STUDIES ON THE BEHAVIOR OF GLASS FIBER REINFORCED GYPSUM WALL PANELS

Maganti Janardhana[1], A. Meher Prasad[2] and Devdas Menon[2]

ABSTRACT

Glass fiber reinforced gypsum (GFRG) wall panel is made essentially of gypsum plaster reinforced with glass fibers. The panels are hollow and can be used as load bearing walls. The hollow cores inside the walls can be filled with in-situ plain or reinforced concrete.

This paper presents guidelines for the use of GFRG wall panel as a lateral load resisting component in buildings based on a numerical analysis procedure to arrive at its capacity estimation under axial compression, compression with in-plane bending and shear. Variation of buckling load of unfilled GFRG wall panels for various widths is reported. The axial load carrying capacity of 1.02 m wide and 2.85 m high wall panel, obtained from the numerical analysis and the test results are comparable for this load case. While assessing the axial load capacity for design under compression, a minimum possible eccentricity (causing out-of-plane bending) is accounted for. An engineering model is proposed to assess the strength of unfilled and concrete filled GFRG wall panels in multi-storied building system subjected to lateral load such as earthquake.

Introduction

In a high seismic intensity zone, resistance of buildings to earthquakes is often ensured by adopting structural systems where seismic actions are assigned to structural walls (shear walls), designed for horizontal forces and gravity loads while columns and beams are designed only for gravity loads. Structural walls provide a nearly optimum means of achieving the important objectives, viz., *strength, stiffness* and *ductility*. Buildings braced by structural walls

[1] Ph D Research Scholar, [2] Professor, Structural Engineering Division,
Department of Civil Engineering, Indian Institute of Technology Madras, Chennai 600 036, India.

are invariably stiffer than framed structures, reducing the possibility of excessive deformations under small earthquakes. The necessary strength to avoid structural damage under moderate earthquakes can be achieved by properly detailed longitudinal and transverse reinforcement. Special detailing measures need to be adopted to achieve, dependable ductile response under major earthquakes (Paulay and Priestley, 1992).

Glass fiber reinforced gypsum (GFRG) wall, a new composite wall product known as Rapidwall®/Gypcrete® in the industry, is made essentially of gypsum plaster, reinforced with chopped glass fibers. The glass fibers about 300 – 350 mm long are randomly distributed inside the panel skins and ribs in the manufacturing process. The fiber content is 0.8 kg/m^2. The 120 mm thick panels are hollow and can be filled with in-situ plain or reinforced concrete to increase the strength. A typical cross section of the panel is illustrated in the Fig. 1.

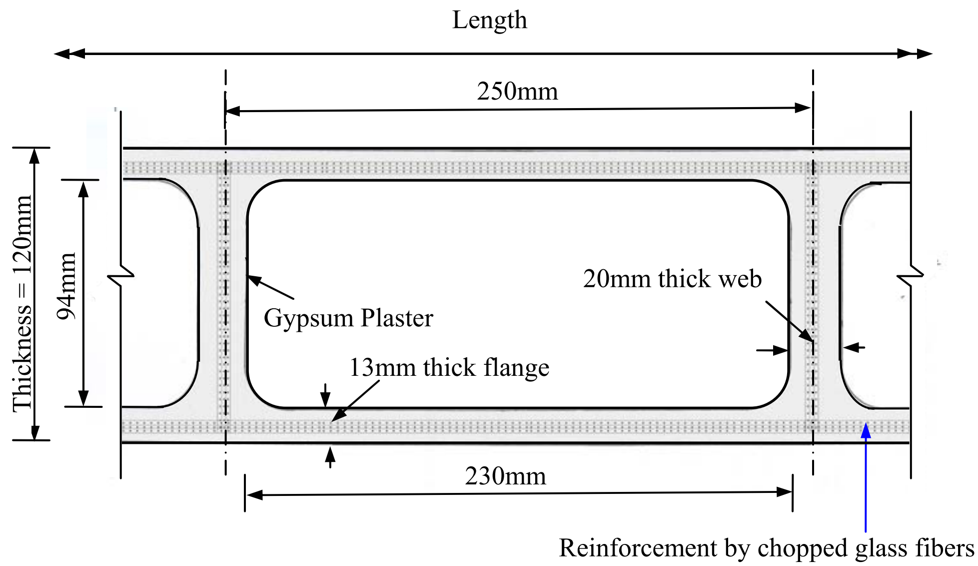
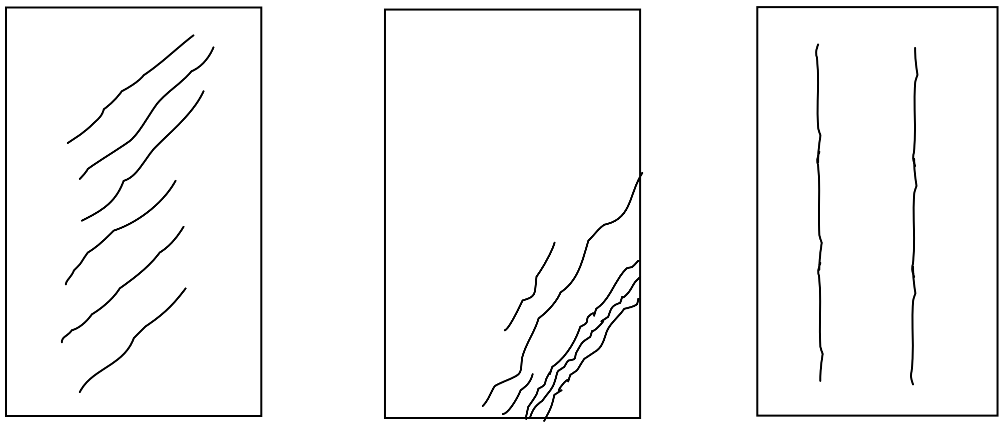


Figure 1. Cross section of GFRG wall panel

Wu and Dare (2004) have carried out axial and shear load tests on GFRG wall panels of standard 2.85 m height. The width of panel specimens was 1.02 m for axial load tests, 1.52 m and 2.02 m for shear load tests. They have reported that under axial load, *unfilled* GFRG wall panels failed due to plaster crushing, irrespective of the eccentricity of axial load. The *concrete filled* specimens all failed due to buckling and flexural tensile breaking of the GFRG walls. The failure load was governed by the eccentricity and support conditions. The compressive strength of unfilled wall panels was governed by the plaster strength and that of concrete filled panels was governed by out-of-plane buckling. The axial load carrying capacity of concrete filled wall panels was only affected by the axial load eccentricity and support conditions. The failure mode for shear load specimens of concrete filled walls was due to the longitudinal tearing of the GFRG panels, which is very different from the failure mode of a traditional RC wall. As a result, the shear strength is governed by the strength of the GFRG panels and was not affected by the strength of the concrete infill. Typical shear failure modes for unfilled panels and concrete filled panels are shown in the Fig. 2.



(a) Unfilled panel – *Diagonal cracks*

(b) Unfilled panel – *End crushing*

(c) Concrete filled panel – *Vertical cracking*

Figure 2. Typical failure modes for GFRG wall panels subjected to shear (*Wu and Dare, 2004*)

Wu (2004) has reported that there are two types of shear failure modes in a building constructed with GFRG walls. The first mode is the shear failure of the panel itself, and the second is shear sliding at the interface of a wall and the floor slab. The continuity of longitudinal reinforcement at the horizontal joint may affect the shear strength of both the failure modes.

Mechanical properties of the GFRG panel, as reported by Wu and Dare (2004) are shown in Table 1. Axial load capacity of wall panels against buckling are estimated and shown in the Table 2 and the in-plane lateral load buckling capacity is shown in Table 5. It is found that the Poisson's ratio of the wall panel is approximately equal to 0.2 from experimental results. The modulus of elasticity of the panel considered is 3000 MPa.

Estimation of GFRG Wall Panel Capacities

In the present study an attempt is made to estimate for design purposes the capacities of GFRG wall panels under (i) Axial loads, (ii) Axial load with out-of-plane bending (iii) Out-of-Plane bending capacity (iv) Axial load and in-plane bending moment and (v) Capacity of wall panel due to shear load.

Table 1. Mechanical Properties of GFRG Building Panel *(Wu and Dare 2004)*

Mechanical Property	**Characteristic Value**	**Remarks**
Unit Weight	40 kg/m^2	Unfilled Single leaf GFRG Panel.
Uni-axial Compressive Strength	160 kN/m	
Uni-axial Tensile Strength	35 kN/m	
Elastic Modulus	3000 – 6000 MPa	
Coefficient of Thermal Expansion	12 x 10^{-6} mm/mm/^{0}C	
Water Absorption	< 5%	By weight after 24 h of immersion
Thermal Resistance	0.36 m^2 K/W	Unfilled Panel
Sound transmission coefficient	28 45	Unfilled panel Concrete filled panel
Fire Resistance Level	> 3 h	For Structural adequacy

Axial Load Capacity

While assessing the axial load carrying capacity of the wall panels a minimum eccentricity causing out of plane bending is accounted for. As per standards such as IS 456 (2000), the design of reinforced concrete walls should take into account the actual eccentricity of the vertical force subjected to a minimum value of 0.05 times the wall thickness t (6 mm for t = 120 mm). According to masonry design codes such as IS 1905 (1987), the design of a wall shall consider appropriate eccentricity, which in no case shall be taken to be less than t/24 (5 mm for t = 120 mm).

In the case of wall panels supporting floor slabs from one side, the eccentricity to be considered should be more than the minimum values indicated above. It is recommended that a value of minimum eccentricity equal to t/6 (20mm for t = 120 mm) shall be considered conservatively. Additional value of eccentricity may be considered when the out of plane bending is explicitly involved.

The characteristic values of axial compressive strength of GFRG wall panels are obtained from the compression test results on 2.85 m full height panel subjected to eccentric loading. In general, it is conservative to assume pinned-pinned condition as shown in Fig. 3. It may be noted that for design purposes, the characteristic values should be divided by partial safety factor 1.7.

Finite element analysis of the wall panel, using plate-shell elements to model both flanges and webs, was carried using the SAP 2000 NL software. These numerical analysis results are compared in Table 2 with the experimental results reported by Wu and Dare (2004). It is seen that the numerical results are comparable with the experimental results for the 1.02 m panel. The axial load carrying capacity of the unfilled GFRG panels of widths 1.52 m and 2.02 m, estimated by finite element analysis are also shown in the Table 2.

Table 2. Axial Load Carrying Capacity of Unfilled GFRG Wall Panels

Width of Panel (m)	**Numerical analysis Results (kN)**			**Experimental Results** (kN)**	
	***e* = 0**	***e* = 6mm (Minimum)**	***e* = 20 mm**	***e* = 0**	***e* = 20 mm**
1.02	173.7	168.7	158.1	132.4 – 166.7	119.6 – 166.7
1.52	245.3	252.4	230.1	–	–
2.02	328.7	319.6	300.0	–	–

e = Eccentricity
** Wu and Dare (2004)

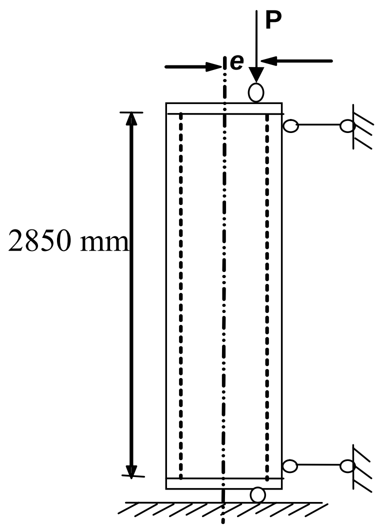


Figure 3. Experimental set- up for Pinned-Pinned panels

The same finite element model was used to estimate the elastic critical buckling load of the wall panel, and the results are listed in Table 3 for different widths of the wall panel. These results indicate values much higher than those shown in Table 2, confirming that buckling is not a likely mode of failure, as evidenced by the testing (which showed crushing of plaster).

Table 3. Buckling Load Values When the Wall is Subjected to only Axial Load

Width of GFRG Wall Panel (m)	**Buckling Load P_{cr} (kN)**
1.02	622
1.52	926
2.02	1233

Out-of-Plane Bending Capacity

The out-of-plane flexural strength of 120 mm thick GFRG wall panel without filling is shown in Table 4. From the test results, it is found that filling without reinforcement does not improve the out-of-plane moment capacity of the panel.

Table 4. Out-of-plane flexural capacity of Unfilled / Concrete filled Panel without Steel Reinforcement

Property	Ribs parallel to span	Ribs perpendicular to span
Out-of-plane moment capacity (kNm / m)	2.1	0.88

In-Plane Bending Capacity

GFRG wall panels can be used as load bearing walls in multi storied buildings capable of resisting fairly large lateral load. Each wall can act as a shear wall resisting vertical load, in-plane bending and shear. Invariably, such a wall shall be infilled with reinforced concrete. The following simplified procedure may be used to calculate the ultimate in-plane flexural strength of concrete filled GFRG wall panel.

1. Assess the stress distribution along the cross section of the wall panel based on linear elastic assumption.

$$\sigma = \frac{P}{A} \pm \frac{M\ y}{I} \tag{1}$$

2. Based on the stress distribution from the Eq. 1, there are two different cases to be considered.

(a). The whole cross section is under compression or there is no tensile stress in the cross section ($\sigma \geq 0$) compare the maximum value of 'σ' in the cross section with the compressive strength of wall panels given in Table 1. If the calculated stress is less than the design compressive strength of the wall, the design is safe otherwise redesign is required.

(b) If tension exists in the cross section ($\sigma < 0$) go to step 3.

The unfilled and concrete filled GFRG wall panel without continuous reinforcement in the cores is not able to transmit tension between floors, therefore case (b) is not applicable to unfilled or concrete filled GFRG wall panel without continuous longitudinal reinforcement.

3. When tension exists in the cross section, the flexural strength can be calculated using the following assumption.

(a) The contribution of the panel to ultimate strength in tension and compression is neglected and the system is treated as filled concrete with concrete effective only in compression and the reinforcement bars in tension.

(b) Tension reinforcement (full length bars) are assumed to act as unbonded bars with constant stress, limited to 90 MPa in the entire tension zone (assuming lack of bond between the infilled concrete and the wall panel).

The assumed stress distribution across the cross section of a GFRG wall panel is shown in Fig.4. From the test results, it has been found that there is practically no bond between concrete and GFRG wall panel. It is assumed that all reinforcing bars are subjected to same stress and that this stress is limited to the following value based on the studies in prestressed concrete section with unbonded tendons.

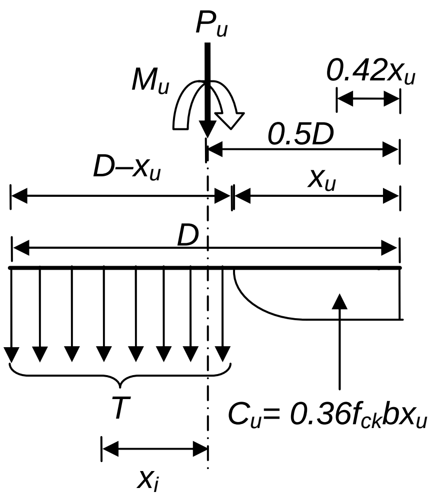


Figure 4. Stress Distribution across the GFRG Wall Panels under Axial Load and in-plane Bending Moment.

Tensile stress in steel rod,

$$f_{st} = 70 + \frac{0.85 f_{ck}}{100\left(\frac{A_{st}}{bd_p}\right)} \qquad \text{in N/mm}^2 \tag{2}$$

Equating forces:

$$P_u = 0.36 f_{ck} b x_u - f_{st} \sum_{i=1}^{n} A_{sti} \tag{3}$$

where A_{sti} is the area of the i^{th} bar located at a distance x_i from the centre.

Equating moments,

$$M_u = f_{st} \sum_{i=1}^{n} A_{sti}\left(D/2 - x_i\right) + 0.36 f_{ck} x_u \left(D/2 - 0.42 x_u\right) \tag{4}$$

where x_i is considered to be positive if it is on the tension side of the mid depth.

From the Eqs. 3 and 4, the value of x_u and A_{sti} can be determined by trial and error. Alternatively, $P_u - M_u$ interaction curves may be generated for a given panel with infill concrete (f_{ck}) and given area of steel per cavity (A_{st}). Such an interaction diagram (using non-dimensional coordinates) has been generated and is shown in Fig. 5. The value of p/f_{ck} can be obtained from this diagram and the suitable bar reinforcement can be identified. It may be noted that minimum eccentricity requirement should be satisfied.

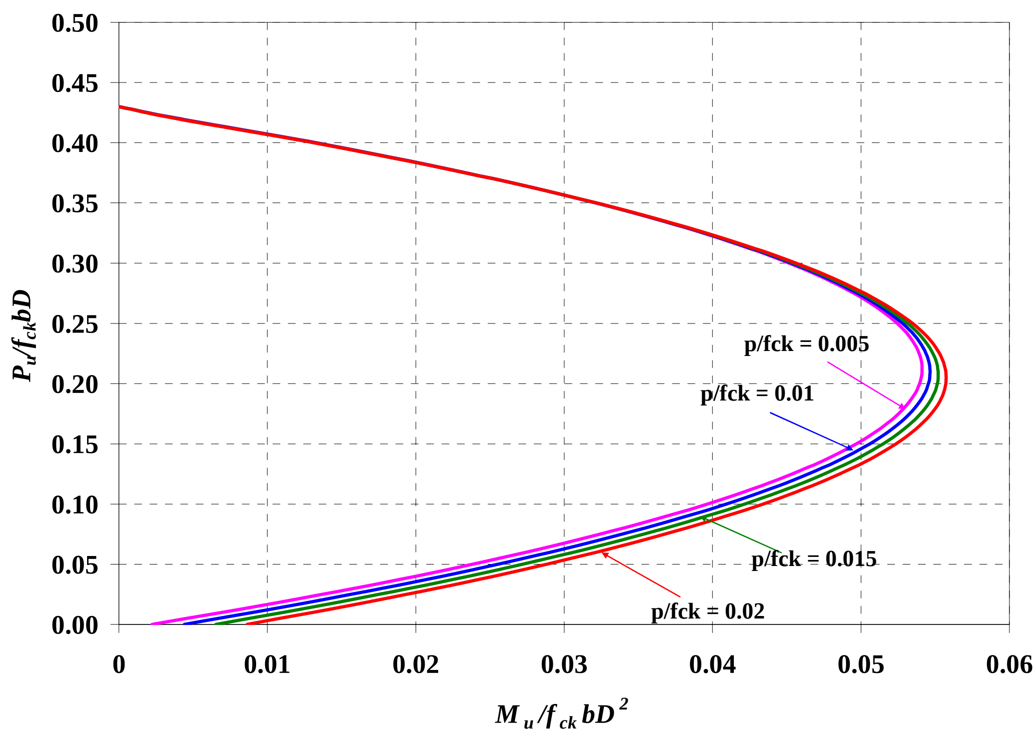


Figure 5. Interaction diagram for panel subjected to axial load P_u and in-plane bending moment, M_u

Shear Capacity of Panels

Wu and Dare (2004) have reported that in all of the shear load tests on unfilled panels, there were visible 45° shear cracks, developed before the peak load was reached, as shown in Fig.2(a), and shear strength varies from 19.1 kN/m to 24.5 kN/m. Using strength of materials approach, the capacity of unfilled panels under shear load can be assessed as follows.

$$\text{Capacity of unfilled panel under shear load} = 2\left(\frac{b}{\cos\theta}\right)t_f\sigma_t$$

where b = Width of wall panel,
θ = Inclination of crack with respect to horizontal axis,
t_f = Thickness of flange of the wall panel,
σ_t = Permissible tensile strength of wall panel.

This capacity works out to 39 kN/m for a GFRG panel of 1.02 m width.

Buckling Capacity under In-Plane Shear Load

The possibility of buckling failure under shear loading was investigated in the present study through numerical analysis using SAP 2000 NL. The results obtained are shown in Table 5, corresponding to *three* widths of wall panel. The values of lateral load obtained (262 to 596 kN) are significantly higher than the capacities reported by Wu and Dare (2004). Hence it is clear that buckling mode of failure is *unlikely* to occur not only under axial load but also under shear loading for the unfilled GFRG wall panels.

Table 5. In-Plane Buckling Capacity due to *shear* Load alone

Width of Wall Panel (m)	**Buckling Lateral Load V_{cr} (kN)**
1.02	262
1.52	425
2.02	596

Conclusions

Axial load carrying capacity of unfilled GFRG wall panels, of various widths when subjected to eccentric loads, is estimated using numerical analysis. The lateral load carrying capacity of panels is also estimated. A simplified procedure has been suggested for assessing in-plane flexural strength of concrete filled wall panels. For a given force demand, reinforcement required for a concrete filled GFRG wall panels can be obtained using interaction diagram that has been developed. Using simple approach, the capacity of unfilled panels under shear load is estimated. It is also established by comparing the results of finite element buckling analysis with the available experimental results, that failure of the GFRG wall panel does not occur due to buckling, on account of in-plane axial and shear loads, as the critical loads are much higher than the actual capacities.

References

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