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Ultimate strength of expanded metal panels subjected to shear loading

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Abstract
In current design, Steel Plate Shear Walls (SPSWs) are composed of framing members (column and beams) and infills thin steel plates. In recent years its use has increased significantly, leading to the development of different design alternatives. Usually the strength of the SPSWs relies on tension field action of the infill plates, and their collapsibility is exploited to avoid the transfer of excessive forces to the framing members. There is trend in steel design to weaken the infill plates by slitting or perforated them, using low-yield carbon steel or reducing the plates thickness considerably. In this regard, expanded metal panel satisfy these requirements. In the manufacturing process of expanded metal sheets, a steel coil is slit and stretched forming a mesh with diamond-like patterns. Experimental and numerical studies have shown the suitability of the expanded metal panels as possible infills for SPSWs. Under shear loading, expanded metal panels exhibit also tension field action. In spite of the similarity with steel plates, the mechanism for expanded metal is transformed into a bending mechanism due to the geometry of the expanded metal cells. This paper presents a mechanism model for the determination of ultimate strength of expanded metal panel subjected to shear loading.

1. Introduction
Expanded metal panels (EMPs) are manufactured through a process based upon the in-line expansion of partially slit thin metal sheets, producing a diamond-like cell pattern (EMMA 2012). In practice, EMPs have been employed mainly for decorative and protective purposes; hence the panels are usually fabricated using low-yield carbon steel. Recently, some research projects have been conducted to investigate the suitability of EMPs as infill plates in steel plate shear walls (SPSWs) (Dung and Plumier 2010, Dung 2011, Graciano et al. 2015, Graciano et al. 2016).

In modern steel construction, SPSWs are employed to provide lateral load strength to structural frames to withstand load conditions as those exerted by wind and earthquake actions. Basically, SPSWs are built-up members composed of a robust frame and infill plates, whose strength relies significantly on tension field action in the infill. Properly designed SPSWs has high ductility,

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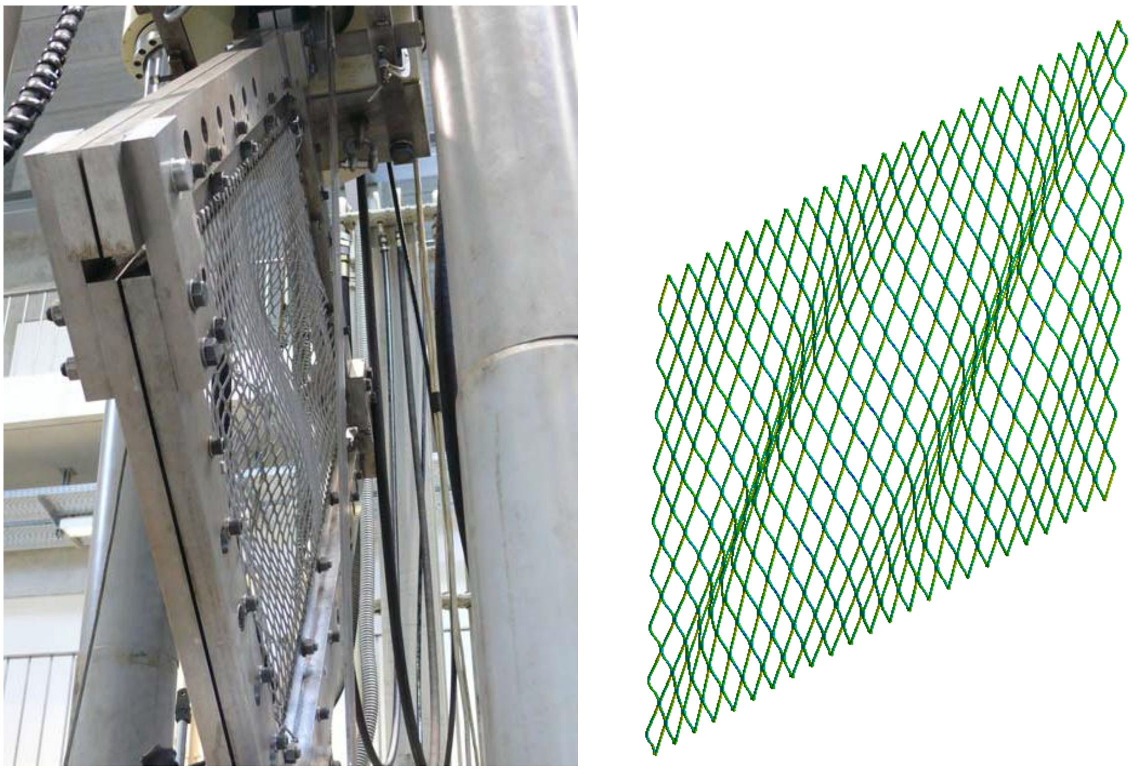
high initial stiffness, high redundancy, and excellent energy absorption capacity in comparison to other conventional lateral load resisting systems. SPWs are also lighter and more ductile than reinforced concrete shear walls and they are relatively easy to install (Bhowmick 2014).

An adequate design of SPSWs requires a proper assessment of both the strength of infill plates and that of the framing members, to avoid the introduction of excessive forces that may increase column demand in the surrounding frame members (Bhowmick 2009). Usually, the framing members are designed to work within the elastic range. Moreover, infill steel plates are designed to rely on the development of tension field action providing postbuckling strength. In order to reduce the transmitted forces to the frame members the infill plates are weakened through perforations (Bhowmick 2009, Bhowmick 2014, Bhowmick and Grondin 2014, Vian and Bruneau 2005, Ergorova 2014) or slits (Hitaka and Matsui 2003, Corte and Liu 2011), using low-yield carbon steel (Chen and Jhang 2011) or using very thin plates (Berman and Bruneau 2003, 2005).



(a) (b)

Figure 1: Deformed expanded metal panels (a) Experimental (Dung 2011); (b) Numerical (Teixeira et al. 2015)

As mentioned above, EMPs are manufactured using low-yield carbon steel, and thin-plates that are stretched and slit, creating a perforated panel. In this manner, EMPs can be a suitable alternative for infill plates in SPWs. In this regards, Dung (2011) conducted an investigation aimed at finding an application for expanded metal meshes for seismically retrofitting of reinforced concrete moment resisting frames. A complete study was performed on pure shear behavior of expanded metal meshes subjected to monotonic and quasi-static cyclic loading, using experimental, theoretical and numerical approaches. Teixeira et al. (2016) performed an extensive numerical study on the shear response of expanded metal panels. The results show that the shear response relied on tension field action (Fig. 1), and that the ultimate strength depending mainly of cell geometry (size and orientation) and panel length. In addition, it was also observed that the cells along the diagonal of the panel undergo a local deformation mechanism which leads to the plastic bending collapse of the cells and a stable load-drift response of the full panel.

This paper presents an analytical expression for the ultimate shear strength of expanded metal panels. Previous studies (Dung and Plumier 2010, Dung 2011, Teixeira et al. 2016) demonstrated that the expanded metal panels develop tension field action, but due to the geometry of the meshes this mechanism is transformed into a plastic bending failure. Then, by using classical beam theory an expression for the ultimate strength is proposed.

2. Proposed model

Fig. 2 shows a schematic view of an expanded metal cell composed by strands and nodes. The geometry of the pattern is mainly characterized by two orthogonal axes: L_1 is the major axis and L_2 the minor, t is the strand thickness, and w is the strand width. The cell are oriented horizontally (α=0°) such as the one shown in Fig. 2.

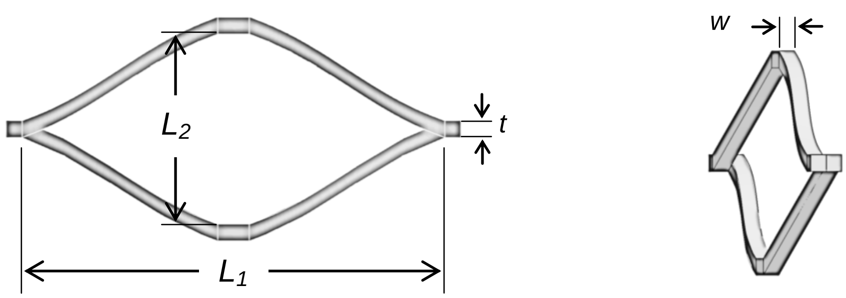


Figure 2: Expanded metal cell (Nomenclature).

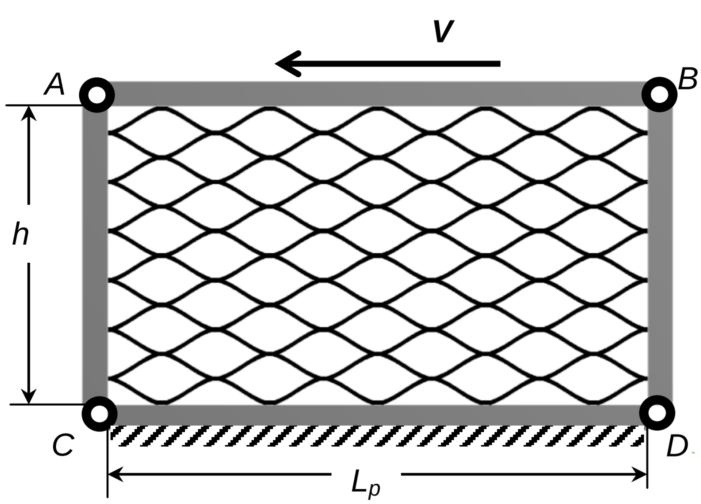


Figure 3: Expanded metal panel (Nomenclature).

A schematic view of an expanded metal panel subjected to a shear load V is shown in Fig. 3. The panels are composed of a number of cells, the panel height h depends on a number of rows N_r in the vertical direction, and the panel length L_p is proportional to the number of columns N_c according to

$$h = N_r(L_2 + 2t) + t \tag{1}$$

$$L_p = N_c(L_1 + 2w) \tag{2}$$

In Fig. 3, the expanded metal panel is hinged in all four corners, and the border CD is clamped. Then, after applying the force V, the panel ABCD is subjected to pure shear. Fig.4 shows the deformation of a single cell

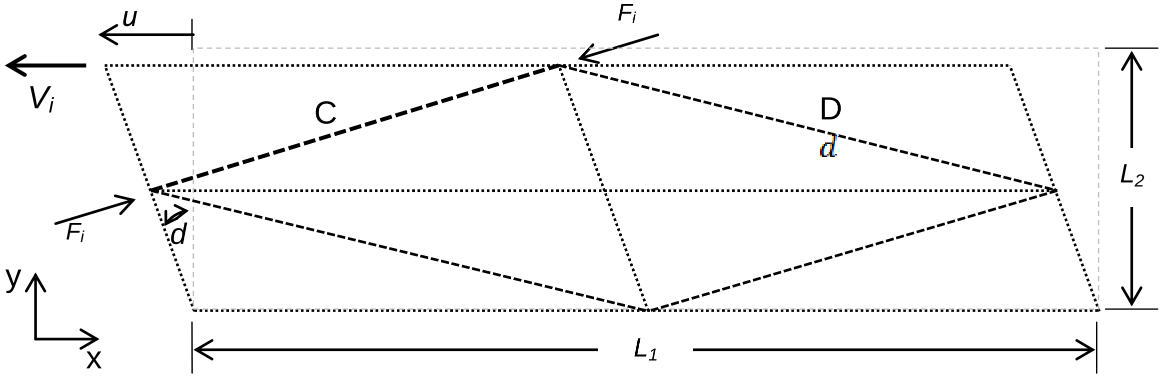


Figure 4: Deformed cell after applying a force V_i.

In Fig. 4, the load V_i causes a lateral displacement u, while the initially right angle deforms a certain amount equal to the drift angle d. This load V_i also exerts a compressive load F_i on strand C, which corresponds to the loaded diagonal in the full panel ought to tension field action. On the opposite side, the corresponding strand D withstands a tensile force.

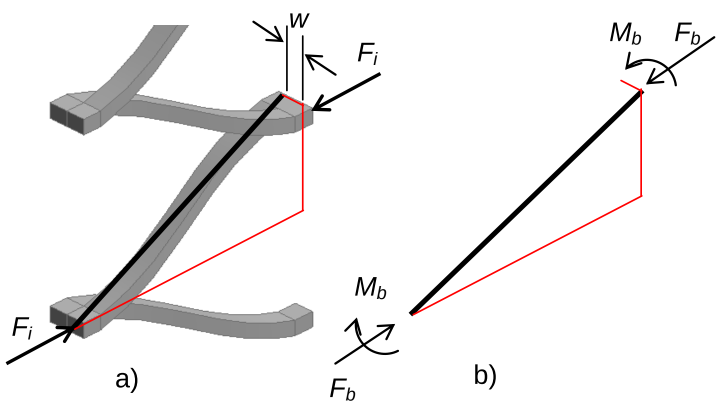


Figure 5: Free body diagram of an expanded metal strand.

Assuming small deformations, the axial stretching of the strand ε_a is approximated to

$$\varepsilon_a = \frac{\sqrt{L^2 - 2\,L_1\,L_2 \sin d}}{L^2} \cong \frac{d\,L_1\,L_2}{4L^2} \tag{3}$$

Eq. (3) is useful to relate the drift to the lateral displacement and cell stretching. Equating the work done by the shear load V_i to the work done by the axial force F_i, the axial force F_i is obtained in Eq. (4)

$$V_i \frac{L_2}{2} d = F_i \frac{L_1\,L_2}{4L} d \tag{4}$$

Solving Eq. (4), the axial force F_i is

$$F_i = 2V_i \frac{L}{L_1} \tag{5}$$

After solving the kinematics for the cell in Fig. 4, and looking at the free body diagram of a cell strand as shown in Fig. 5, the load F_i acting on the cell strand has an eccentricity given by the expanded metal fabrication pattern that is proportional to the strand width w. This eccentricity, generates a bending moment M_b (=F_b w) in the cell strand, in addition to the compression force F_i= F_b.

For a given load F_b acting on a cell strand and considering first-order bending theory for the bending stresses a limit load analysis at cross section is developed. The superposition of axial stresses with bending stresses moves the neutral axis a distance b, which can be calculated solving the stress profile at the cross section by means of Eq. (6)

$$\sigma = \frac{M_b y}{I} + \frac{F_b}{A} \tag{6}$$

Solving Eq. (6) for σ=0, the distance b is calculated in Eq. (7)

$$\frac{F_b w b}{(tw^3/12)} + \frac{F_b}{wt} = 0 \Rightarrow b = \frac{w}{12} \tag{7}$$

Next, for the fully plastic condition, the yielding stress S_y is attained in the portion of the cross section delimited by [+b ,-b], then the plastic collapse load F_c gives

$$F_c = 2b\,tS_y \tag{8}$$

The plastic collapse load F_c is related to the shear load V_i by means of Eq. (5). Therefore, the collapse load of a cell strand is

$$V_i = \frac{2(w/12)\,t\,L_1}{2L} S_y \qquad (9)$$

Multiplying V_i in Eq. (9) by the number of columns N_c, the panel collapse load V_C can be approximated to

$$V_C = 4N_c\left(\frac{w\,t\,L_1}{12L} S_y\right) \qquad (10)$$

At the onset of yielding, the yielding stress is achieved at the outer fibers under compression. The outer fiber is at $y{=}w$, and solving the stress equation setting the stress to S_y

$$\frac{F_b w^2}{(tw^3/12)} + \frac{F_b}{wt} = S_y \qquad (11)$$

Finally, the expression for the ultimate strength of the panel V_u is obtained to approximate the lower bound of the panel yield load

$$V_u = 4N_c\left(\frac{w\,t\,L_1}{13L} S_y\right) \qquad (12)$$

3. Validation of the theoretical model

Teixeira et al. (2016) conducted an extensive numerical analysis to study the influence of various geometrical parameters on the shear response of expanded metal panels. The numerical models were validated using the experimental results obtained by Dung (2011). Table 1 shows the dimensions of the three expanded metal cells used by Teixeira et al. (2916), namely A, B, C. These geometries are also used to validate the theoretical model proposed herein.

Table 1. Dimensions of the expanded metal cells.

Type	L_1 (mm)	L_2(mm)	w (mm)	t (mm)
A	80	36	3.2	3
B	58	28	5.2	5
C	85	20.6	7.65	6.35

Table 2 shows the results for 18 expanded metal panels examined previously by Teixeira et al. (2016) with the cells are oriented at α=0°, *i.e.* panels EMS A0, EMS B0 and EMS C0. In the results reported in Table 2, a good agreement between numerical V_u^{NUM}, and predicted values $V_u^{Eq.(12)}$ obtained with Eq. (12) for the ultimate shear strengths is observed.

The dependence of panel height or the number of rows is neglected in the formulation. Expanded metal panels undergo a local deformation mechanism which leads to cell plastic collapse. The corresponding ultimate shear strength only depends of the number of columns N_c or panel length L_p, cell geometry and material properties.

Table 2. Dimensions and numerical results for EMS panels with α=0° and α=90°.

Panel	N_c	N_r	L_p (mm)	h (mm)	V_u^{NUM}	$V_u^{Eq.(12)}$
EMS A0	8	10	688	395.2	8.64	10.54
		12		473.6	8.54	10.54
		14		552	8.48	10.54
	16	20	1376	787.2	16.17	21.09
		24		944	16.12	21.09
		28		1100.8	16.13	21.09
EMS B0	8	10	547.2	335	27.33	28.45
		12		401	27.24	28.45
		14		467	27.20	28.45
	16	20		665	52.64	56.90
		24	1094.4	797	50.14	56.90
		28		929	53.63	56.90
EMS C0	8	10	802.4	275.85	62.01	57.24
		12		329.75	60.17	57.24
		14		383.65	62.91	57.24
	16	20	1604.8	545.35	118.58	114.47
		24		653.15	118.59	114.47
		28		760.95	118.69	114.47

4. Conclusions

An analytical expression for the ultimate strength of expanded metal panels subjected to shear loading was developed herein. A simple model was developed using basic solid mechanics principles. Based on the failure mode observed both experimentally and numerically, it was possible to define a bending mechanism for the individual metal cells when the whole panels were subjected to pure shear. The results showed a close correlation with other results available in the literature. Moreover, these results also show a step forward for the suitability of expanded metal panels to improve the performance of steel shear walls.

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