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Determination of Lateral Force on Steel Plate Shear Wall by using European Code

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ABSTRACT: Earthquake seems to be the vibration of the surface of the earth by the shocks originating from those in the center of the earth's disruption through the energy release from the top of the earth. Earthquakes can be divided into two categories depending upon their origin *viz*. tectonic and volcanic. Tectonic earthquakes are associated with the sudden dislocation of large rock masses along the geological fault. Volcanic earthquakes are those associated with volcanic eruptions and have limited field. Currently, various kinds of lateral load resisting system among the structural engineers through select among. Every building system are separate with several considerations take over play when choosing the most applicable building structure lateral load resisting system. Steel plate shear wall (SPSW) was often advised effective for lateral load resisting system (LLRS) together with mid – rise structures to high – rise structures. Steel plate shear wall may be adopted as a main lateral load resisting system or as a secondary lateral load resisting system (SLLRS) usually substituting the traditional cement concrete shear wall importance in the high – rise systems. The steel plate shear wall framework also become adopted from 1970s, device use has been increasing steadily for the last several decades.

A steel plate shear wall is a lateral load resisting system comprises of a steel thin web – plate enclosed along and connected to a assisting device. The framework columns are known as vertical boundary elements (VBEs) and nearby beams are known as horizontal boundary elements (HBEs).

Challenge of the study is to form a stepwise procedure to determine the lateral force and the base shear by using the European code.

Keywords: Base Shear, Design Ground Acceleration, Fundamental Period of Vibration, Lateral Force, Steel Plate Shear Wall, Storey Forces.

Abbreviations: HBEs, Horizontal Boundary Elements; LLRS, Lateral Load Resisting System; SLLRS, Secondary Lateral Load Resisting System; SPSW, Steel Plate Shear Wall; VBEs, Horizontal Boundary Elements.

I. INTRODUCTION

Cold – Formed Steel (CFS) systems has indeed built across the ages through form the essential structural mechanism for low – rise structures and medium – rise structures owing with its benefits of minimal cost, rapid assembly, simple transport and manufacturing, high strength, un – combustibility, *etc*. The lateral strength resistance mechanism in CFS structures is usually built for CFS frames sheathed through flat plain steel plates or wood based blocks just for an Oriented Strand Boards (OSBs) and plywood boards. All shear walls of steel with more strong ductility and strength are therefore full required in the low – mid – rise CFS structures, especially in the seismic places [1].

In earthquake design implementations, the principal energy depleting features of steel plate shear walls (SPSWs) resilient to lateral forces are un – stiffened plates infill (webs) that mostly bends by shear also generate a succession of tension field actions (TFAs) diagonally. Across a design by capacity approach point of view, the tensile load of the plates infill should always live controlled by the horizontal and vertical boundary elements (HBEs and VBEs) enclosing the plates.

While rigid links are identified in both the HBEs with the VBEs, and in – between the VBEs together with perhaps the surface (as defined in several SPSW implementations), the SPSWs often gain from the moment of resistance of the border wall frame to a lateral forces attributed [11].

Evaluation of propensity for failure with seismic efficiency is performed being steel plate shear walls (SPSWs) with plates infill constructed according to two various approaches. The present evaluation endure already performed approaching SPSWs analysis and designed to ignore involvement about in its own border moment actively opposing steel frames in order to combat the powers of story shearing. This evaluation of the propensity for failure was replicated for SPSWs that have already been designed to share the forces of story shearing between boundary frameworks and infill sheets. Depending on those other measurements, for both groups, the seismic efficiency parameters [i.e., the over – strength of the system Ω_0, the coefficient of response adjustment (R – factor), and the deflection amplification factor C_d] [12].

The advantage of the proposed stepwise procedure may be useful for the determination of the lateral force and the base shear.

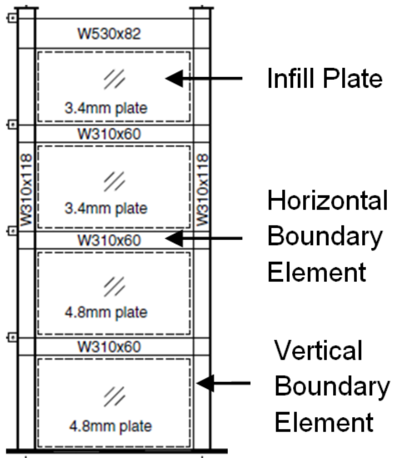

Fig. 1. Steel Plate Shear Wall [13].

II. LITERATURE REVIEW

Nava and Serrette (2015) studied the aspect ratio (height – to – width ratio) less than 4:1, and greater than 2:1, depending upon the vertical wall aspect ratio requires decrement in an nominal strength for design provision of Cold – Formed Steel (CFS), light gauge frame shear walls. Flexibility of wall is increased due to the reduction in nominal strength. The proposed reduction in strength was no more endorsed in reach available data moreover do no more given the justification the lateral movements requirements of building code another available expression for estimating wall movement. Therefore, it is not beneficial to greatly underestimate nominal or peak strength when capacity – based architecture is adopted. The research explores the output of a laboratory experimental system with a 2:3 and 8:1 aspect ratios comprising oriented strand board (OSB) of 11 mm thick sheathed Cold – Formed Steel Shear Walls [9].

Purba & Bruneau (2015) the earthquake achievement of Steel Plate Shear Walls (SPSWs) with plates infill were analyzed and designed for withstanding distinct amount based on effective loads adapted was investigated. Development of component strength is described in study for deterioration models necessary to perform Steel Plate Shear Walls (SPSWs) collapse assessment, focusing on relationships of stress – strain ($\sigma - \varepsilon$) either force – deformation ($f - \delta$) for boundary portion and infill plate. With the identification of the Steel Plate Shear Walls began the approach, modes of degradation along with collapse from 36 specimens tested. A comparative study shows the measures and outcomes of the seismic performance evaluations [11].

Purba & Bruneau (2015) an evaluation of the potential for failure and seismic stability was carried out for Steel Plate Shear Walls (SPSWs) with two various theories designed for plates infill. The test was carried out with Steel Plate Shear Walls which more designated ignore the augmentation based on own border moment resisting frames through withstand the forces in shear of storey. The present possibility for decline evaluation is replicated as steel plate a shear wall which was constructed taking into account the distribution of shear forces of story surrounded by infill plates and border frames. Adjustments for improved performance of the collapse and factors affecting capacity for collapse were implemented [12].

Mehdi and Robert (2016) a modular model at a large – scale at the University of Alberta was tested in the sense of a research done experimental on its behaviour of an Steel Plate Shear Walls (SPSWs) along composite columns, moderately enclosed to observe and measure some essential parameters with any of this system relevant to an seismic design. The one – the bay, two – story, experiment was cyclically filled until it collapsed. The modular process of construction adopted for creating the model has been shown on overall behaviour to have very small effect. The model shows high rigidity at initial, acceptable ductility for displacement and more capacity for distraction of energy. The findings all the while the laboratory check suggested a particular a composite column had been studied for seismic performance improvement [10].

Jeffrey *et al.,* (2016) have reviewed and addressed contemporary procedures for design in several influential production codes to hollow structural sections (HSS) for fillet welds. Based on a laboratory tests set of welded connections by fillet between rigid end – plates and hollow structural sections, the more structural durability linked to spatial capacity – improvement component is examined and found in North American Specifications. A 33 of total attachments, in the welding which were built will have being an effective features, endure checked loading by the axial tensile force appealed to all the components of an hollow structural sections to fail. The weld strengths obtained experimentally were compared with the nominal strengths forecasted [6].

Cheng and Guowang *et al*., (2016) has studied and presented a laboratory study made from the Light – Gauge Cold – Formed Steel (CFS) Shear Wall applying circular holes in steel corrugated sheets. Corrugated steel exterior sheathing receive demonstrated an improvement in Cold – Formed Steel (CFS) Shear Walls strength substantially, yet it's the wall ductility has not been ideal as long as usance in seismic region. Incorporates one solution possible building aperture for improvement of ductility in the corrugated sheets. A 10 total the shear wall was full – scale in the test program included tests that had been carried out configurations of three circular various aperture and non perforated two sheathing configurations. Therefore, the researchers do not suggest in corrugated sheets using circular holes regardless of Cold – Formed Steel (CFS) Shear walls investigated toward achieving ductility [1].

Kara *et al*., (2016) have the goal of the study is to provide design details and response coming from Cold – Formed Steel (CFS) – Framing structure with full – scale which a sequence of dynamic arousals were tested throughout various construction stages. The reply in seismic about entire structure constructed against Cold – Formed Steel (CFS) is largely unexplored, significant work although has been done other behaviour for that members and subsystems concerning Cold – Formed Steel (CFS), especially shear walls.

The experiments mentioned more ingrate first investigation showing a Cold – Formed Steel – Framed structure designedly meet tectonic requirements in

North America. Under seismic excitation comprehensive, study illustrates a performance outstanding of these systems, thus stressing because it still is the result linked tether entire program the amount of reply and aren't just a lateral force resistant component [7].

Kara *et al.*, (2016) have the research aims to use the findings of the comprehensive instrumentation mounted on the full – scale recently tested Cold – Formed Steel (CFS) – Framed structures into provide an understanding deprave building's behaviour during seismic arousals. The research complements, in particular, as accompaniment analysis it thus emphasizes with device zone response, and design. In this direction, the reactions including walls as well as diaphragms in general structure reaction are isolated also analyzed using located strategically strain gauges, string potentiometers, and accelerometers. The findings are used to demonstrate the subsystem – level results preceding the sophisticated mechanism stage reply to Cold – Formed Steel (CFS) – Framed Building earthquake excitation is experienced [8].

III. DETERMINATION OF LATERAL FORCE: USING EUROPEAN CODE

Step 1: Preliminary Data:

– Type of Structure: Steel Framed Structure
– Material Used: Steel
– Site Location: Nashik, Maharashtra State. India
– Ground Stratigraphic Profile: Hard Rock
– Ground Type: A (As per Table 3.1, Page 34, EN 1998 – 1: 2004)
– Seismic Zone: III (As per Annex E, Page 37, I.S. 1893 (Part 1): 2016)
– Average Shear Wave Velocity Parameters v_s,30 ≥ 800 m/s (As per Table 3.1, Page 34, EN 1998 – 1: 2004)
– Importance Class: II (As per Table 4.3, Page 53, EN 1998 – 1: 2004)
– Importance Factor, γ_I = 1.0 (As per Clause 3.2.1 (3), Page 35, EN 1998 – 1: 2004)
– Response Spectra: As per clause 3.2.2.1 (1) P, Page 36, EN 1998 – 1: 2004, Elastic Ground Acceleration Spectra is also called as Elastic Response Spectrum.

Step 2: Ground Acceleration Calculation:

As per EN 1998 – 1: 2004, clause 3.2.1 (3), Page 35,
The design ground acceleration a_g on Type A ground = a_gR times the importance factor,
$a_g = \gamma_I a_g R$
where, a_g = Design ground acceleration on type A ground,
a_gR = Reference peak ground acceleration on type A ground
As per EN 1998 – 1: 2004, clause 3.2.1 (5) P, Page 35,
The design ground acceleration a_g on Type A ground, is not greater than 0.04 g = 0.39 m/s^2, or those where the product a_gS is not greater than 0.05 g = 0.49 m/S^2
The horizontal component of seismic action = S_cT
The vertical component of seismic action = $S_{vc}T$

Step 3: Horizontal elastic response spectrum:

As per EN 1998 – 1: 2004, clause 3.2.2.2 (1) P, Page 37,
The elastic Response Spectrum = S_cT
From Table 3.2, Page 38, EN 1998 – 1: 2004, For Ground type A,
S = 1.0, T_B(s) = 0.15, T_C(s) = 0.4, T_D(s) = 2.0

$0 \le T \le T_B$: $S_c\ (T) = a_g \times S \times \left(1 + \frac{T}{T_B} \times (\eta \times 2.5 - 1)\right)$

As per clause 3.2.2.2 (3), page 40, EN 1998 – 1: 2004

$$\eta = \sqrt{\frac{10}{(5+\xi)}} \geq 0.55$$

where ξ = viscous damping ratio of structure, (expressed in %) = 5 %

$$\therefore \quad \eta = \sqrt{\frac{10}{(5+5)}} \geq 0.55$$

$\therefore \eta = 1.0 \ge 0.55$

$$\therefore \quad S_c\ (T) = 0.39 \times 1.0 \times \left(1 + \frac{0}{0.15} \times (1.0 \times 2.5 - 1)\right)$$

$$\therefore \quad S_c\ (T) = 0.39$$

Step 4: Vertical elastic response spectrum:

As per EN 1998 – 1: 2004, clause 3.2.2.3 (1) P, Page 40,
The elastic Response Spectrum = $S_{vc}(T)$

$0 \le T \le T_B$: $S_{vc}\ (T) = a_{vg} \times \left(1 + \frac{T}{T_B} \times (\eta \times 3.0 - 1)\right)$

As per EN 1998 – 1: 2004, from Table 3.4, page 41, for Type 1,
$a_{vg} / a_g = 0.90$
$a_{vg} = 0.90 \times a_g$
$a_{vg} = 0.90 \times 0.39$
$a_{vg} = 0.351$

$$S_{vc}\ (T) = 0.351 \times \left(1 + \frac{0}{0.05} \times (1.0 \times 3.0 - 1)\right)$$

$$\therefore S_{vc}\ (T) = 0.351$$

Step 5: Design Ground Displacement (d_g):

As per EN 1998 – 1: 2004, clause 3.2.2.4 (1) P, Page 41,
$d_g = 0.025 \times a_g \times S \times T_C \times T_D$
$d_g = 0.025 \times 0.39 \times 1.0 \times 0.4 \times 2.0$
$d_g = 0.0078$

Step 6: Design spectrum for elastic analysis: S_d(T):

As per EN 1998 – 1: 2004, clause 3.2.2.5 (4) P, Page 41,

$0 \le T \le T_B$: $S_d\ (T) = a_g \times S \times \left(\frac{2}{3} + \frac{T}{T_B} \times \left(\frac{2.5}{q} - \frac{2}{3}\right)\right)$

where, a_g = 0.39, S = 1.0, T = 0, T_B = 0.15
As per clause 3.2.2.5 (6), Page 42, EN 1998 – 1: 2004
q = behaviour factor = 1.5

$$\therefore S_d\ (T) = 0.39 \times 1.0 \times \left(\frac{2}{3} + \frac{0}{0.15} \times \left(\frac{2.5}{1.5} - \frac{2}{3}\right)\right)$$

$$\therefore S_d\ (T) = 0.260$$

Step 7: Lateral Force Calculation:

As per EN 1998 – 1: 2004, clause 4.3.3.2 (2) a, Page 56,

Fundamental Period of Vibration = T_1

$$T_1 \le \begin{Bmatrix} 4 \cdot T_C \\ 2.0\ s \end{Bmatrix}$$

$\therefore T_1 = 4 \times T_C \le 2.0\ s$
$\therefore T_1 = 4 \times 0.4 \le 2.0\ s$
$\therefore T_1 = 1.6 \le 2.0\ s$
As per EN 1998 – 1: 2004, clause 4.3.3.2.2 (1) P, Page 56,

Seismic Base Shear Force = F_b

$F_b = S_d \times (T_1) \times m \times \lambda$
$S_d = 0.260$
As per EN 1998 – 1: 2004, clause 4.3.3.2.2 (3), Page 57,
$T_1 = C_t \times H^{3/4}$

$T_1 = 0.050 \times (12)^{3/4}$
T_1 = 0.322 seconds
m = 20025.818 / 9.81
m = 2041.368 kg
λ = Correction Factor
If $T_1 \leq 2T_C$, λ = 0.85 otherwise λ = 1.0
T_1 = 0.322, T_C = 0.4, $2T_C$ = 0.8

∴ 0.322 < 0.8, $\lambda = 0.85 F_b = 0.260 \times 0.322 \times 2041.368 \times 0.85$
F_b = 145.268 kN
As per EN 1998 – 1: 2004, clause 4.3.3.2.3 (3), Page 58,

$$F_i = (F_b) \times \frac{Z_i \times m_i}{\sum Z_j \times m_j}$$

Table 1: Horizontal Forces on Storey (F_i) with Base Shear (F_b).

Storey Level	Z_i (m)	m_i (kg)	$Z_i \times m_i$	$\frac{Z_i \times m_i}{\sum Z_j \times m_j}$	$F_i = (F_b) \times \frac{Z_i \times m_i}{\sum Z_j \times m_j}$
4	12	4991.267	59895.204	0.399	57.962
3	9	5011.517	45103.653	0.301	43.726
2	6	5011.517	30069.102	0.200	29.053
1	3	5011.517	15034.551	0.100	14.527
Sum =		20025.818	150102.510	—	145.268

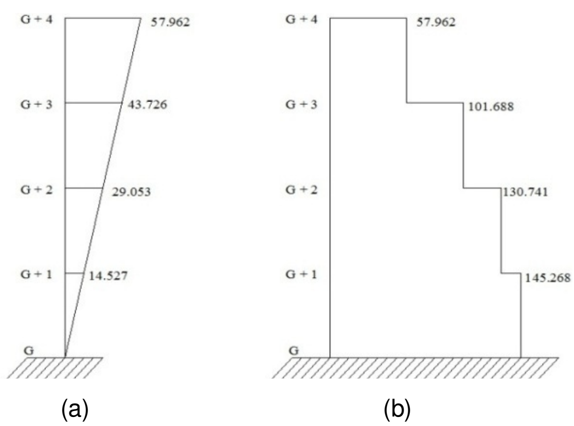

Fig. 2. G + 4 storey showing (a) Horizontal Force on each Storey and (b) Base Shear Force.

IV. RESULTS AND DISCUSSION

At top floor (roof level) the maximum horizontal force on storey (lateral force) is at the G + 3 and it is 57.962 kN as per European code.
At ground level the minimum horizontal force on storey (lateral force) is at base and it is 0 kN as per European code.

V. CONCLUSION

The horizontal force on storey (lateral force) at free end i.e. at top floor is maximum and at the fixed end it is zero.
Formulated a proposed a stepwise procedure to determine the lateral force and the base shear by using the European code.

VI. FUTURE SCOPE

The analysis and design may be done by using other codes viz., American code, Australian code, Canadian code, etc.

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