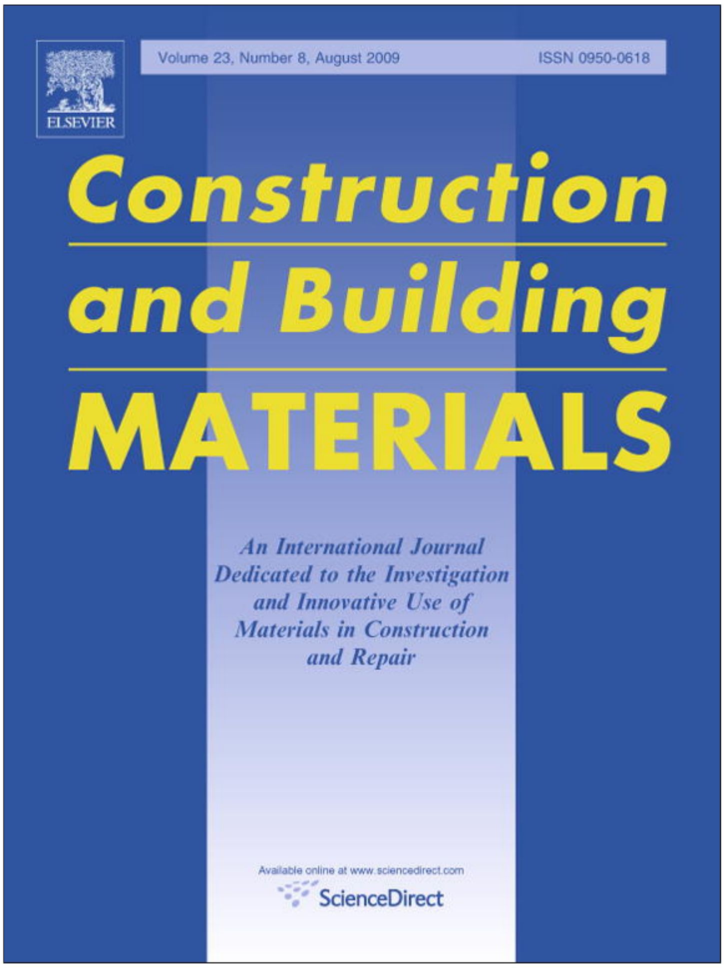
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The structural behavior and design methodology for a new building system consisting of glass fiber reinforced gypsum panels

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ABSTRACT

Glass fiber reinforced gypsum (GFRG) walls are prefabricated large gypsum panels with hollow cores. Developed in Australia in the early 1990s and subsequently adopted by other countries, including China and India, this material is used in residential, commercial, and industrial buildings. GFRG walls are used both architecturally and structurally as walls and slabs, with no columns and beams required. They have already found wide application, even without mature structural design codes, largely because of their environmental friendliness. In India, GFRG walls have been approved by the World Bank as being eligible for Carbon Credits under the Kyoto Protocol. GFRG panels are a composite material consisting of gypsum plaster and glass fibers. When the cavities are filled with reinforced concrete, the interaction between the concrete and the GFRG panels produces another composite. As a result, the structural behavior of GFRG walls and the associated building system are more complicated than that of conventional structural systems. This paper presents the results of extensive experimental and theoretical investigations into the structural behavior of GFRG walls, and offers a structural design methodology for GFRG walls and the associated building system.

1. Introduction

Known as Rapidwall in the building industry, glass fiber reinforced gypsum (GFRG) walls were developed in Australia in the early 1990s. GFRG walls are hollow machined panels made of modified gypsum plaster and reinforced with cut glass fiber. A typical panel and GFRG building is shown in Fig. 1. During the manufacturing process, glass fibers of about 300–350 mm in length are randomly distributed inside the panel skins and in the ribs. The fiber volume in the panel is about 0.8 kg per square meter of wall surface area. The physical properties of standard GFRG panels are listed in Table 1.

In building construction, standard large GFRG panels are tailor-cut in the factory into building components that may have window and door openings. These components are then transported to the construction site and erected in a similar way to the construction of precast concrete panels. The cavities (hollow cores) inside the panel can be filled with various materials, such as concrete or insulation materials, to serve different purposes, such as to increase the strength or improve the thermal and sound insulation of the walls. In a GFRG building, most or all the components are constructed with GFRG panels, which means that the walls serve as a combination of architectural partitions and structural walls. Research has shown that GFRG building assemblies have a smaller embodied energy (EE) coefficient and CO_2 gas emission (from the manufacturing of panels to the completion of building construction) than other traditional building construction materials, such as bricks, reinforced concrete, and precast concrete panels [1]. GFRG paneling is thus considered to be a green product that helps to save energy and protect the environment.

GFRG buildings are a new type of construction to which conventional structural theories and design codes are not applicable. Therefore, extensive research work has been undertaken by the author, both in Australia and Hong Kong, to gain a better understanding of the structural behavior of GFRG walls and the associated building system with a view to developing design guidelines. A comprehensive investigation that included about 120 experimental tests and theoretical studies was completed at the University of Adelaide and the University of South Australia in 2002 [2], and structural design theories and guidelines were developed based on this investigation [3–7].

Large-scale experimental tests, similar to those conducted in Australia, have also been undertaken in India at the Indian Institute of Technology Madras [8,9]. The Indian tests also included six full-scale shake table model tests simulating two-story houses [10]. In China, large-scale experimental tests similar to those in Australia and India have been completed at Tianjin University [11–14] and Shandong Construction University [15,16]. A full-scale five-story GFRG building was constructed for a destructive test at Shandong

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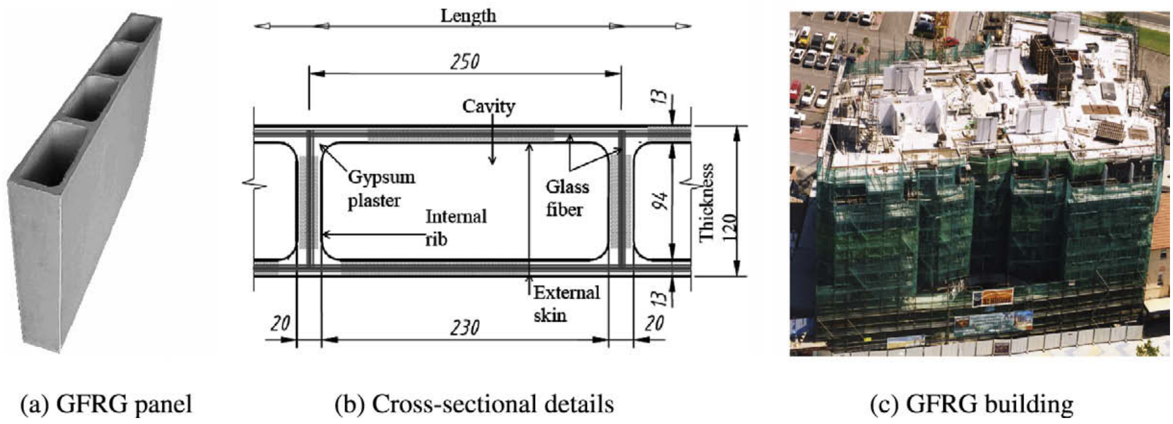


Fig. 1. GFRG panel and building.

Construction University [17], and an in-situ, non-destructive dynamic test was conducted on a recently built six-story GFRG building in Tianjin [12]. These Chinese tests were part of a combined Australian-Chinese test program aiming to develop Chinese design guidelines for GFRG construction.

This paper reports experimental and theoretical studies on GFRG walls and associated structural system undertaken by the author in Australia and Hong Kong since 2002. It should be noted that the experimental results for the axial and shear testing of GFRG walls have been published separately, but for completeness the results of the two tests are briefly introduced in this paper.

2. GFRG building system

2.1. Structural integrity and robustness

With infill reinforced concrete in their cavities, GFRG walls have significant axial and shear strength, and are suitable for the construction of multi-story buildings. GFRG buildings are similar to constructions with precast concrete wall panels. As the main structural issue for construction with precast concrete wall panels is making adequate connections between the precast units, it is believed that GFRG buildings suffer a similar problem.

The typical horizontal joints between two GFRG walls and the vertical joint between the walls and a slab are shown in Fig. 2a and b, respectively. It is clear that the joints are significantly weaker than the wall itself, and it is this inherent weakness of the joints that has caused serious concern about the seismic performance of GFRG buildings, as the seismic design principle of "strong columns, weak beams, and stronger joints" is usually applied to GFRG buildings, especially in mainland China.

In fact, the GFRG structural system is very different from the conventional rigid frame structural system that must abide by the "strong columns, weak beams, and stronger joints" principle. Indeed, in the typical structural form of GFRG buildings, as illustrated in Fig. 3, the horizontal joints and the out-of-plane resistance of the vertical joints can be completely ignored. Obviously, the structural system is stable and sound as long as the walls and joints have sufficient in-plane axial, flexural, and shear strength. Although the GFRG panels stop at the floor joints, which reduces the out-of-plane flexural resistance of the walls, this reduction in strength does not affect the overall stability of the system, as the whole structure relies only on the in-plane resistance of the walls. The joints only provide axial and shear resistances, which are virtually unaffected by the discontinuity of the GFRG panels. The infill concrete cores inside the GFRG panels and the slabs are monolithically cast in-situ, as with reinforced concrete constructions. Furthermore, the continuous reinforcement bars inside the concrete cores of the GFRG walls and slabs form a strong, closely spaced, and continuous tie system like a net, which avoids the weak connections found in constructions with precast concrete walls and forms a highly robust structure. The typical failure mode of progressive collapse for precast wall constructions is unlikely to occur in GFRG buildings, as long as the reinforcing bars inside the concrete cores of the GFRG walls satisfy the requirement of the minimum tie strength specified by the relevant reinforced concrete design codes.

Several strong earthquakes in the past have demonstrated that the in-field seismic performance of properly designed precast con-

Table 1
Physical and mechanical properties of GFRG panels.

Property name	Value	Note
Unit weight	40 kg/m^2	
Thermal expansion coefficient	12×10^{-6} mm/mm/°C	
Water absorption	<5%	By weight after 24 h immersion
Thermal resistance	0.36 m^2 K/W	Unfilled panel
	1.63 m^2 K/W	With 35 kg/m^3 and R2.5 rockwool batts infill and standard texture finishing
Sound transmission coefficient (STC)	28	Unfilled panel
	45	Concrete-filled panel
Fire resistance level (FRL)	>3 h	For structural adequacy
Young's modulus	3–5 GPa	
Compressive strength	167 kN/m	
Tensile strength	36 kN/m	

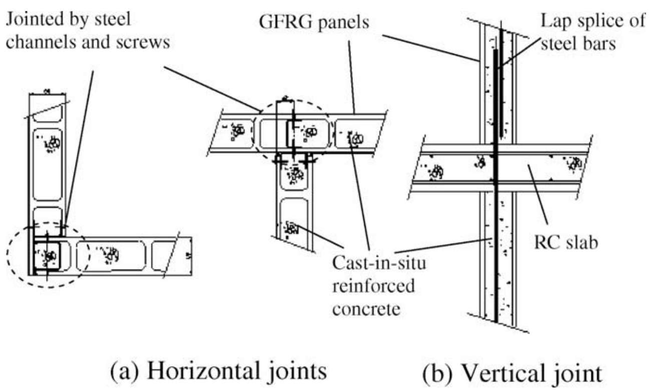


Fig. 2. Typical joints.

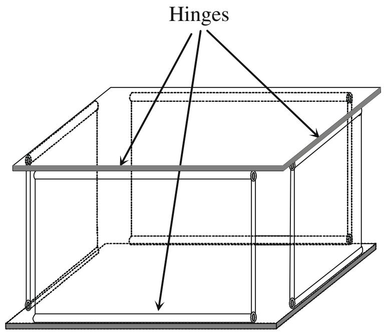


Fig. 3. GFRG building system.

crete panel constructions is outstanding [18], which indicates that GFRG buildings should be at least as good as, if not better than, precast concrete wall buildings in terms of seismic performance. The Indian shake table tests on full-scale two-story houses demonstrated the excellent seismic performance of the GFRG structural system [10]. Only minor cracks were noticed in one of the six models under 0.36 g earthquake excitations, and no structural distress was found in any of the six houses tested. In a destructive test on a five-story full-scale building at Shangdong Construction University [17], no visible structural cracks or other distresses were observed in the building under a cyclic horizontal force of a magnitude of 100 tons, which is equivalent to a zone 8 earthquake in the Chinese seismic code of practice. The testing is ongoing, and the loading will be increased from 100 to 200 tons in subsequent tests.

2.2. Relative slips and partial interactivity

Although the core concrete is cast into the cavities of the GFRG panels, the bond between these two materials is neither strong nor reliable, which may result in relative movements or slips on the interfaces when the structure deforms. The slips reveal that the composite action between a GFRG panel and the concrete cores is only partially, rather than fully, interactive. This partial interactivity causes extra complexities both in the structural behavior and in the structural analysis of the GFRG walls.

However, this partial interactivity is beneficial in terms of seismic performance, as the relative slips between the infill concrete cores and the GFRG panels dissipate the seismic energy arising from internal frictional movement. Furthermore, the bond can still overcome a certain amount of interfacial shear stress, and slips only occur with the significant deformation of the walls. This property does not reduce the stiffness of the walls significantly at a low service load, but can significantly increase the deformation capacity and ductility. A slip is actually a relief from local strain in higher stress areas and an increase in strain in lower stress areas [2], which delays material failure and increases the deformation capacity and ductility.

These structural characteristics have been substantiated by in-situ, non-destructive dynamic tests on an existing six-story GFRG building, which revealed that the natural frequency of the building is relatively lower and the damping ratio relatively higher than that of other similar buildings constructed with reinforced concrete and masonry [12]. These test results reflect the better seismic performance of GFRG buildings compared with more traditional constructions.

3. Shear strength of GFRG walls

The typical shear failure modes of conventional reinforced concrete shear walls are diagonal tension failure, diagonal compression failure, and shear sliding failure [19]. However, the shear failure mode of GFRG walls is completely different from that of conventional reinforced concrete walls due to the separation of the concrete by the internal ribs of the GFRG panel. Therefore, the shear design of GFRG walls must differ from the design of reinforced concrete walls. It is noted that similar design equations to those used in the reinforced concrete design code for reinforced concrete shear walls are adopted in the design code for GFRG walls in Shandong Province, China [20]. However, this may be inappropriate, for reasons discussed later.

3.1. Experimental tests

The purpose of the shear tests was to establish the shear strength of GFRG walls when they are used as load-bearing walls in buildings to resist both vertical and lateral loads. The test setup used for the shear tests conducted in Australia is shown in Fig. 4. The rather complicated test rig was used to spread the lateral (shear) load uniformly into the panel without causing the local failure of the plaster, as the strength of gypsum plaster is relatively low and the skins of the wall panels are thin. Another feature of the test rig was that the axial load could be adjusted (see Fig. 4c) so that the axial load effect on the shear strength could be measured. Both concrete-filled and unfilled GFRG panels were studied by the author, but only the results for concrete-filled walls are reported and discussed in this paper.

As the results of the monotonic and cyclic shear tests have already been published [3,4,6], only important conclusions from the tests are provided in the following paragraphs. More details can be found in [2–4,6].

3.1.1. Longitudinal shear failure mode

The typical shear failure mode was longitudinal shear in the gypsum plaster panel between two adjacent concrete cores, as shown in Fig. 4d. This indicates that the adhesive and frictional bond between the concrete cores and the GFRG panel was loosened such that relative slip occurred between the two components. This resulted in the over-stressing and longitudinal tearing off of the two 13 mm thick panel skins. Although visible 45-degree shear cracks, as shown in Fig. 4d, developed at about 50–70% of the ultimate shear load, these diagonal cracks did not affect the shear strength of the panel. The lateral resistance (shear strength) dropped quickly after the longitudinal cracks developed.

The shear failure of GFRG walls is more ductile than that of conventional reinforced concrete walls because the glass fibers inside the panel prevent the brittle tensile failure of the materials and the unique longitudinal shear failure mode creates a more ductile nature.

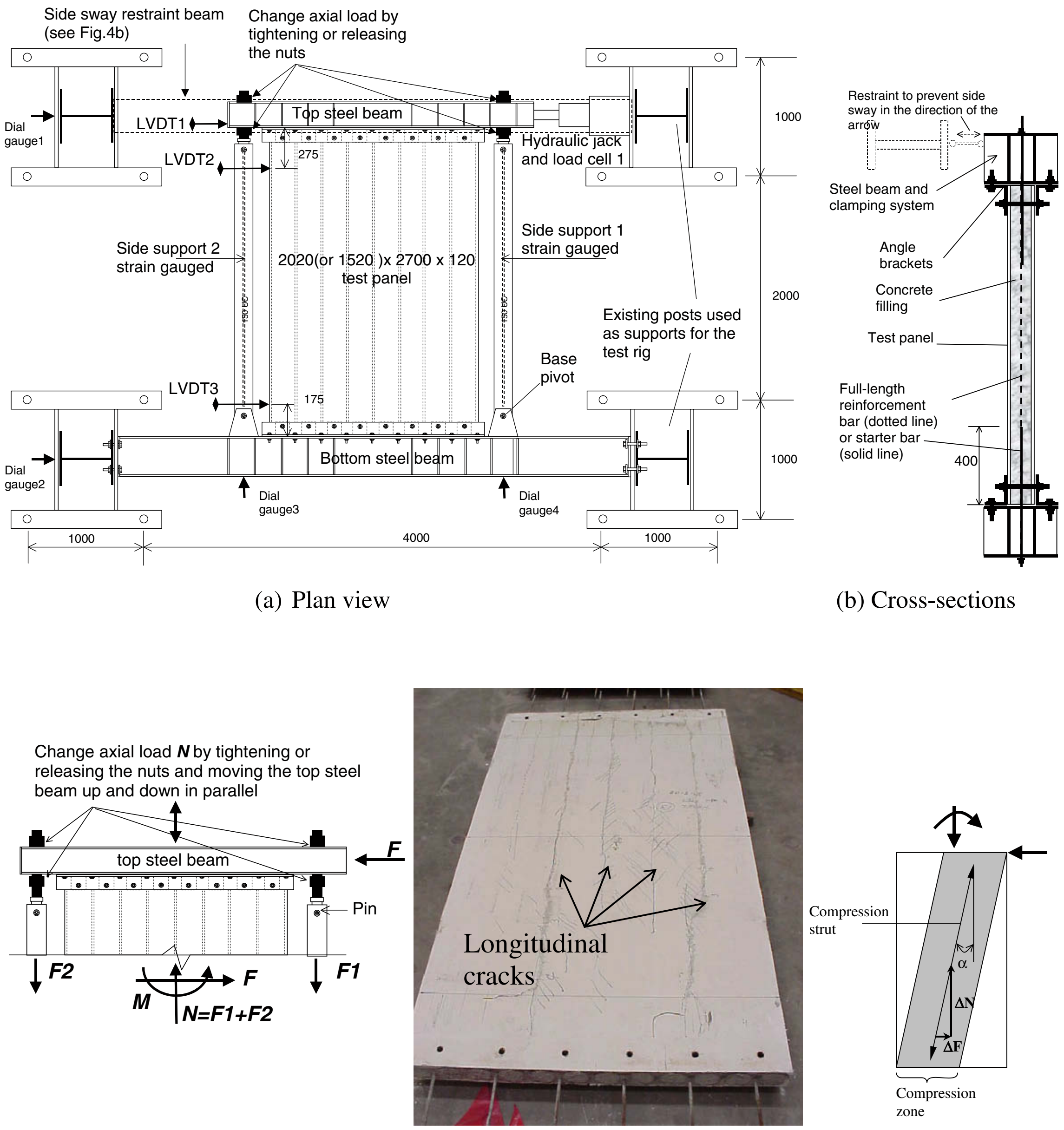
3.1.2. Unit shear strength

The shear strength of the walls was found to be proportional to the length of the wall. This experimental observation is consistent with the theoretical analysis. It can be easily derived that the longitudinal shear strength of an elastic wall is given by [7]

$$V_u = \frac{2L}{3} q_L = q_h \cdot L, \tag{1}$$

where L is the horizontal length of the wall, q_L is the unit longitudinal tearing strength of the GFRG wall, and q_h is the unit horizontal shear strength of the wall, which was found to be around 50 kN/m from the shear tests [3].

Therefore, the shear strength of a concrete-filled GFRG wall is simply equal to the unit shear strength q_h multiplied by the length of the wall. The unit strength depends on the properties of the gypsum plaster and the quantity and distribution of the glass fibers. As a result, the manufacturing process will affect the unit shear



(a) Plan view (b) Cross-sections

(c) Internal forces (d) Longitudinal shear failure (e) Axial load effect

Fig. 4. Shear tests.

strength. This was evident from the results of shear tests with GFRG panels produced on different production lines [6].

3.1.3. Composite action

The longitudinal shear through the gypsum panel was due to relative weakness caused by the discontinuity of the concrete and the reinforcement in the horizontal direction. This indicates that GFRG walls have a much smaller shear strength than reinforced concrete walls. The gypsum panel acted as a "link" between concrete cores that restricted the free relative movement between the cores and caused the longitudinal shear stress. The gypsum panel therefore acts compositely with the concrete cores and cannot be considered only as a lost-formwork for construction.

3.1.4. Concrete strength

It was observed that there was no damage to the concrete cores after the longitudinal shear failure of the walls. This failure mode suggests that the shear strength was largely determined by the longitudinal shear (tearing) strength of the plaster panel and was not affected by the concrete strength and the reinforcement inside the concrete cores. In other words, concrete strength is not related to shear resistance in GFRG walls.

3.1.5. Effect of the reinforcing bars

Shear tests with different arrangements of longitudinal (vertical) reinforcement bars inside the concrete cores also showed that the shear strength of GFRG walls is not affected by the longitudinal reinforcement [4]. However, transverse (horizontal) reinforcement bars passing through concrete cores can significantly increase the resistance of the longitudinal shear and hence increase the shear strength of the walls. The transverse bars need to penetrate the ribs of the GFRG walls, which increases the complexity and cost of the construction, and thus it is not common construction practice to use transverse bars inside GFRG walls, except in Tianjin, China.

Transverse bars are required within the remaining part of the GFRG panel above a door or a window opening if it is to be used as a lintel to support loads, as the bars act as tension or compression reinforcements [5].

Although the shear strength of GFRG walls is not affected by the longitudinal bars, the continuity of the longitudinal reinforcing bars inside the walls is critical to the overall integrity and stability of multi-story GFRG buildings, as discussed in Section 2.

3.2. Axial load effect

Shear test results have shown that the shear resistance increases with an increase in axial load [2,3]. This axial load effect can be explained using the strut model of Priestley et al. [21], as shown in Fig. 4e, where an axial force N causes an additional horizontal resistance F. However, the strut cannot be formed if a concrete wall is cut into several longitudinal strips and the contact surfaces between these strips are smooth. The concrete inside a GFRG wall is divided into longitudinal strips by the ribs of the GFRG panel. The adhesive bond and friction between the gypsum panel and the concrete cores provide a certain longitudinal shear resistance to form the diagonal strut. However, the diagonal strut action is lost when the interfacial bond disappears. This phenomenon was observed in shear tests of GFRG panels with a smoother internal surface, during which the axial load had a much smaller effect on the shear strength of the GFRG walls [6].

A finite element model was developed to further analyze and understand the axial load effect [6]. A GFRG panel is in itself a complicated three-dimensional structure, but the three-dimensional contact interfaces between the concrete cores and the panel make the FEM modeling unusually complicated and difficult. To resolve this problem, the author and his co-worker developed a two-dimensional FEM model that can effectively convert the 3D problem into a 2D problem with a good accuracy of solution [6]. The FEM analyses verified that the roughness of the interface between the concrete core and the GFRG panels is the factor governing the axial load effect.

As the roughness of the internal face of the GFRG wall is difficult to control and quantify during the manufacturing of GFRG panels, the axial load effect on the shear strength should be omitted in practice to ensure a conservative design.

3.3. Internal frame action and partially concrete-filled walls

When floor slabs have a significant flexural strength or beams are provided at two adjacent stories (i.e., at the top and bottom of a wall), internal reinforced concrete cores can form a significant frame action with the floor slabs or beams. In this case, a shear deformation between two adjacent stories not only induces shear resistance from the wall panel itself, but also produces a lateral (shear) resistance from the internal frame due to flexural actions of the frame.

Shear tests in the absence [2–4] and presence [6] of internal frame action have demonstrated that the lateral resistance of the internal frame may be in the same order as that of the shear resistance of the wall panel. Test results have also shown that the superposition law is applicable in this case. In other words, the total shear strength of the wall equals the shear resistance of the GFRG panel plus the lateral resistance of the internal reinforced concrete frame. This is because that the peak of the lateral resistance of the internal frame occurs at or near the peak shear strength of the wall due to the ductile longitudinal shear failure mode. These findings have been verified by finite element analyses [6].

Thus, the lateral resistance (total shear capacity) of a GFRG wall with internal frame action can be calculated by summing the lateral flexural resistance of the internal reinforced concrete frame and the shear strength of the GFRG panel, which equals the length of the panel multiplied by the unit shear strength. The lateral resistance of the internal frame can be calculated using conventional reinforced concrete theory by treating the internal frame as a standalone frame structure [6].

In the presence of significant internal frame action, that is, where there are thick floor slabs or beams above and below a GFRG wall, it may be unnecessary to fill every core of the GFRG panel with concrete, because a concentrated load can be spread to a large area of the wall by the slab or a beam to avoid the local crushing of the gypsum plaster. In this case, the wall can be partially filled with concrete (some cores are fully filled and some cores are left empty) to save costs, provided that the vertical strength of the wall is sufficient. Experimental and theoretical studies have found that the longitudinal shear failure for partially filled GFRG walls is still the typical failure mode. Furthermore, the lateral resistance of the walls also consists of two parts: the flexural resistance of the internal reinforced concrete frame and the shear strength of the GFRG panel, which equals the length of the panel multiplied by the unit shear strength [6].

4. Axial strength

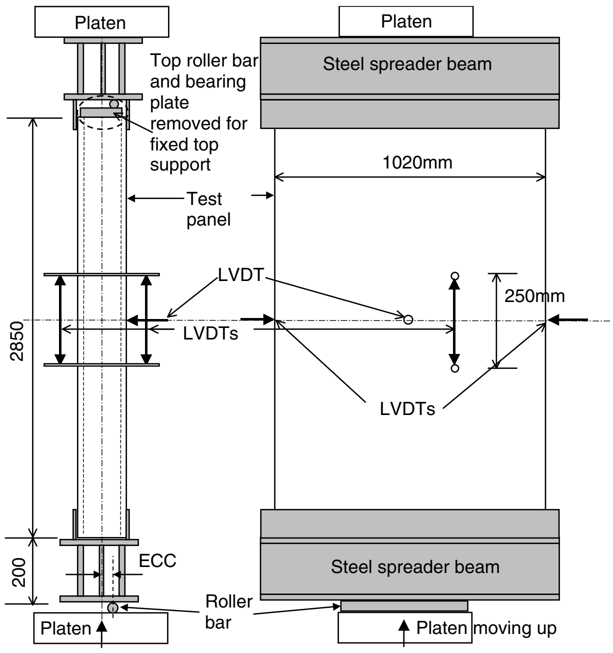
Compression tests were undertaken to investigate the axial strength of the GFRG walls [2,3]. The test setup is shown in Fig. 5. The following conclusions were drawn from the axial load tests.

1. The buckling of the wall was the typical failure mode for all test conditions, because the walls were slender for a typical floor height of about 3 m.
2. The strength of the infill concrete and the reinforcing bars inside the cores had no significant effect on the compression strength of the walls.
3. The failure load was governed by the eccentricity and the support conditions.

The axial strength of the fully concrete-filled GFRG walls was derived from the tests and is given in Fig. 6. The values in Fig. 6 are not the average values but the design values, and have a safety index of 3.0 for a confidence level of 90% in accordance with the Australian code. Theoretical calculations also showed that the Euler buckling load of the wall is higher than the test results but within the same order, the difference apparently stemming from imperfect test conditions, and in particular the inaccurate eccentricity, which was difficult to control.

5. In-plane flexural strength

In lateral load tests on a full-scale five-story GFRG building, the deformation of the building was of the flexural type at small deformations and a combination of the flexural and shear types at relative larger deformations [17]. These phenomena can be explained with Fig. 7, which shows the strain profiles of a GFRG wall for different cases. If the concrete infill cores are not connected by the GFRG wall, then there is no longitudinal connectivity between the concrete cores, and the structural system will deform in a shear type fashion similar to that occurring in a frame system. The corresponding strain profile for this case is shown in Fig. 7a. If it is assumed that the concrete cores are fully bonded to the GFRG, then the plane section assumption is applicable and the strain profile will be a straight line, as shown in Fig. 7b. In this case, a GFRG wall is similar to a reinforced concrete wall, and will therefore deform in a flexural type fashion. However, the bond between the concrete



(a) Test set up

b) Typical failure mode

Fig. 5. Axial load tests.

cores and the GFRG wall is limited. It is assumed that the maximum bond strength is achieved inside the wall at a curvature κ, after which the bond is overcome and relative slips between the concrete cores and the GFRG wall occur. When the curvature of the wall is increased to κ', as shown in Fig. 7c, the strain profile deviates from the straight line, as shown by the dashed lines. The difference in strains at a certain location then gives the slip strain between the adjacent concrete cores. The average strain profile can be shown as a continuous curve (the solid line in Fig. 7c). It can be seen from Fig. 7c that the strain profile is a combination of the curves in Fig. 7a and those in Fig. 7b. In other words, the deformation type will be a combination of flexure and shear.

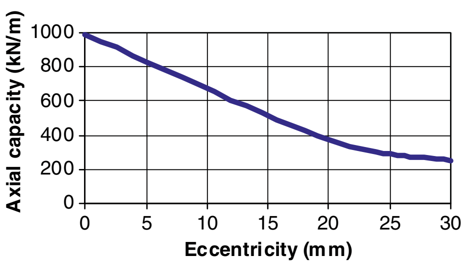
The slips at the interfaces are nonlinear and highly complicated, which makes the strain profile shown in Fig. 7c difficult to quantify and an analytical solution to the problem difficult, if not impossible, to find. As a result, numerical simulations must be used for quantitative studies. A 2D FEM model has been developed for the flexural analysis of concrete-filled GFRG walls [6], but this numerical method is only suitable for research purposes and is inconvenient for use in engineering design. Although accurate and simple design equations similar to those for the flexural design of reinforced concrete members are difficult to derive, approximate design equations can still be obtained from rational analyses of the structural system, and are developed and reported in this work.

5.1. Upper bound solution

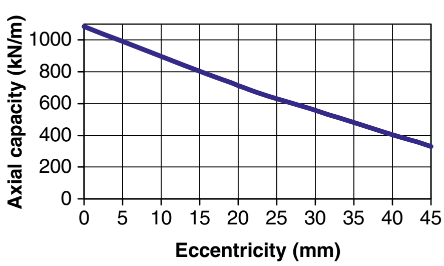
An upper bound solution for the in-plane ultimate moment of the wall can be obtained from the full plastic analysis of the section, as shown in Fig. 8a. The assumption of full plasticity is correct for the tensile resistance, as the tensile strength of the GFRG panel and the concrete is small and can be ignored, which means that only the reinforcement bars inside the concrete cores need to be considered. The compressive resistance of the wall is governed by its bulking strength, and for thin walls such as GFRG walls the compressive bulking is relatively ductile, as observed from experimental tests [2]. Therefore, a uniform distribution of the compressive stress can be assumed. From the balance of the forces in the cross-section, the depth of the compression zone x is calculated to be

$$x = \frac{N^* + D \cdot f_y \cdot a_s}{p \cdot t + f_y \cdot a_s}, \tag{2}$$

and the ultimate moment resistance is given by



(a) For two pinned supports



(b) For one pinned and one fixed support

Fig. 6. Axial load capacity of 2.9 m high concrete-filled GFRG walls.

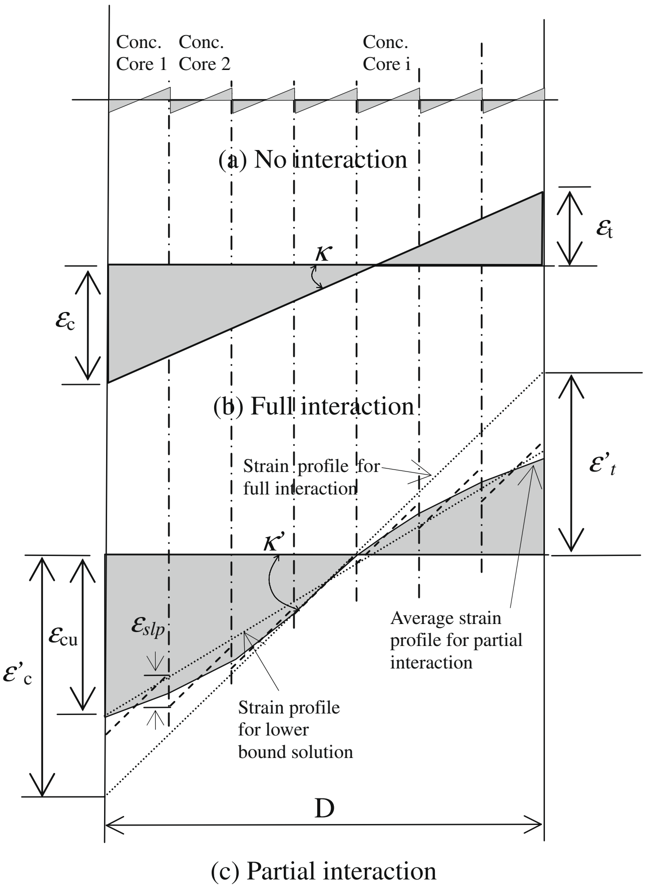


Fig. 7. Strain profiles.

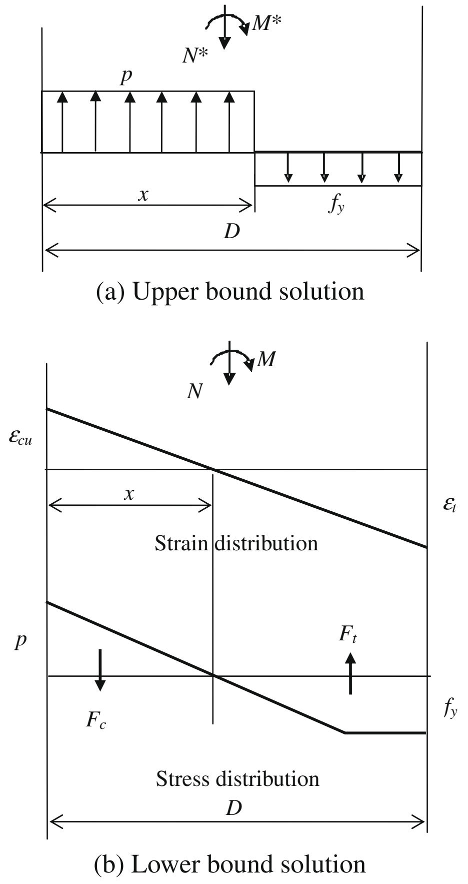


Fig. 8. Upper and lower bound solutions.

$$M^* = \frac{x}{2}(p \cdot t \cdot D - N^*), \tag{3}$$

where N^* and M^* are the axial force and the moment resistance of the upper bound solution, t is the thickness of the wall, D is the depth of the cross-section, f_y is the yield strength of the reinforcement bar, a_s is the area of reinforcement bar inside a unit length of wall, and p is the compressive stress of the wall at the axial buckling load.

5.2. Lower bound solution

A lower bound solution can be obtained from the cross-sectional analyses. As shown in Fig. 7c, the actual strain profile is nonlinear at the ultimate load due to the interfacial slips. A linear assumption of the strain profile where the ultimate compressive strain ε_{cu} is the same as that of the nonlinear strain profile, as shown in Fig. 7c, will underestimate the strain and hence the stress in the cross-section. As the slip is difficult to quantify, the linear strain profile can be used to calculate the lower bound solution that corresponds to the case of a perfect bound without slips. In this case, the ultimate strain at the extreme compression fiber can be calculated by

$$\varepsilon_{cu} = \frac{p}{E}, \tag{4}$$

where E is the effective Young's modulus of the wall and is given by

$$E = \frac{E_c \cdot A_c + E_R \cdot A_R}{A_c + A_R}, \tag{5}$$

in which A_c and A_R are the areas of the concrete and GFRG wall, respectively; and E_c and E_R are the Young's modulus of the concrete and the GFRG panel, respectively [2]. The flexural resistance can then be calculated with the same method as is used for reinforced concrete sections. However, it should be noted that the ultimate strain given by Eq. (4) is not a constant, but varies with the bulking strength p of the wall.

The typical strain and stress distributions for the lower bound solution are shown in Fig. 8b. With a linear strain distribution, the compression stress distribution is also linear, because the buckling stress is much smaller than the material strength. The tensile stress distribution may be linear or trapezoidal, depending on the axial force N. Before the yielding of the tensile reinforcement, the tensile stress is linear and the axial force is given by

$$N = F_c - F_t = p \cdot t \cdot x/2 - E_s \cdot \varepsilon_t \cdot a_s \cdot (D - x)/2, \tag{6}$$

where E_s is the Young's modulus of the reinforcing bars. From Eq. (6), we have

$$x = \frac{2N + E_s \cdot \varepsilon_t \cdot a_s \cdot D}{p \cdot t + E_s \cdot \varepsilon_t \cdot a_s}. \tag{7}$$

When the reinforcement at the extreme tension fiber just reaches the yield strain εy, the corresponding compression zone depth x_0 satisfies

$$x_0 = \frac{D \cdot \varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y}. \tag{8}$$

By letting $\varepsilon t = \varepsilon y$ in Eq. (7) and from Eq. (7) = Eq. (8), the following equation is obtained.

$$N_0 = \frac{D \cdot (\varepsilon_{cu} \cdot p \cdot t - \varepsilon_y \cdot f_y \cdot a_s)}{2(\varepsilon_{cu} + \varepsilon_y)}, \tag{9}$$

where N_0 is the threshold axial load that separates the linear and nonlinear distributions of the tensile stress. When the axial load satisfies $N \geqslant N_0$, the tensile stress distribution is triangular and the moment of the section is given by

$$M = F_c \cdot \left(\frac{D}{2} - \frac{x}{3}\right) + F_t \cdot \left(\frac{D}{2} - \frac{D-x}{3}\right). \quad (10)$$

The compression zone depth x can be calculated from the balance of axial forces in the cross-section. When $N < N_0$, the tensile stress block will be trapezoidal, and similar equations can easily be derived based on the stress block in Fig. 8b.

Although the lower bound solution underestimates the actual moment resistance of the walls, it can, and should, be used for the flexural design of concrete-filled GFRG walls. This is not only due to the difficulties involved in the quantification of the interfacial slips, but also due to the scattering in the roughness of the internal faces of the GFRG panels, which is found to be significantly different for panels made from different factories [6]. Experimental and numerical analyses have demonstrated that the roughness of the internal faces of the GFRG panel will significantly affect the slip behavior and hence the composite action of the walls [6].

5.3. Design example

A design example is provided in this section to demonstrate the design method of the lower bond solution.

Question: calculate the moment resistance of a 4 m long GFRG wall fully filled with concrete in all of the cores. One ∅8 mm mild steel bar with f_y = 250 MPa is provided in the middle of each concrete core. The applied ultimate axial load of the wall is 1600 kN; and the axial resistance of the wall is 900 kN/m.

Solution:

$$\text{As } E_R \cdot A_R \langle\langle E_c \cdot A_c, E = \frac{E_c \cdot A_c + E_R \cdot A_R}{A_c + A_R} \approx \frac{E_c \cdot A_c}{A_c + A_R} = \frac{24{,}400 \times 21{,}485}{250 \times 120} = 17475\,\text{MPa}.$$

$$p = \frac{900 \times 10^3}{1000 \times 120} = 7.5\,\text{MPa},$$

$$\varepsilon_{cu} = \frac{p}{E} = \frac{7.5}{17745} = 4.227 \times 10^{-4}, \text{ and}$$

$$N_0 = \frac{D \cdot (\varepsilon_{cu} \cdot p \cdot t - \varepsilon_y \cdot f_y \cdot a_s)}{2(\varepsilon_{cu} + \varepsilon_y)} = \frac{4000 \times (4.227 \times 10^{-4} \times 7.5 \times 120 - 1.25 \times 10^{-3} \times 250 \times 201 \times 10^{-3})}{2 \times (4.227 \times 10^{-4} + 1.25 \times 10^{-3})} = 379.8\,\text{kN}.$$

N = 1600 kN > N_0, the tensile stress profile is linear.

$$F_c = \frac{p \cdot t}{2} \cdot x, F_t = \frac{(D-x) \cdot \varepsilon_{cu}}{2x} \cdot E_s \cdot a_s \cdot (D-x),$$

from $F_c - F_t = N$, $x = 3556.6$ mm, therefore

$$M = F_c \cdot \left(\frac{D}{2} - \frac{x}{3}\right) + F_t \cdot \left(\frac{D}{2} - \frac{D-x}{3}\right) = 1304.4\text{ kN m}.$$

6. Design philosophy and procedure for GFRG buildings

GFRG buildings are a new type of structure with a history of less than 20 years. Nevertheless, significant experience has been accumulated through construction practice and research and development work in several countries. The development of a national design code of practice is underway in India and China, and a provincial design code of practice has been in effect since October 1, 2007 in Shandong Province, China [20].

On the one hand, the GFRG building system is different from the existing and conventional structural systems, and thus engineers need a separate and different design code. On the other hand, it would be unwise and impractical to develop a completely new design system from scratch. Therefore, the best way of developing a design code for GFRG buildings is to fit it into the existing design frameworks while incorporating the differences that are unique to GFRG buildings. However, it should be reiterated that care must be exercised in adopting design methods and guidelines directly from existing codes that are meant for reinforced concrete walls, as that may be inappropriate and sometimes even dangerous.

Based on many years of design, construction, and research work, the author recommends the following methodology for the design of GFRG buildings.

6.1. Design of walls under vertical loads

The design principle for GFRG walls under vertical loads is similar to that for reinforced concrete walls or plain concrete walls. For example, the eccentricity of the vertical loads can be decided with reference to the relevant reinforced concrete design codes and by treating a GFRG wall as a plain concrete wall. The vertical load capacity can then be decided from design charts or tables similar to those in Fig. 6. As buckling is the only failure mode, the compressive strength of the cross-sections is not applicable to GFRG walls.

The out-of-plane bending resistance of concrete-filled GFRG walls has been found experimentally to be relatively small compared with that for reinforced concrete walls [2]. Therefore, in the presence of significant out-of-plane moments, reinforced concrete walls can be used to replace GFRG walls. Similarly, when a significant point load exists, a reinforced concrete column should be provided to support the point load.

6.2. Distribution of lateral loads

Lateral loads can be distributed to individual walls based on their stiffness in a manner similar to that employed in the design of reinforced concrete walls. The stiffness of a GFRG wall can be calculated by treating it as a uniform, monolithic wall 120 mm thick with an equivalent modulus of elasticity given by Eq. (5).

It is important that the stiffness of a GFRG wall be calculated based on individual GFRG panels without a joint, because a joint in a GFRG wall is relatively weak and cannot provide a rigid connection between panels. Another reason is that the shear lag effect is very significant in GFRG walls due to the partial interactivity. In the case of L, T, I, or other shaped flanged sections, the flanges should be omitted and only the web considered in calculating the stiffness. T, L, or other shaped flanged sections are allowed in the design guidelines in [20], which is inappropriate and non-conservative.

6.3. Seismic design considerations

In seismic design, an important and difficult task is the determination of the response modification factor R_f. R_f factors in the existing design codes for various conventional structural systems were selected by committee consensus based on the following.

(a) The general observed performance of similar buildings during past earthquakes.
(b) Estimates of general system toughness.
(c) Estimates of the amount of damping present during inelastic response.

Hence, there is little technical basis for the values of R_f in the existing design codes. Currently, a full review and comprehensive research work are underway in the United States to develop a rational means for evaluating the R_f factors [22]. As GFRG buildings are a new type of structure, a reasonable choice of R_f factor can only be made by comparing the GFRG building system with traditional structures, such as reinforced concrete wall building systems for which the response modification factors are already available.

GFRG walls are composite members with partial interaction, and the ductility of a partially interactive member is generally greater than that of a fully interactive reinforced concrete member [2]. In terms of strength reserve, it is recommended that the safety margin adopted for the design of GFRG walls be at least as large as or greater than those adopted in the respective code for concrete structures. Therefore, it is not unreasonable to treat buildings constructed with GFRG walls as reinforced concrete shear wall structures and to adopt the R_f values from the respective code of practice.

The structural period of vibration for GFRG buildings may also be determined by treating a GFRG wall as a conventional reinforced concrete walls. As a GFRG wall is more flexible than a similarly shaped reinforced concrete wall due to the partial interactivity, GFRG buildings have a longer period of vibration than similar reinforced concrete walled buildings. Therefore, a further safety margin is included in the calculation of the design base shear for normal multi-story GFRG buildings by treating them as conventional reinforced concrete shear-wall buildings.

6.4. Design procedure

A GFRG building supporting lateral loads can be designed using the following procedure.

1. When seismic loads are applicable, the base shear method is used, and the structural response modification factor R_f is selected by treating the concrete-filled GFRG walls as reinforced concrete walls.
2. The total shear force is allocated to each wall in proportion to its stiffness.
3. The shear strength of the walls is checked for all load cases to ensure that a shear failure does not occur. Sliding shear should also be checked using the conventional method for reinforced concrete walls.
4. The in-plane ultimate flexural strength of the walls is checked using the method provided in Section 5.2. It should be noted that in calculating the flexural strength, the compressive strength of the wall should be determined based on the axial strength of the wall as described in Section 4 (and not on the cross-sectional strength).
5. Checking the lateral deflection is usually not an issue for a GFRG building provided that the height/breadth ratio in any direction is less than two. It is not recommended that GFRG walls be used to support lateral loads alone in buildings taller than eight stories at present, but this limitation may be removed in the future when more construction experience has been accumulated. GFRG walls bearing a vertical load only may be used in taller buildings provided that other structural systems such as reinforced concrete shear walls are deployed to support the lateral wind and seismic loads.

7. Conclusion

This paper has introduced GFRG walls and the associated building system, with the structural characteristics of that system described. Experimental and theoretical investigations undertaken by the author since 2002 have been presented from the structural element and overall building performance points of view.

The accurate calculation of the in-plane flexural strength of GFRG walls is a difficult, if not impossible, task due to the relative slips between the infill concrete cores and the GFRG panel. An upper and a lower bound solution for the in-plane flexural strength have been presented, of which the lower bound solution is most suitable for design use. Based on the results of the experimental and theoretical investigations and on a rational analysis of the existing design frameworks, a methodology and associated procedure for the design of GFRG buildings has been offered.

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