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Behaviour of composite walls under monotonic and cyclic shear loading

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Abstract. The novel form of composite walling system consists of two skins of profiled steel sheeting with an in-fill of concrete. Such walling system can be used as shear elements in steel framed building subjected to lateral load. This paper presents the results of small-scale model tests on composite wall and its components manufactured from very thin sheeting and micro-concrete tested under monotonic and cyclic shear loading conditions. The heavily instrumented small-scale tests provided information on the load-deformation response, strength, stiffness, strain condition, sheet-concrete interaction and failure modes. Analytical models for shear strength and stiffness are derived with some modification factor to take into account the effect of quasi-static cycling loading. The performance of design equations is validated through experimental results.

Key words: composite wall; monotonic; cyclic; concrete; profiled sheeting; strength; stiffness.

1. Introduction

The novel form of composite walling system (Wright *et al.* 1992, Gallocher 1993, Wright *et al.* 1994, Wright and Evans 1995) comprises vertically aligned profiled steel sheeting and an infill of concrete as shown in Figs. 1(a)-(b). Composite walling has many advantages when used in conjunction with composite flooring and is thought to be especially applicable to shear or core walls in steel framed buildings. It has potential in concrete buildings, basements and blast resist structures. The advantages of this system arise from the type of construction where profiled steel sheeting acts as a formwork for in-fill concrete. In addition, it also acts as a bracing system to the building frame (Hossain and Wright 1995) against wind and destabilising forces in the construction stage. In the service stage, profiled steel sheeting also acts as reinforcement.

Previous research was concentrated on the axial load behaviour of such composite walls (Wright and Gallocher 1995, Wright 1998, Hossain 2000, Bradford *et al.* 1998). Concern about the performance of composite walls under axial loading (Wright and Gallocher 1995, Wright 1998,

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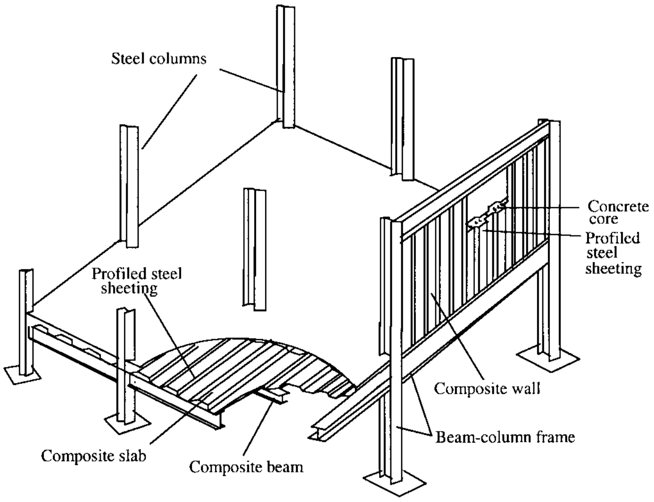


Fig. 1(a) Schematic of composite wall in a building

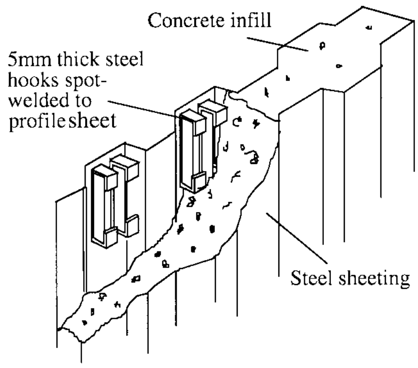


Fig. 1(b) Schematic of a composite wall with additional shear connection

Hossain 2000) was associated with the difficulty in the transfer of load between the steel skins and the concrete core, the buckling of the steel sheeting in the flanges and the reduced capacity of the concrete core due to profiling. Axial loading caused brittle failure at the interface, with complete loss of chemical bond, without sufficient strain to mobilise the strong ductile force developed by the embossments. Embossments are interlocking devices of various geometric shapes, which are pressed into the surface of the ribs, and crests of the profiled steel sheeting to provide longitudinal and transverse shear resistances at the steel-concrete interface. The problem of load transfer was overcome by using additional shear connection devices at the head and foot of the wall. The effective connections between pair of sheeting were achieved by steel hooks spot welded to the pair of sheeting and tied together through concrete by steel stirrups (Fig. 1b).

The information produced during the full scale tests (Wright and Gallocher 1995, Wright 1998) had shown that the reduction in axial capacity was closer to 30% rather than the nominal 10% allowed to account for imperfection and nominal eccentricity of loading in B.S.8110 (1985). Taking into account these factors, a design equation for the axial capacity of composite walls subjected to nominally concentric loading was developed (Wright and Gallocher 1995, Wright 1998).

The design criteria associated with composite walling includes its axial and lateral load resistance and design guidelines are not currently available in any specification. The main objective of this research is to study the overall behaviour of this walling system and to identify their potential application as shear or core walls in steel framed buildings. The behaviour of such walls under monotonic and cyclic shear loading conditions is different from its axial behaviour (Hossain and Wright 1998a). This paper presents the results of small-scale model tests on composite wall and its associated components (concrete core and profiled steel sheeting) under monotonic and cyclic shear loading conditions. It also reports the development of analytical models for strength and stiffness and their performance validation through experimental results.

2. Research programme

The theoretical and experimental investigations were based on the concept that the composite walls resist shear loading in three ways: i. shear resistance of the profiled steel sheeting as a skin, ii. shear resistance of the concrete core and iii. shear resistance arising from the interaction of the sheeting and core. Consequently both theoretical and experimental investigations had concentrated upon the individual behaviour of the component parts before considering the composite wall as a whole.

2.1 Experimental investigation

Small-scale model tests of approximately 1/6th scale were carried out on three specimen types (1) profiled steel sheeting, (2) concrete core and (3) their combination representing a composite wall. Eight tests on profiled micro-concrete panels, six tests on profiled sheet panels and six tests on composite walls were conducted to provide information on the shear strength, shear stiffness, strain conditions and failure modes under monotonic and cyclic shear loading conditions.

The model panels had an overall dimension of 620 mm × 620 mm that provided an effective dimension of 560 mm × 560 mm. The model profiled steel sheets with no embossments were manufactured in-house from plain sheeting of 0.45 mm thickness conforming to the model scale by using an especially fabricated fly press. A gap-graded micro-concrete (Hossain and Wright 1998b) was used to manufacture the infill concrete core. The detail of a typical model composite wall specimen showing the profiled sheeting and concrete core is shown in Fig. 2(a).

A shear rig (Fig. 2b) had been designed (Hossain and Wright 1998a,b) and fabricated to impart monotonic and cyclic shear loading conditions in the model panels. The test panels (abcd) were connected to the test frame using 10 mm bolts and panels were tested by applying tension or compression forces along the diagonal (Figs. 2b-c). The shear load-deformation response as indicated in transformed shear simulation (ab’c’d’) was obtained from the corresponding diagonal

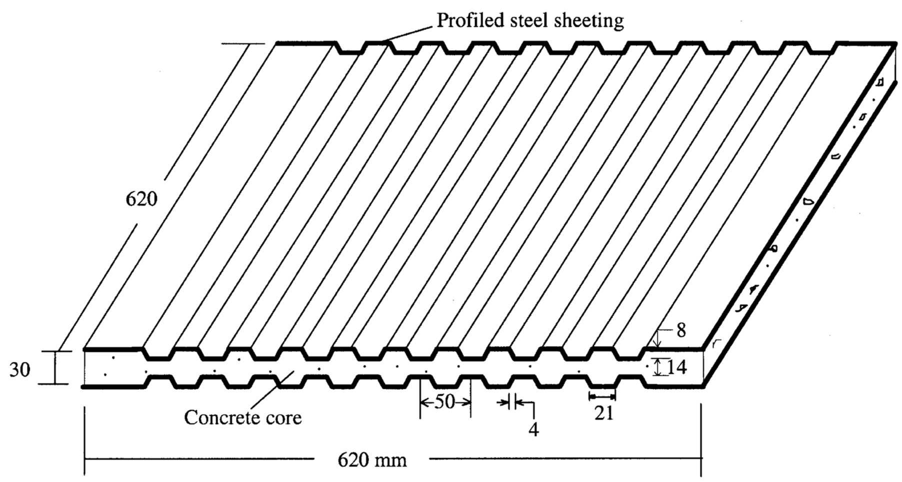


Fig. 2(a) Detail of composite wall, profiled steel sheeting and concrete core

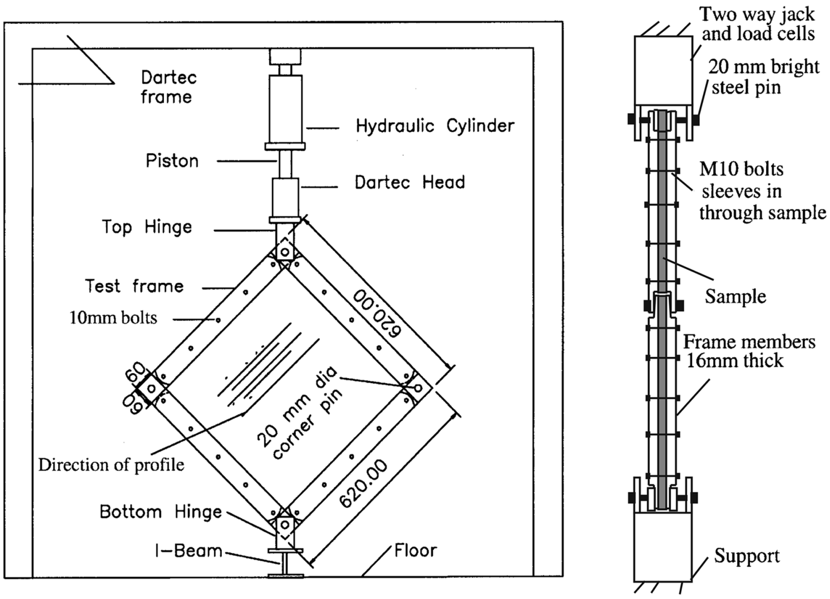


Fig. 2(b) General detail of shear rig

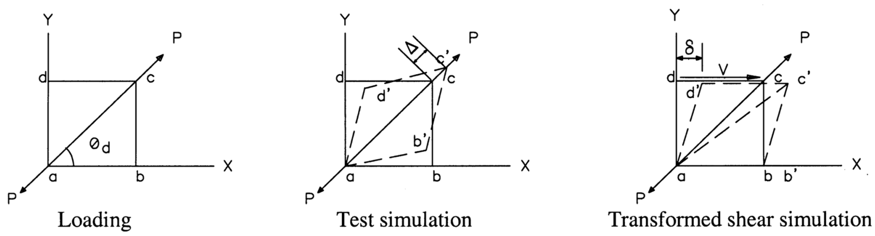


Fig. 2(c) Load and deformation realisation in the shear rig

load-deformation response indicated in the test simulation (ab'c'd') by using Eq. (1).

The diagonal force P and corresponding diagonal deformation Δ can be related to the panel shear force V and shear displacement δ by Eq. (1) (Fig. 2c):

$$V = P\cos\theta_d \quad \text{and} \quad \delta = \frac{\Delta}{\cos\theta_d} \tag{1}$$

2.1.1 Casting, curing, test conditions and instrumentation

During casting, the micro-concrete was compacted on a vibrating table in different layers. Control specimens in the form of cylinders and cubes were cast at the same time. After four or five days the panels were removed from the mould and then cured in air until they were tested. Control specimens were taken out of the mould after 24 hours and then cured in air.

Before testing, the panels were assembled in the test frame and strain gauges (rosettes and single gauges) were installed at key locations on the panel surface. To simulate fully clamped boundary condition and to secure proper transfer of force from the frame to the panel, profiled gaps between specimen boundary and the frame were filled with resin filler (Hossain and Wright 1998b).

All model panels were connected to the test frame through intermediate bolts (Fig. 2b). Bolts also provided the mechanical connection between pair of sheeting and concrete core in case of wall specimens. The test frame assembly was then connected to the loading frame through the corner pins along one of the two diagonals. The schematic of the experimental set-up with a panel is shown Fig. 2(b).

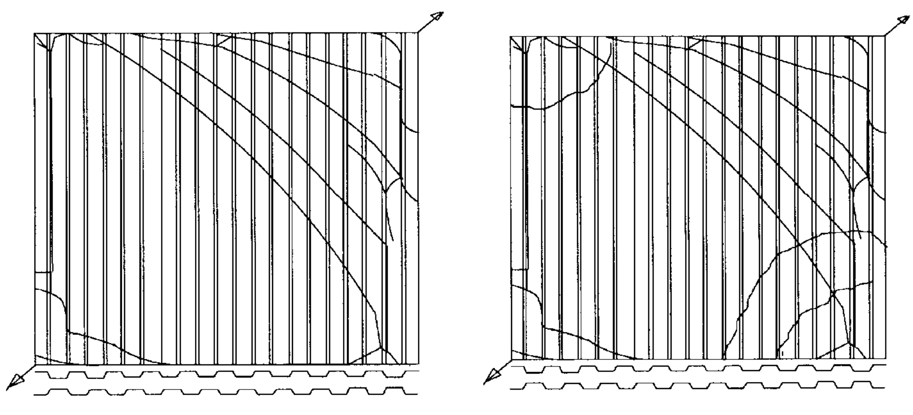
2.1.2 Testing, test observation and failure modes

Tests were performed by applying tensile or compressive forces along the diagonals of the panels (Fig. 2b). LVDT's and dial gauges were used to record the diagonal load-deformation response. The loads were applied in increments and at each load increment diagonal load-deformation and strains were monitored through the data logger.

Monotonic shear behaviour

The panels were tested by applying tensile or compressive force along the loaded diagonal. For the profiled concrete panels, first cracking load, failure load and crack patterns were recorded. A series of cracks parallel to the off-diagonal were gradually developed one after the other during loading. Cracks parallel to the corrugation profile were also found to develop near the boundary frame along the trough where the thickness of the panel is smaller. The development of cracks parallel to the off-diagonal (Fig. 3a) represented to a great extent the pure shear condition within the panel.

For profiled sheeting, the failure was mainly due to buckling of the sheeting. No yielding or tearing of the sheet at intermediate bolts along the boundaries was observed. Post-buckling behaviour was characterised by the formation of localised tension field or buckles parallel to the direction of the applied load at trough or crest sections of the profiles. Local buckling seemed to be restricted to the plane part of the folds of the cross-section. The local tension field, extended with the increase of load and the extended tension field while crossing the folds, forced the sheeting to



(a) Monotonic shear loading (b) Cyclic loading

Fig. 3 Crack pattern in profiled concrete panels

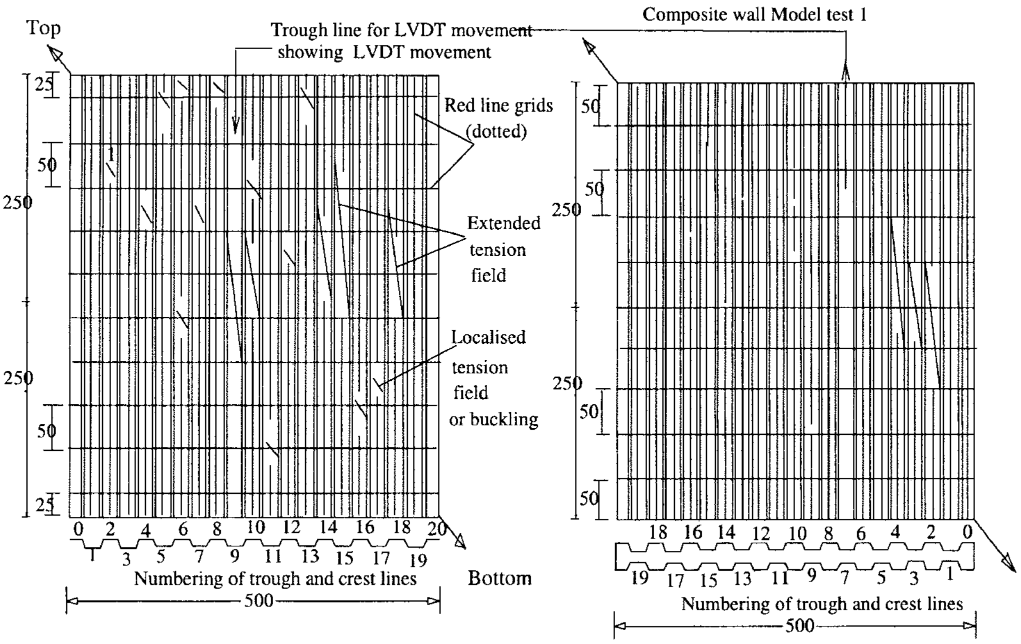
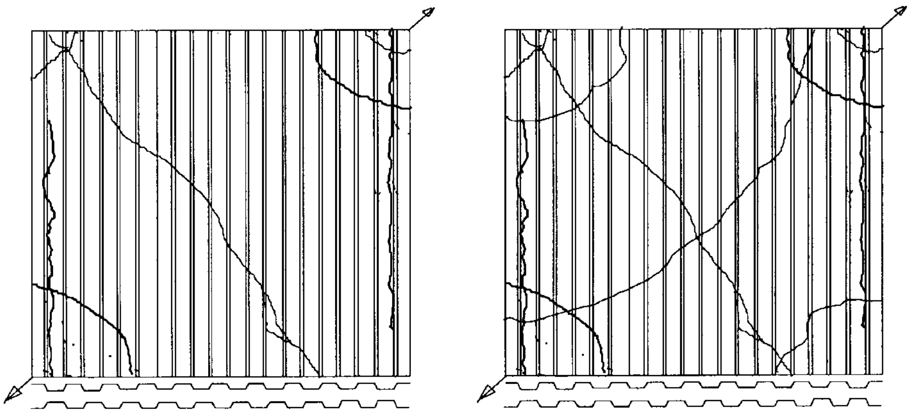


Fig. 4(a) Failure of steel sheeting Fig. 4(b) Failure of sheeting in wall

lose its geometric shape. As a result the sheet yielded and lost its stiffness very rapidly in the post-buckling stage. The tension field action was limited to either trough or crest sections and the webs of the profile acted as a stiffener preventing the extension of tension field over the whole panel. The trough or crest profiles acted as stiffening plates accommodating a tension field entirely in their own territory. The resulting action significantly enhanced the overall buckling capacity of profiled steel panels. At the failure stage, tension fields were extended over some length (Fig. 4a) of the trough and crest profiles accompanied by severe distortion and bending of profile with the loss of profile geometry. The failure was sudden due to the sudden transition to tension field action.

The behaviour of the composite wall was found to be dependent on the interaction between sheeting and concrete core. The interface connection between sheeting and concrete was derived only from chemical bond due to the absence of embossments in the sheeting. The chemical bond between sheeting and concrete can be considered as negligible. The failure of the composite wall, after initial stages of debonding due to failure of the chemical bond, was started with visual signs of buckling of sheeting locally and progressive outward buckling of sheeting from the concrete. The sheeting buckled outward from the concrete and slid over the profiled core of concrete. In this process, it formed a tension field extending over some length (Fig. 4b) similar to that described in individual profiled steel sheeting behaviour. In the final stages, the sheeting slid over the profiled concrete core and an extended tension field caused the sheeting to twist and eventually the sheet lost its profiled geometry. The transition from the first sign of buckling to failure was very quick resulting in a sudden failure of the panel. The crack pattern (Fig. 5a) in the concrete core was similar to that of the profiled concrete panel (Fig. 3a) representing the development of diagonal tension. The composite wall showed much higher ductility than its components.



(a) Monotonic shear loading (b) Cyclic loading

Fig. 5 Crack pattern in concrete core in composite wall

Cyclic shear behaviour

The panels were tested by applying alternate tensile and compressive forces along the loaded diagonals. The cyclic application of tensile and compressive force along the loaded diagonal was continued until the failure of the panels.

Profiled concrete panels failed due to the formation of cracks parallel and perpendicular to the loaded diagonal as shown in Fig. 3(b) due to cyclic loading. The failure of the profiled steel sheet panels was due to the formation of extended tension fields and localised buckling similar to that of monotonic shear loading condition (as shown in Fig. 4a). For composite walls, several cycles of loading were applied and the load was increased at an increment of 6 kN in each cycle up to a load of ± 60 kN. Beyond 60 kN, the load was increased at an increment of 30 kN until failure of the panel. The cracking load of concrete, buckling load of sheeting and failure load of the wall were recorded. The buckling and failure of sheeting (Fig. 4b) were similar to that of the wall under monotonic shear loading condition. The crack patterns in concrete as shown in Fig. 5(b) were also similar to those of concrete core under cyclic loading (Fig. 3b).

2.1.3 Strain characteristics

Monotonic shear

A typical variation of strain along the off-diagonal (gauges 2, 8 and 9) for wall test 2 is presented in Fig. 6(a). The variation was marked by the abrupt change in strain at several loading stages. This was due to the initial cracking and subsequent progressive cracking of concrete and interface characteristics between sheeting and concrete. The strain gauges showed tensile strain along the off and compression strain along the loaded diagonal throughout the loading history. This confirmed the mechanism of diagonal tension and compression state within the panel. The lower strains in crest gauges 12 and 14 confirmed the presence of higher stress in trough sections. The strains at the loaded and off-loaded corners were higher than those at the centre. The strains in the steel reached the yield only after the buckling of the sheeting.

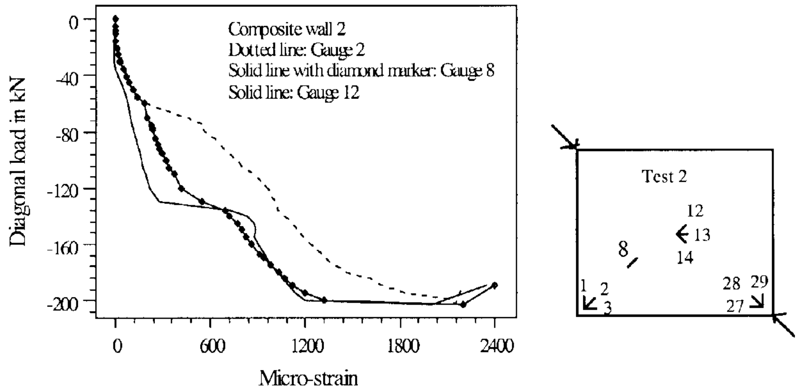


Fig. 6(a) Typical variation of diagonal strain

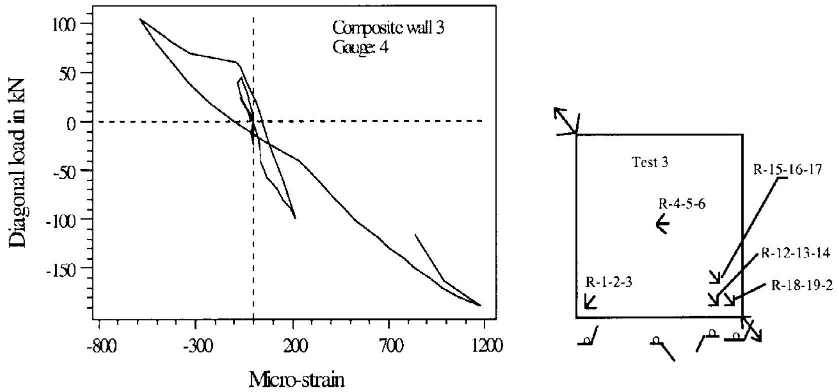


Fig. 6(b) Hysteretic loops for diagonal strains in test 3

Cyclic shear loading

The diagonal strains under cyclic loading from test 3 are presented in Fig. 6(b). It was found that the strain reversal did not follow the same path due to cyclic application of tension-compression along the loaded diagonal creating distinct loops. This was due to the nonlinear steel-concrete interface behaviour resulting from debonding and mechanical friction, cracking of concrete and buckling of sheeting. Cyclic loops confirmed the mechanism of diagonal tension and compression state within the panel as found in the monotonic shear tests.

2.2 Development of analytical models

Composite walls are assumed to resist shear loading in three ways: shear resistance of the sheeting, shear resistance of the concrete core and from the sheet-concrete interaction. For the development of analytical models for the strength and stiffness, let us consider a composite wall in a practical building frame as shown in Fig. 7. The wall is connected to the building frame by sheet-frame fasteners. In practical circumstances, to construct such walls several steel sheets may be necessary and they should be connected together through seem fasteners. Let us consider b and a be

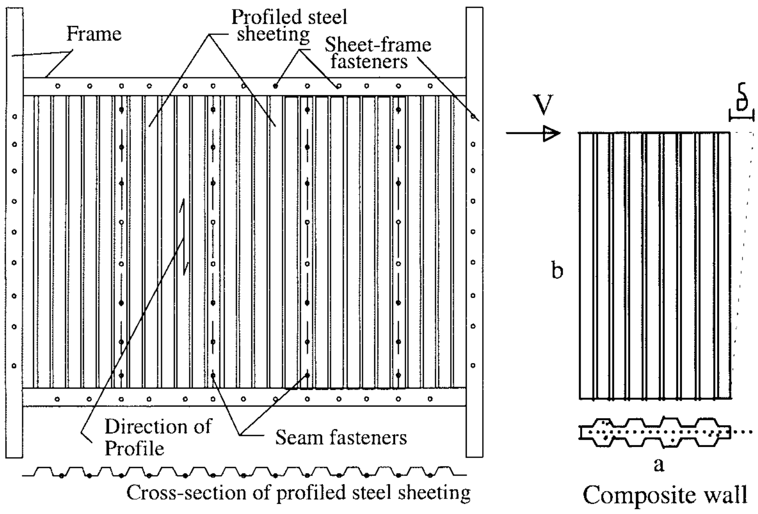


Fig. 7 Schematic of a framed composite wall under in-plane shear

the height and width of the wall respectively. The applied shear load and shear deformation are represented by V and δ respectively.

2.2.1 Analytical model for the stiffness of the composite wall

Shear stiffness of the profiled concrete core

The derivation of analytical model includes the idea of transforming the profiled concrete core into an equivalent plain concrete core of rectangular cross-section having an average thickness of t_{eq}. This simplified the problem and was used by Davies and Fisher (1979) and Easterling and Porter (1994a,b) successfully in analysing steel-deck-reinforced concrete diaphragms. The shear stiffness of the profiled concrete core is derived (Hossain and Wright 1998b) based on the strain energy approach. The shear flexibility (c_c) and stiffness (k_c) of the concrete core is expressed as:

$$c_c = \frac{1}{k_c} = \frac{2b(1 + \nu_c)}{E_c a t_{eq}} \tag{2}$$

where ν_c and E_c are the Poisson's ratio and elastic modulus of concrete respectively.

Shear stiffness of the profiled steel sheeting

The shear flexibility (c_s) of the sheeting in composite wall is the displacement per unit shear load (δ/V) applied normal to the corrugation profile as shown in Fig. 8. The total shear flexibility of the profiled sheet, c_s, can be taken as the sum of terms, each for one of the various factors involved (Wright and Hossain 1998). The main components considered are due to: shear deformation of sheet (c_1), bending or distortion of corrugation profile (c_2), axial deformation of the boundary frame members (c_3) and local deformation of sheet at the sheet-frame and seam connections (c_4 and c_5).

$$c_s = c_1 + c_2 + c_3 + c_4 + c_5$$

- for the case of model composite walls, the confining effect of the concrete (as confirmed from model tests) eliminates c_2.

- at seams between adjacent steel sheets for practical construction, the concrete carries almost all the shear forces and, therefore, c_5 may be ignored. In model tests, there was no seam connection as single sheet was used.
- if the frame is considered to be very rigid, the axial deformations in frame members can be considered negligible, which eliminates the factor c_3.
- if the connection details are such that the local deformation of sheeting is not allowed in sheet-frame fasteners than the factor c_4 can be omitted. This is the case for model tests where load is applied through both steel and concrete.

So the stiffness of the sheeting in model tests by applying strain energy approach can be written as (Wright and Hossain 1998):

$$c_s = \frac{1}{k_s} = c_1 = \frac{2\alpha b(1+\nu_s)}{E_s a t_s} \tag{3}$$

where t_s, ν_s and E_s are the thickness, Poisson's ratio and elastic modulus of steel respectively. α is the ratio of the developed length of a profile to its projected length.

Stiffness of the composite wall

The boundary frame in the model tests is considered to be formed by infinitely rigid elements pinned together at the corners and induces pure shear on the infill panel. No bending or distortion of the corrugation profile is allowed due to boundary condition and also the infill concrete acts as stiffener causing the flat cross section of the steel sheets to remain flat. This was confirmed from the model tests where sheet distortion was found to occur at the ultimate stages of loading associated with buckling of the sheeting.

The stiffness of the composite wall is derived from the shear deformation of sheeting (Eq. 3), shear deformation of concrete core (Eq. 2) and from their degree of composite interaction. The composite flexibility (c_w) and stiffness (k_w) of composite walls can be derived as the sum of flexibility and stiffness of the double skins of sheeting and concrete core:

$$c_w = \frac{1}{k_w} = 2c_s + c_c = \frac{2}{k_s} + \frac{1}{k_c} = \frac{2\alpha b(1+\nu_s)(1+\nu_c)}{a[E_c t_{eq}\alpha(1+\nu_s) + 2E_s t_s(1+\nu_c)]} \tag{4}$$

2.2.2 Analytical model for the shear strength of composite wall

The shear strength of composite wall is controlled by the diagonal tension failure of concrete, buckling of sheeting, the shear-transfer mechanism at steel-concrete interface and failure of wall-frame connections (Hossain and Wright 1998a). The actual post-cracking behaviour of the wall is a combined phenomenon of wall-frame connection and sheet-concrete interaction. The boundary connections should be rigid enough (as is the case for model tests) to induce the failure in the wall panels. The rigid boundary connections increase the sheet-concrete interaction by keeping sheet-concrete-sheet sandwich intact until buckling of the sheeting commences. The practical use of such walls in conjunction with the building frame is to increase the shear resistance of the frame under monotonic or cyclic loading. In such cases, the frame failure (which may lead to the total collapse of the building) is not desired and the failure of the infill wall governs the design.

Shear strength of the concrete core

The analytical model for the shear strength of concrete core has been derived based on bi-axial stress conditions in the concrete (Kupfer and Gerstle 1973). The model (Hossain and Wright 1998b) includes diagonal tension limit state, which is the normal phenomenon in monotonic or cyclic shear loading condition as the failure criteria. The model also includes the idea of transforming the profiled concrete core into an equivalent plain concrete core of rectangular cross-section having an average thickness of t_{eq} (Davies and Fisher 1979, Easterling and Porter 1994a,b). The shear strength of the profiled concrete core (V_c) can be written as:

$$V_c = \frac{b t_{eq} f'_c \cdot f'_t}{f'_c + f'_t} = 0.074 a t_{eq} f_{cu} \tag{5}$$

where f'_c, f_{cu} and f'_t are cylinder, cube and splitting tensile strength of concrete.

Shear strength of the profiled steel sheeting

The type of failure either buckling of sheeting or failure at sheet-frame connections depends on the boundary conditions. If the sheet-frame connection is sufficiently rigid then the failure will be due to buckling of the sheeting. The ultimate shear resistance of the sheeting for failure in the sheet-frame fasteners and also for elastic buckling mode of failure in case of rigid connections are derived (Wright and Hossain 1998). For the case of buckling mode of failure as observed in model tests, the general critical buckling formula suggested by Easley (1975) based on othotropic model can be used and the shear resistance of the sheeting can be written as:

$$V_s = 36\beta \frac{D_x^{1/4} D_y^{3/4} a}{b^2} \tag{6}$$

where D_x and D_y are orthotropic constants for the profiled steel sheeting. β is a co-efficient ranging between 1.00 and 1.72, depending on the boundary conditions. β = 1.00 for simply supported conditions and β = 1.72 for clamped conditions (Easley 1975, Wright and Hossain 1998).

Shear resistance of the composite wall

Model tests on profiled steel sheet, profiled concrete and composite walls have revealed that the ultimate shear capacity of the composite wall can be conservatively obtained from the summation of individual shear resistances of sheeting and concrete core. The ultimate shear resistance of the composite wall for monotonic shear can be derived as:

$$V_w = 72\beta \frac{D_x^{1/4} D_y^{3/4} a}{b^2} + 0.074 a t_{eq} f_{cu} = 72\beta \frac{D_x^{1/4} D_y^{3/4} a}{b^2} + \frac{f'_c f'_t a t_{eq}}{f'_c + f'_t} \tag{7}$$

The first term in Eq. (7) represents sheet resistance (from Eq. 6) and the second term represents profiled concrete core resistance (from Eq. 5). The possible increase in concrete core capacity due to composite action and the resistance of the sheeting in contact with concrete are not taken into account. This makes Eq. (7), conservative. The influence of local buckling is not included, as it is not found important in pre-buckling stages of the sheeting and in the case of composite wall.

3. Validation of analytical models under monotonic and cyclic loading

3.1 Strength and stiffness of profiled concrete panels

Analytical and model test results for strength and stiffness are summarised in Table 1. The slopes of the initial and re-loading part of the shear load-deformation responses (Figs. 8a-b) are used to determine the experimental stiffness of the panels. The experimental pre-cracking stiffness of the panels under monotonic and cyclic loading is found to be in good agreement and cyclic loading seems to have no effect. The cracking load of panels with cyclic loading is decreased by about 30% compared with monotonic shear panels (Table 1). The cyclic loading also reduces the ultimate shear load by about 28% compared to monotonic loading.

For monotonic shear condition, the ratio of test to analytical stiffness ranges between 0.80 and 0.91. The corresponding strength ratio ranges between 1.12 and 1.26. They are supposed to be in reasonable agreement. Analytical Eqs. (2) and (5) can therefore be used safely to predict the shear stiffness and ultimate strength of profiled concrete panels under monotonic loading.

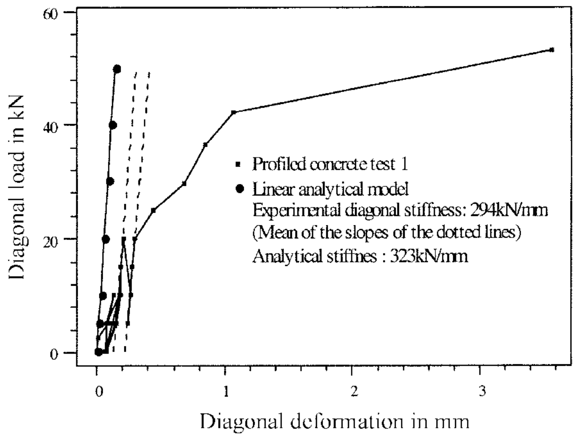


Fig. 8(a) Typical load-deformation response of profiled concrete panels (Monotonic shear)

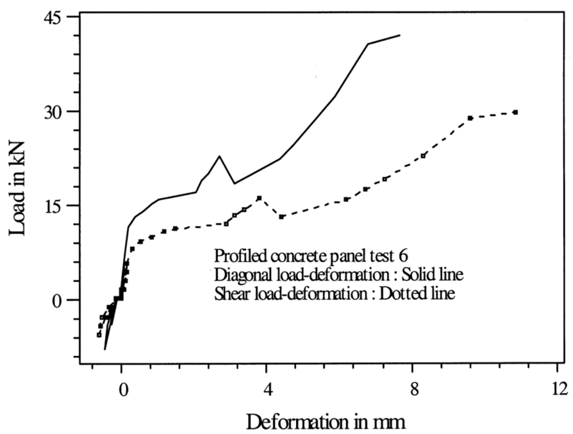


Fig. 8(b) Typical load-deformation response of profiled concrete panels (Cyclic shear)

The design Eq. (5) derived for monotonic shear loading condition seems to overestimate the ultimate load for cyclic loading condition (ratio of test to analytical ranges between 0.78 and 0.85). Eq. (5) is therefore, modified to Eq. (8) where a factor, ϕ, is introduced to take into account the reduction in strength due to cyclic loading.

$$V_c = 0.074\phi b t_{eq} \cdot f_{cu} = \phi \frac{f_c' f_t' a t_{eq}}{f_c' + f_t'} \tag{8}$$

The value of ϕ, can be varied from 0.73 to 0.80. Eq. (8) can safely (Table 1) be used to predict the strength of profiled concrete panels under cyclic loading (as the ratio of test to analytical model ranges between 1.03 and 1.07 (Table 1).

Table 1 Strength, stiffness and design equation validation for profiled concrete panels

Test No.	Concrete strength MPa $f_{cu}(f'_t)$	Cracking load kN	Shear stiffness kN/mm			Ultimate shear load kN		
			Test	Analytical Eq. (3)	Ratio	Test	Analytical Eq. (5)	Ratio
Monotonic loading								
1	21 (2.3)	15.5	147	161	0.91	33	27	1.26
2	24 (2.3)	13.5	+	161	-	31	27	1.15
4	24 (2.5)	13.4	129	161	0.80	31	27.6	1.12
5	23 (2.2)	13.8	132	161	0.82	32	27.2	1.18
Cyclic loading								
3	24 (2.4)	9.0	+	161	-	22	26.8 21.5*	0.82 1.03*
6	25 (2.5)	9.2	141	161	0.88	23	26.89 21.5*	0.85 1.07*
7	24 (2.4)	9.0	132	161	0.82	22	26.8 21.4*	0.78 1.03*
8	25 (2.3)	8.8	127	161	0.79	22.5	26.6 21.3*	0.85 1.06*

*Using modified Eq. (8) for cyclic loading (ϕ = 0.80)
+panel damaged due to machine malfunction

3.2 Strength and stiffness of profiled steel sheet panels

Figs. 9(a)-(b) show typical load-deformation responses of profiled steel sheet panels under monotonic and cyclic loading conditions. The sudden drop in load with large deformation after the peak load is due to the very sudden transition (snap-through) to tension field action and the substantial increase in associated deformation (as can be seen from the monotonic behaviour in Fig. 9a). This is the reason why buckling load is considered as the design load in the proposed model (Eq. 6) and post-buckling shear reserves between buckling and peak load are not considered. This will prevent the risk of sudden snap-through type failure in such panels. This also reflects the recommendations of Luo and Edlund (1996) and the ASCE-AASHTO Task Committee (ASCE-AASHTO 1977).

The experimental pre-buckling load and stiffnesses as well as ultimate loads are summarised in Table 2. The buckling and ultimate loads for panels under cyclic loading are decreased by about 9% and 18% respectively compared with those under monotonic loading. The pre-buckling stiffness is also reduced by about 8%. The ratio of ultimate load to buckling load is decreased from about 1.24 (for monotonic) to 1.12 (for cyclic). This means that post-buckling shear reserve is decreased by about 9% due to cyclic loading. The deformation at ultimate load (peak load) is also found higher for panels with cyclic loading (Fig. 9b).

Eq. (3) is found to over predict the pre-buckling stiffness as the mean ratio of experimental to predicted values are found to be 0.94 and 0.86 for monotonic and cyclic loading respectively. However, Eq. (3) can reasonably predict the stiffness of the panels under both cyclic and monotonic

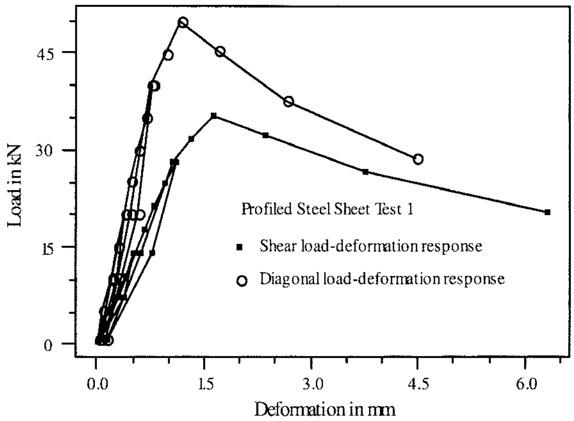


Fig. 9(a) Typical load-deformation responses for profiled steel sheeting (Monotonic shear)

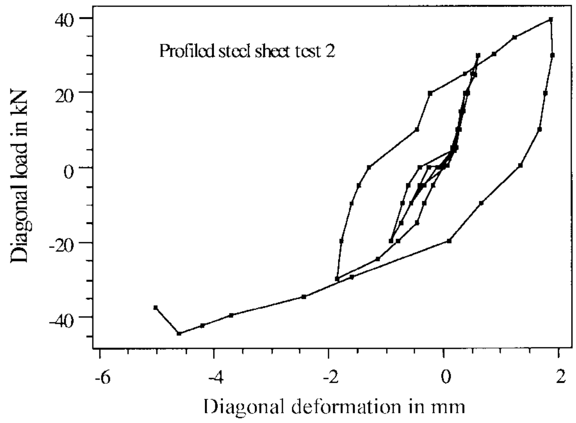


Fig. 9(b) Typical load-deformation responses for profiled steel sheeting (Cyclic shear)

Table 2 Comparison of analytical and experimental results

Type	Test shear load (kN) Buckling (1)	Ultimate (2)	Ratio (2)/(1)	Shear stiffness (kN/mm) Analytical (Eq. 3)	Test	Design shear (kN) Eq. 6 (3)	Predicted β (1)/(3)
Monotonic loading							
Test 1	29.7	36.8	1.24	30.3	28.3	17.23β	1.72
Test 3	30.1	37.2	1.24	30.3	28.0		1.75
Test 5	30.6	38.0	1.24	30.3	28.6		1.77
Cyclic loading							
Test 2	28.2	31.8	1.13	30.3	26.5	17.23β	1.64
Test 4	27.1	30.5	1.13	30.3	26.1		1.57
Test 6	27.0	29.6	1.10	30.3	25.6		1.56

Predicted mean value of β: 1.74 (for monotonic loading) and 1.59 (for cyclic loading)

loading conditions. The predicted mean value of 1.74 for β correlates very well with the value of 1.72 recommended by Easley (1975) for panels under clamped boundary condition (Table 2). For cyclic loading, a value of 1.60 for β can be suggested for panels with clamped boundary conditions. The strength of the panels can be predicted safely by using Eq. (6) using β = 1.72 for pure monotonic shear and β = 1.60 for cyclic shear loading conditions.

3.3 Strength and stiffness of composite walls

Typical load-deformation responses of profiled sheeting, concrete core and composite wall under monotonic loading are superimposed in Fig. 10. The composite wall exhibits higher strength-stiffness and ductility than its components and similar behaviour is also observed for cyclic loading. The failure load and stiffness of composite wall are around 30% and 22% higher than the sum of respective contributions from pair of sheeting and concrete core.

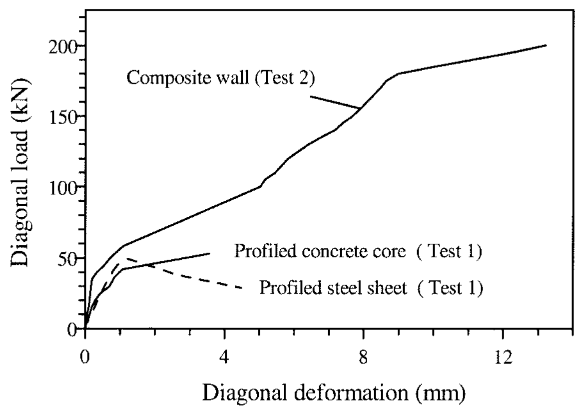


Fig. 10 Comparative study of load-deformation responses

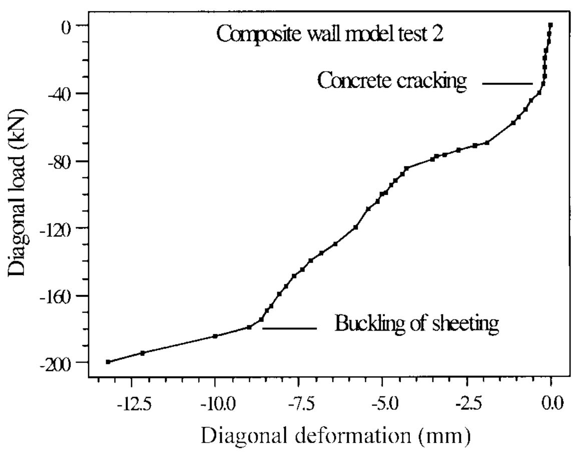


Fig. 11(a) Typical load-deformation responses for composite wall (Monotonic shear)

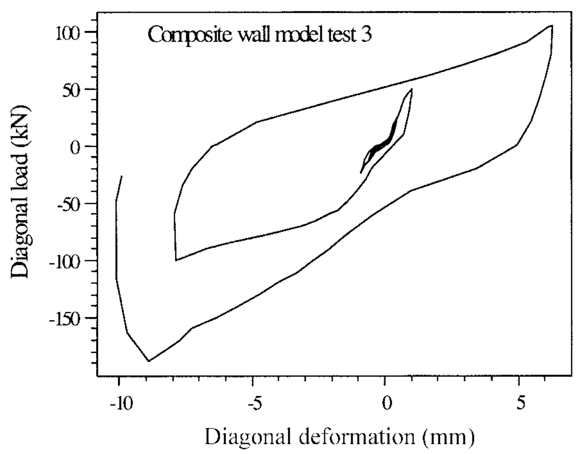


Fig. 11(b) Typical load-deformation responses for composite wall (Cyclic shear)

The load and initial stiffness values from composite wall model tests are summarised in Table 3. Typical load-deformation responses of composite walls under monotonic and cyclic loading are compared in Figs. 11(a)-(b). After the initiation cracking in concrete, the stiffness of the panels is greatly reduced (Figs. 11a-b). The experimental failure loads are reduced by only 11% due to cyclic loading compared to monotonic loading. The experimental pre-cracking stiffness is not affected by the cyclic application of load (Table 3).

Analytical shear stiffness and strength are also compared with those from experiments in Table 3. Eq. (4) can predict the shear stiffness safely for both monotonic and cyclic loading conditions as the ratio of test to analytical stiffness ranges between 1.19 and 1.20.

Eq. (7) underestimates the strength for both cyclic and monotonic loading conditions as the ratios of test to analytical load ranges between 1.33 and 1.49. This is obvious, as Eq. (7) does not reflect the strength enhancement due to sheet-concrete interaction. The improved performance of sheeting in contact with concrete medium and enhancement of concrete strength due to confined environment should be taken into consideration to develop an optimised design Eq. (9). A sheet-concrete interaction factor, ψ, is introduced in Eq. (7) to derive the optimised design Eq. (9):

$$V_w = \psi\left[72\beta\frac{D_x^{1/4}D_y^{3/4}a}{b^2} + 0.074at_{eq}f_{cu}\right] = \psi\left[72\beta\frac{D_x^{1/4}D_y^{3/4}a}{b^2} + \frac{f_c' f_t' at_{eq}}{f_c' + f_t'}\right] \quad (12)$$

Based on model tests, the value of ψ is suggested to be around 1.25 so that the design Eq. (9) can be used safely for both monotonic and cyclic loading conditions.

Table 3 Comparison of analytical and experimental results

Test No	Concrete strength MPa	Pre-cracking shear stiffness (kN/mm)				Failure shear load (kN)			
	$f'c$ (f_{cu}) $f't$	Test		Anal. Eq. (4)	Mean ratio Test/Anal.	Test		Anal. Eq. (7)	Mean ratio Test/Anal.
		Monotonic loading			* mean values				
1	21 (24) 2.35	248		222		122		92	
2	20 (24) 2.43	280	267*	222	1.20	140	137*	92	1.49
4	20 (23) 2.23	273		222		148		92	
		Cyclic loading			* mean values				
3	25 (27) 2.64	262		222		124		92	
5	22 (25) 2.34	268	265*	222	1.19	122	122*	92	1.33
6	21 (24) 2.29	264		222		120		92	

4. Conclusions

This paper describes the behaviour of composite walls under monotonic and cyclic shear loading conditions. The strain conditions and crack patterns confirmed the development of diagonal tension in the panels. The strain condition in composite wall is affected by the initial debonding of steel-concrete interface, concrete cracking with subsequent propagation of cracks and buckling of sheeting with subsequent development and extension of tension field. The walls can provide high shear resistance under both monotonic and cyclic loading conditions if adequate boundary connections between sheeting and concrete are ensured. Simple analytical models for the shear strength and stiffness of profiled concrete, profiled steel sheet and composite wall panels are derived with some modification factors to take into account the effect of cyclic loading. The performance of the design equations is validated through model tests. The proposed equations for shear strength and stiffness are found safe when compared and can be used for design purposes. The research findings are used to develop design guidelines for the use of composite walls as shear elements in buildings.

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