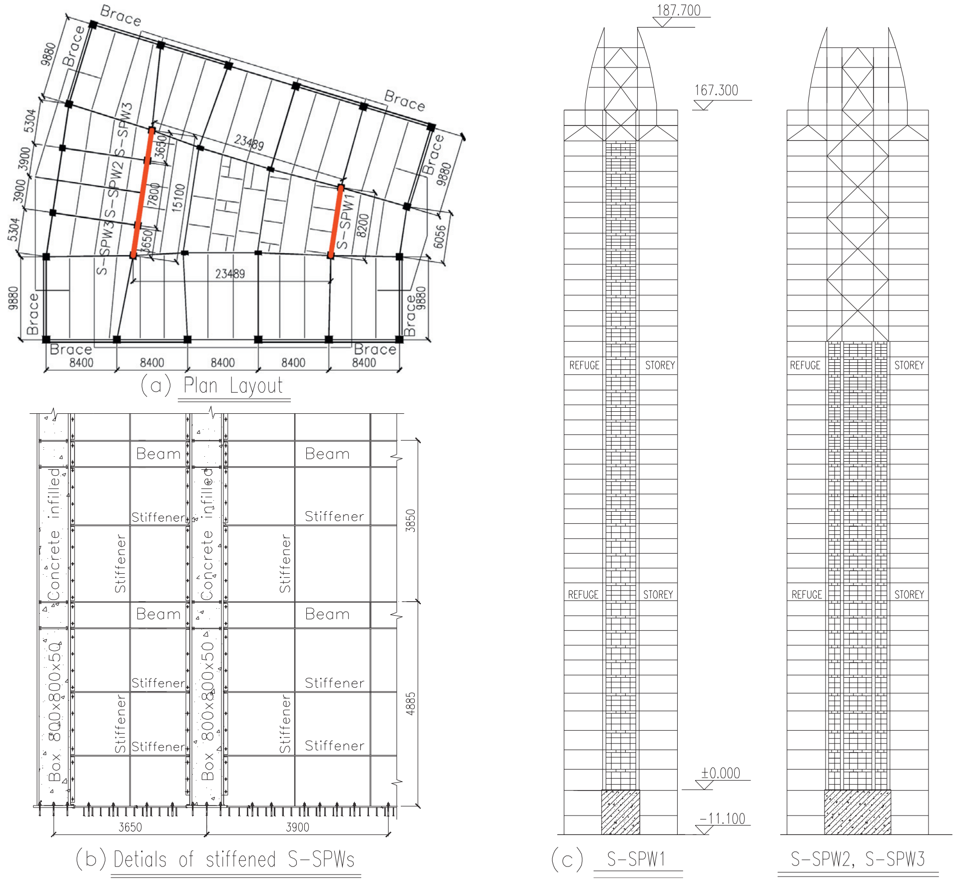

Fig. 1. a) Building plan, b) details of the S-SPWs, c) elevation of the S-SPWs.

elastic properties of the whole building can be improved. The reason is that the braces provide only shear stiffness for the building, while the S-SPWs also provide the bending stiffness. With a proper design procedure (i.e. not reducing the boundary columns' size even if the S-SPWs contribute in carrying vertical load) and the constraining effect of the vertical boundary elements (columns), the ductility of such a S-SPW might be ensured. Nie et al. [3] reported a test on a S-SPW with the gravity load ratio (=vertical load over the sum of columns' and S-SPW's yield load) 0.33 and found excellent hysteretic behaviour, which indicated the potential for wide application of S-SPWs in high-rise buildings. In the wind-controlled design of Hyatt Regency Hotel in Dallas (completed in 1978), S-SPWs were even used to carry vertical load to reduce the column size [2].

In many cases, the required stiffness on stiffener to keep compressive buckling within subpanels of the S-SPW is larger than the requirement for shear buckling. Stiffeners can mitigate the unfavourable effect of vertical stress on the shear-resistant capacity and the ductility. However, investigations on the S-SPWs under compression are currently very limited.

Elastic buckling of plates stiffened by 1 or 2 vertical stiffeners under compressive load was studied by Bleich [4] analytically and by Timoshenko & Gere [5] through energy method. These studies focused mainly on long plates which find wide applications in steel box bridges and ships. The first approximation (solution by taking only the first term of the double trigonometric series) of Timoshenko's [5] analytical solution is accurate for long plate with overall width-to-length ratio <0.5. However, solutions become complicated for shorter plates since more terms are involved in the buckling wave assumption.

Yoo et al. [6] and Thang et al. [7] investigated elastic stiffness requirements for box-girder flanges longitudinally stiffened with T-shape and flat-bar stiffeners respectively. Choi et al. [8] did further experimental study on the solution proposed by Yoo et al. [6]. The results showed that the ultimate strength of stiffened plates reached the anti-symmetric buckling strength. Kwon & Park [9] studied the behaviour of longitudinally stiffened plate undergoing distortional buckling.

The studies mentioned above are limited to the relatively long plates which are commonly used in bridge girders. Both plates and stiffeners

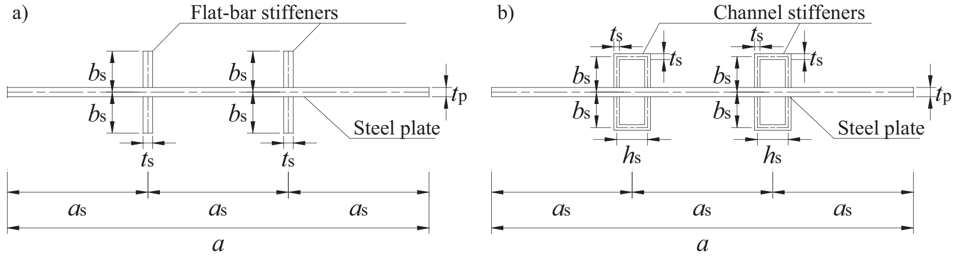

Fig. 2. Notation of stiffened steel plate walls with: a) flat-bar stiffeners, b) channel stiffeners.