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Finite element modelling of the shear behaviour of profiled composite walls incorporating steel-concrete interaction

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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. The behaviour of such walling under in-plane shear is important in order to utilise this system as shear elements in a steel framed building. Steel sheet-concrete interface governs composite action, overall behaviour and failure modes of such walls. This paper describes the finite element (FE) modelling of the shear behaviour of walls with particular emphasis on the simulation of steel-concrete interface. The modelling of complex non-linear steel-concrete interaction in composite walls is conducted by using different FE models. Four FE models are developed and characterized by their approaches to simulate steel-concrete interface behaviour allowing either full or partial composite action. Non-linear interface or joint elements are introduced between steel and concrete to simulate partial composite action that allows steel-concrete in-plane slip or out of plane separation. The properties of such interface/joint elements are optimised through extensive parametric FE analysis using experimental results to achieve reliable and accurate simulation of actual steel-concrete interaction in a wall. The performance of developed FE models is validated through small-scale model tests. FE models are found to simulate strength, stiffness and strain characteristics reasonably well. The performance of a model with joint elements connecting steel and concrete layers is found better than full composite (without interface or joint elements) and other models with interface elements. The proposed FE model can be used to simulate the shear behaviour of composite walls in practical situation.

Key words: profiled composite wall; shear strength; finite element modeling; steel-concrete interface.

1. Introduction

The novel form of composite walling system comprises vertically aligned profiled steel sheeting and an infill of concrete as shown in Fig. 1 (Wright *et al.* 1994, Wright and Gallocher 1995, Wright

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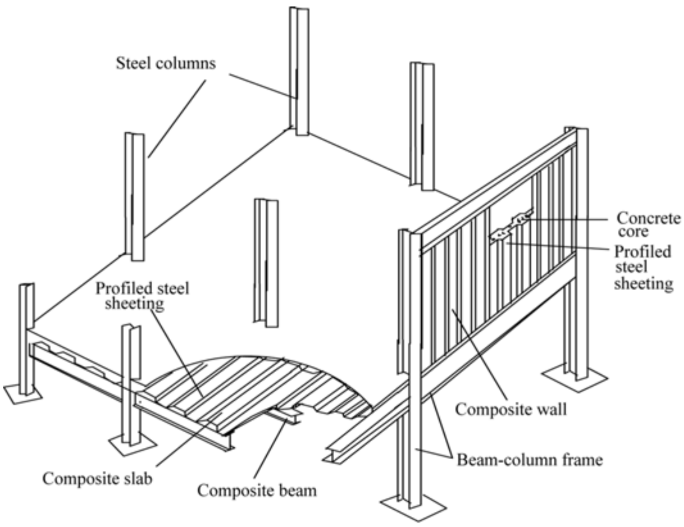

Fig. 1 Schematic diagram of a composite wall in a building

and Evans 1995). Composite walling has many advantages when used in conjunction with composite flooring and is thought to be especially applicable as shear or core walls in a steel framed building. It also 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 (Wright and Gallocher 1995), in addition, also acts as a bracing system to the building frame against wind and destabilising forces in the construction stage (Hossain and Wright 1995). In the service stage, it also acts as reinforcement.

The idea of using composite steel and concrete walls to resist offshore loads was introduced by the Hitachi Shipbuilding and Engineering Company (Adams 1987). Link and Elwi (1995) studied the ultimate and post-peak capacity of composite-steel plate walls subjected to transverse and longitudinal loading. The project investigated simple sandwich walls consist of double skin steel plates with concrete infill. The composite action was provided by the internal steel diaphragm plates connecting the two outer skin plates. The main application of these composite walls lies in the design of offshore structures subjected to large forces from wave action or moving ice.

Yarushalmi (1988) in the United States proposed a form of composite walling known as the ASP Construction System. The development of the system was primarily for use in protective structures from blast resistance and weapons. The proposed wall element consists of exterior steel panels and diagonal interior steel lacing panels with a concrete fill. The walls vary in thickness from 8 to 16 inches and can be filled with concrete, crushed stone or sand. The performance of ASP system was assessed against fragments generated by near miss air bombs. High resistance to penetration was achieved as spalling of the inside surfaces was prevented by the inner steel surface.

Researches were conducted on the axial, bending and shear behaviour of pierced and non-pierced double skin composite walls. The behaviour of such walls was associated with the difficulty in the transfer of load between the steel skins and the concrete core, the buckling of the steel sheeting and the reduced capacity of the concrete core due to profiling (Bradford *et al.* 1998, Wright 1998a,b, Hossain 2000a, Hossain and Wright 2004).

The sheet-concrete interface of the walls under in-plane shear behaves differently than that under

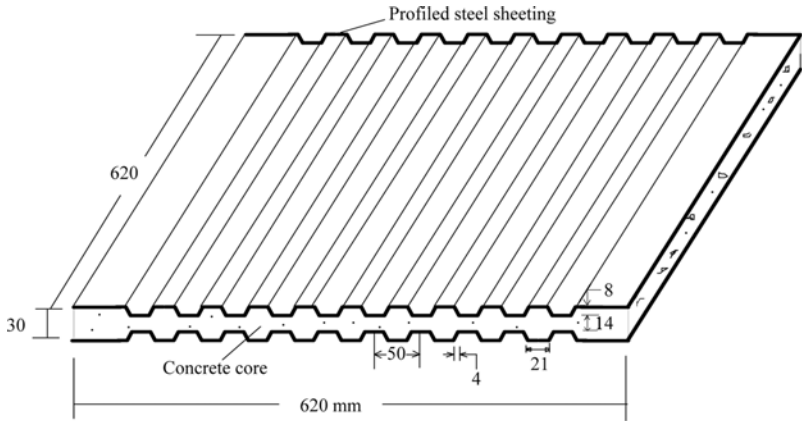

Fig. 2 Details of a model composite wall showing profiled steel sheeting and concrete core

axial or bending loading because the profiled ribs play an important role in providing mechanical bond as the steel tends to slide over the concrete as the chemical bond fails. In this case, the transverse shear bond perpendicular to the profiles (derived mainly from friction) plays an important role rather than longitudinal shear bond parallel to the profile (as in the case of axial behaviour of walls). If it is possible to mobilise this transverse shear bond fully, the composite wall will provide high shear resistance (Hossain and Wright 1998).

The finite element modelling of such walling is a novel approach and should address the complex sheet-concrete interaction at the interface. This paper presents the development of FE models for the simulation of in-plane shear behaviour of composite walls and their performance validation through small-scale model tests. The development of a reliable FE model is important to carry out extensive numerical studies on the application of such walls in a practical building minimizing the need of complex, time-consuming and expensive tests.

2. Experimental behaviour of composite walls

Sixteen small-scale model tests of approximately 1/6th scale using micro-concrete were carried out on profiled steel sheet, profiled concrete core and composite wall to provide information on shear strength, shear stiffness, strain conditions and failure modes under shear loading conditions. The detail of a typical model composite wall specimen showing the profiled steel sheeting and the concrete core is shown in Fig. 2. The model panels had an overall dimension of 620 mm × 620 mm that provided an effective dimension of 560 mm × 560 mm. Model profiled steel sheets were manufactured in-house from plain sheeting of 0.45 mm thickness conforming to the model scale by using an especially fabricated fly press (Fig. 3a).

2.1 Material properties

Micro-concrete, widely accepted as a material to model reinforced concrete structures, can simulate most of the properties of the actual concrete if stress-strain curves up to failure, the ratio of compressive to tensile strength (C_t), Poisson's ratio (υ_c) and modulus of elasticity (E_c) can be made

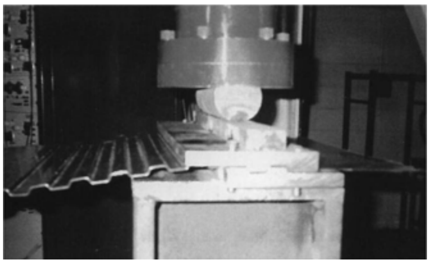

Fig. 3(a) Fly press

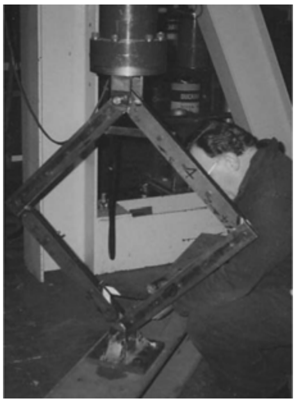

Fig. 3(b) Shear rig showing frame

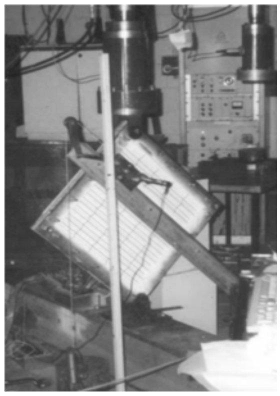

Fig. 3(c) Shear rig with wall

homologous to the actual normal concrete (NC) (Waldron *et al.* 1980, Waldron and Perry 1980, Sommervile *et al.* 1965, Hossain and Wright 1998). A slightly modified version of a well known gap-graded micro-concrete widely used for ¼ th to 1/16 th scale models (Waldron *et al.* 1980, Waldron and Perry 1980, Sommervile *et al.* 1965) was used to manufacture the infill concrete core (Wright and Hossain 1998). The micro-concrete had an aggregate-cement ratio of 3.0 and water-to-cement ratio of 0.6 by weight. Portland cement and locally available sand (B.S. Sieve 7-14 = 50% and B.S. Sieve 52-100 = 50%) were used. The following points were considered in the selection: (i) gap-graded mix had the advantage of increasing workability without using excess water or an admixture, (ii) mix approximated to a 1/8 th scale concrete based on 19-mm maximum size aggregate, (iii) maximum size of the aggregate in the mix was 2.41 mm which satisfied the recommended maximum aggregate size as per White *et al.* (1966) for minimum model dimension (14-mm in this study) and (iv) absence of aggregates finer than B.S. 100 (maximum recommended is only 10%) provided favourable effect of reducing the amount of air voids and water requirements for mortar. Cylinders (100×200-mm) were cast to determine compressive strength (f_c') and splitting tensile strength (f_t'). Cylinders (100×200-mm) were also tested according to B.S.1881, 1970 to determine stress-strain curves, E_c as well as peak (ε_p) and ultimate (ε_{cu}) compressive strains. Properties of micro-concrete are presented in Table 1.

Table 1 Properties of micro-concrete and steel

Micro-concrete properties							
Age	f_c'	f_t'	Wet density	C_t	E_c	Peak strain	Ultimate compressive strain
days	N/mm^2	N/mm^2	kg/m^3	f_c'/f_t'	kN/mm^2	at f_c' (ε_p)	ε_{cu}
Range	24-28	2.4-2.7	2260-2360	9.0-11.0	17-18	0.0019-0.0022	0.0024-0.0027
Mean	26	2.5	2317	10.0	17.5	0.0021	0.0026
St. Dev	2.2	0.15	20	1.0	0.5	0.0005	0.0004
Steel properties							
Yield strength, f_y: 375 MPa; Modulus of elasticity, E_s: 200 kN/mm^2; Ultimate strength, f_u: 400 MPa; Poisson's ratio, ν_s = 0.25; Yield strain, ε_y: 0.0024							

The mean wet density of micro-concrete was 2317 kg/m^3 which was close to the 2400 kg/m^3 of NC according to CEB-FIP Model code (1990). It is known that C_t decreases with the reduction of aggregate size as is the case for micro-concrete. For micro-concrete, C_t is low compared to NC and as a result, the cracking loads are high and crack widths too broad. It was therefore, necessary to develop a micro-concrete of less tensile strength (to achieve C_t similar to NC) without affecting the E_c-value or the compressive strength. The E_c of micro-concrete was well within the range of NC. The mean C_t (= f_c'/f_t') for micro-concrete was 10.0 which satisfied the normal requirement, as C_t of NC normally lies above 9.0.

Steel properties determined from coupon tests are also presented in Table 1.

2.2 Testing and instrumentation

A shear rig (Fig. 3b) had been designed and fabricated to impart pure shear loading condition in the model panels (Hossain and Wright 1998, Wright and Hossain 1998). Preliminary tests (where load was applied through concrete only) showed that the strength of walls was governed by concrete capacity (with negligible steel contribution). As a result, model tests on composite walls were conducted by applying load through both steel and concrete.

All model panels were connected to the test frame of the shear rig through intermediate bolts that also provided the mechanical connection between pair of steel sheeting and concrete core in wall specimens (Fig. 3c). Strain gauges (rosettes and single gauges) were installed at key locations on the surface of the panels and typical strain gauge arrangements are shown in Fig. 4(a). 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 as can be seen in Fig. 4(b). Tests were performed by applying tensile or compressive forces along a diagonal of the wall panel (Figs. 3b-c). The loads were applied in increments and at each load increment diagonal load (P), diagonal deformation (Δ) and strains were monitored through a data logger. Out-of plane displacements were also recorded by movable linear voltage displacement transducer (LVDT) along the profiles as shown in Fig. 3(c). The shear load-deformation (V-δ) response was then obtained

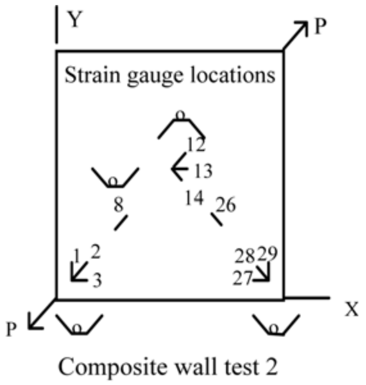

Fig. 4(a) Typical strain gauge locations

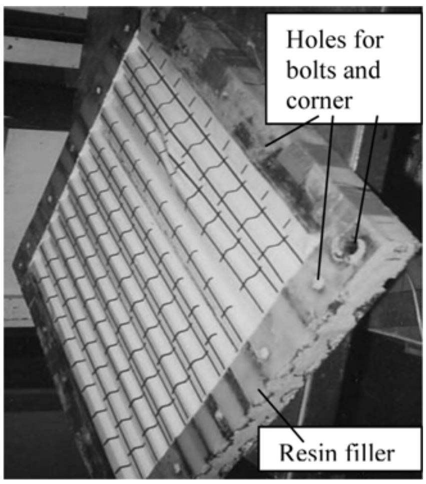

Fig. 4(b) Resin filler to fill profiles

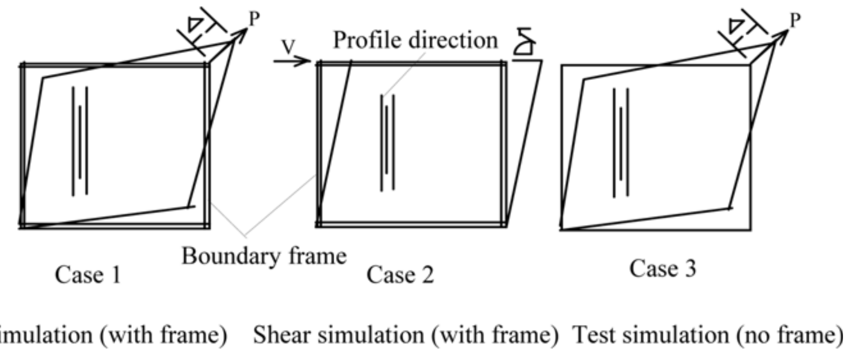

Fig. 5 Different cases of simulation

from the corresponding diagonal load-deformation (P-Δ) response as indicated in Fig. 5 (Hossain and Wright 1998, Wright and Hossain 1998).

2.3 Analysis of experimental results

The behaviour of walls was found to be dependent on the interaction between steel sheeting and concrete core. The failure of the composite wall, after initial stages debonding due to failure of the chemical bond, was started with visual sign of buckling of sheeting locally and progressive outward buckling of sheeting from the concrete (Fig. 6a). Post-buckling behaviour was characterised by the formation of localised tension field. Local tension field extended with the increase of load and the extended tension fields while crossing the folds forced the sheeting to loose its geometric shape in the post-buckling stage. Each trough or crest profile acted as stiffening plates accommodating a tension field entirely in its own territory. The resulting action significantly enhanced the overall

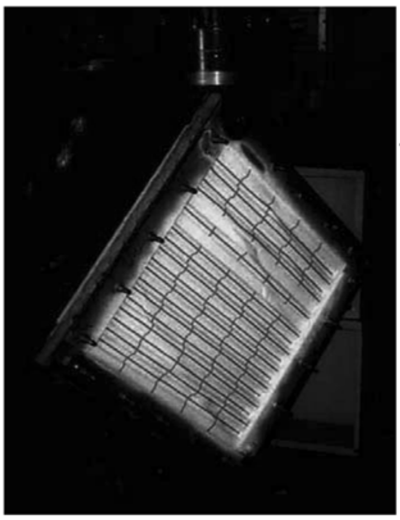

Fig. 6(a) Failure of a composite wall

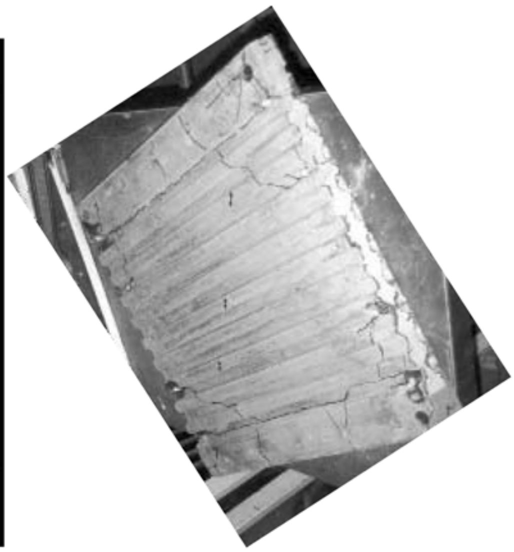

Fig. 6(b) Failure of concrete core

buckling capacity of profiled steel panels. The sheeting buckled outward of the concrete and in the final stages, it slid over the profiled concrete core and eventually lost its profiled geometry. Concrete core failed due to the formation of cracks perpendicular to the loaded diagonal (Fig. 6b) representing the development of diagonal tension that to a great extent exhibited the pure shear condition within the panel.

Typical development of principal strains at the centre and strain along the loaded diagonal in a typical composite wall is shown in Figs. 7(a-b). The development of tensile strain (+ ve values indicate tension and – ve values indicate compression) along the loaded diagonal confirms the development of diagonal tension in the composite wall. The kinks at various stages of strain diagrams showing an increase in strain represents the development of concrete cracking, steel-concrete interface debonding and buckling in the sheeting.

The load-deformation response of a typical composite wall is shown in Fig. 8 which also indicates the initiation of concrete cracking and buckling of steel sheeting. The composite wall exhibited satisfactory strength and ductility before failure. The enhancement of shear strength and pre-cracking stiffness (slope of pre-cracking part of load-deformation response) of the composite wall depends on the degree of mobilisation of mechanical shear bond (chemical bond at sheet-concrete interface is

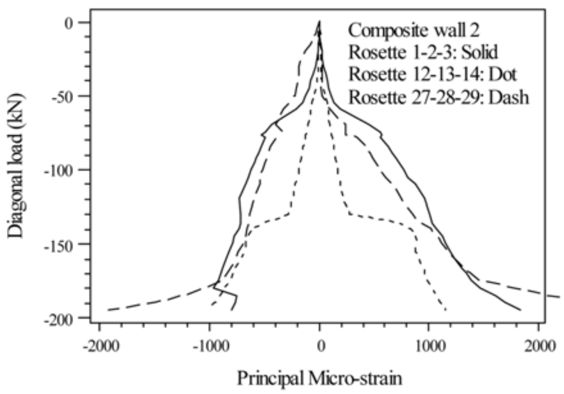

Fig. 7(a) Development of major and minor principal strains in a composite wall

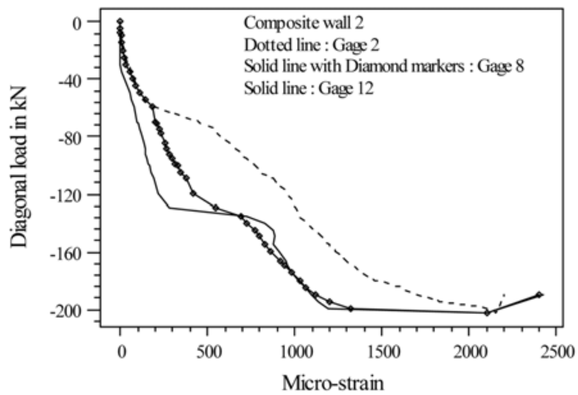

Fig. 7(b) Development of diagonal strain in a composite wall

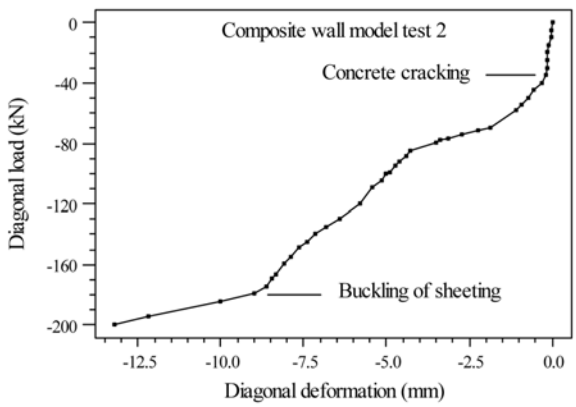

Fig. 8 Typical load-deflection response of a composite wall

assumed to be negligible) between sheeting and concrete. Furthermore, adequate steel sheet-concrete boundary connections as well as transfer of loads through both steel and concrete especially at the boundary should be ensured to enhance shear bond mobilisation at steel-concrete interface.

3. Experimentation on steel-concrete interface

Previous researches were concentrated on the shear connection between steel and concrete in composite slabs with profiled decks using push-out, pull-out or push-off tests (Lloyd and Wright 1990, Daniels 1988, Easterling and Porter 1994). In this study, push-off tests were conducted to determine strength and stiffness properties of the shear transfer mechanism at steel-concrete interface in a composite wall. Typical push-off test arrangement is shown in Fig. 9 (Hossain 2000b). Specimens, approximately 600 mm × 600 mm, were constructed to model segments of symmetric half-thickness of composite wall in which ribs were oriented transverse (to study transverse shear bond) to the applied load. Push-off specimens were made of full-scale profiled steel sheet and concrete (made of 16-mm maximum size concrete having a 28-day cylinder compressive strength of 25 MPa). The load was applied to the concrete and was transferred to the steel-concrete interface and reacted by the connection (between steel sheet and push beam) to the push beam. The load path was such that the shear transfer mechanism at steel sheet-concrete interface (due to chemical bonding, mechanical resistance of profiled ribs and friction) must transmit the applied horizontal load.

Up to the stage of chemical debonding, horizontal slip or vertical separation was not observed. After debonding, concrete slided over the profiled sheet that led to horizontal slip and vertical separation between sheet and concrete. The horizontal slip and vertical separation were monitored by LVDT installed at various strategic locations of the specimen throughout the loading history (Fig. 9a). Typical load-slip and load-vertical separation curves are presented in Fig. 9(b). These

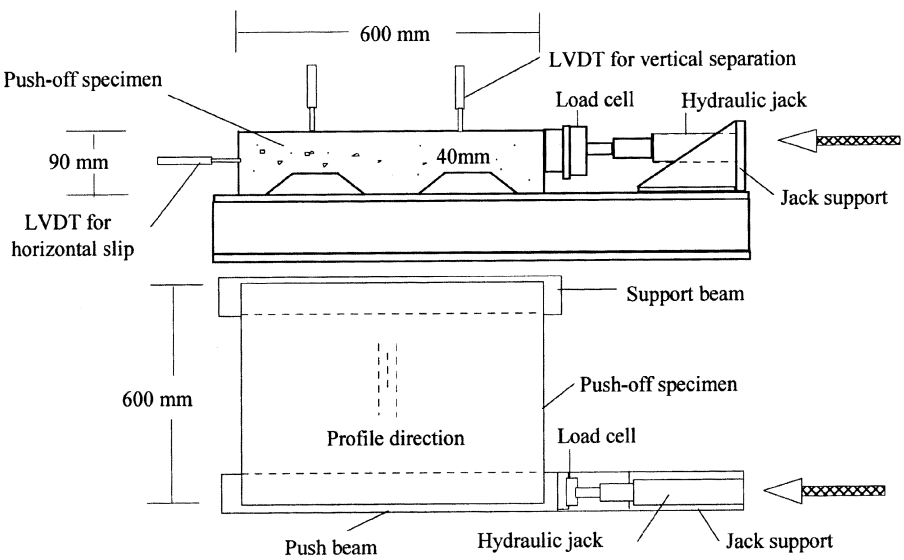

Fig. 9(a) Push-off test arrangements

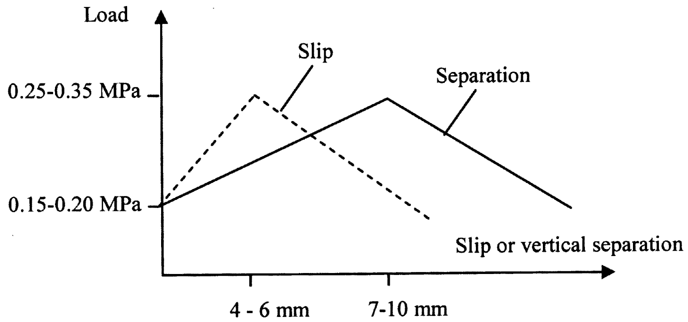

Fig. 9(b) General load-slip or load-separation relationship at steel-concrete interface

curves were used as a guideline to simulate general steel-concrete interface behaviour in finite element models.

4. Development of finite element models

The finite element (FE) analysis of composite walls was carried out using package program LUSAS (FEA Limited, Surrey, UK, www.lusas.com). The FE analysis was complex due to the non-linear steel-concrete interface behaviour in a composite wall. Such complex interface problem was addressed by developing various FE models.

4.1 Simulation strategy

FE models were primarily classified into two groups based on different ways of modelling non-linear steel-concrete interface: (a) full composite model- assuming full steel-concrete interaction (perfect bond) and (b) model with non-linear partial steel-concrete interaction- allowing sliding and vertical separation between steel and concrete by incorporating various joint or interface elements available in LUSAS. The performance of FE models was validated through test results. Three loading conditions were simulated in FE modelling as shown in Fig. 5:

Case 1: Model test simulation with boundary frame: non-linear modelling of actual experimental conditions to simulate load-deformation (P-Δ) response as well as strains, strength and stiffness characteristics.

Case 2: Transformed shear simulation with boundary frame (V-δ response): non-linear modelling to simulate behavioural characteristics similar to Case 1.

Case 3: Test simulation without boundary frame (P-Δ response): non-linear modelling similar to Case 1.

4.2 Modelling of concrete and steel

A biaxial non-linear concrete model that includes strain-softening (tension stiffening) and shear retention effects was used. Failure envelope for biaxial concrete model formulated in terms of principal stresses (σ_1 and σ_2) is presented in Fig. 10(a). A linear decay strain softening model was used in the concrete model (Fig. 10b), whereby the rate of stress release normal to the crack was

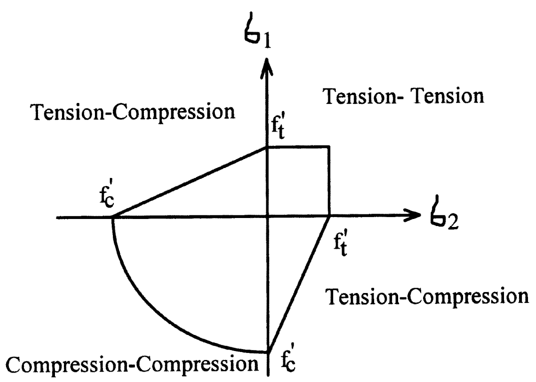

Fig. 10(a) Failure envelope for biaxial concrete model

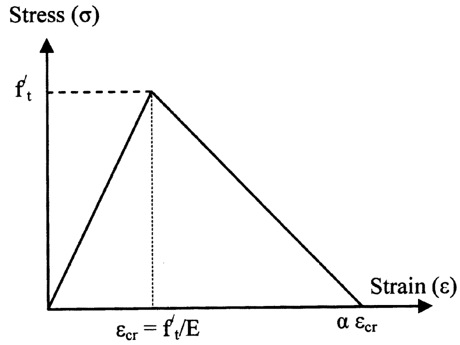

Fig. 10(b) Linear strain softening model

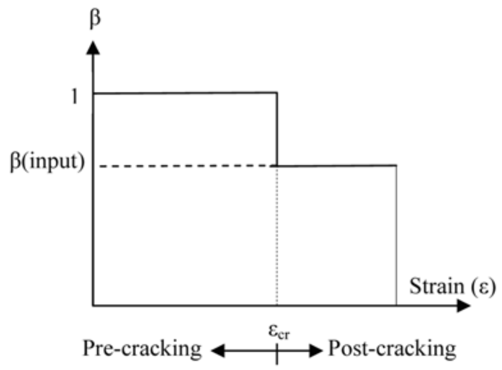

Fig. 10(c) Uniform shear retention model

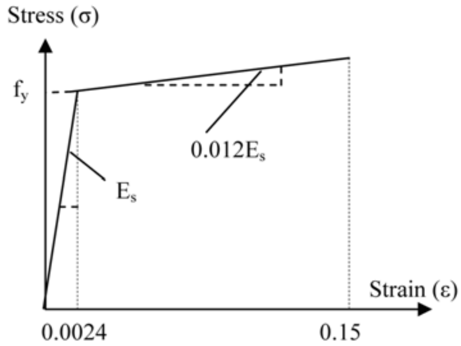

Fig. 10(d) Bi-linear elasto-plastic model for steel

controlled by the specification of a softening parameter (α). The softening parameter relates the initial cracking strain (ε_{cr}) to the ultimate tensile strain. Cracks, which are subject to tensile strains in excess of the ultimate tensile strain, are assumed to be fully open and transmit no normal stress. Typical values of softening parameter range from 5 to 50 depending on the failure modes. Shear dominated behaviour exhibits a brittle response for which lower values is appropriate while higher values are more effective for the ductile nature of flexurally dominated response. The transfer of shear stress between the crack surfaces is dependent upon the aggregate interlock of the concrete and was modelled numerically via the in-plane shear modulus (G). A constant fraction shear retention model was used (Fig. 10c) where the in-plane shear stress for cracked concrete was reduced by modifying the shear modulus by a factor called “shear retention factor (β)”. For undamaged concrete ($\varepsilon < \varepsilon_{cr}$), β is equal to 1. As a general rule, the lower values (β < 0.5) are appropriate for shear dominated failure. A shear retention factor (β) of 0.3, softening parameter (α) of 30, maximum compressive strain (ε_{cu}) of 0.0026, and a Poisson’s ratio (υ_c) of 0.18 were used in the current study. These numerical parameters were determined from extensive parametric finite element studies (Hossain and Wright 1994, Hossain 1995) and were found to suitably model the behaviour of the used micro-concrete. Other parameters such as tensile (f_t') and compressive (f_c') strengths were derived from the actual control specimens of micro-concrete (Table 1). A bi-linear elasto-plastic stress-strain model was used for steel (Fig. 10d).

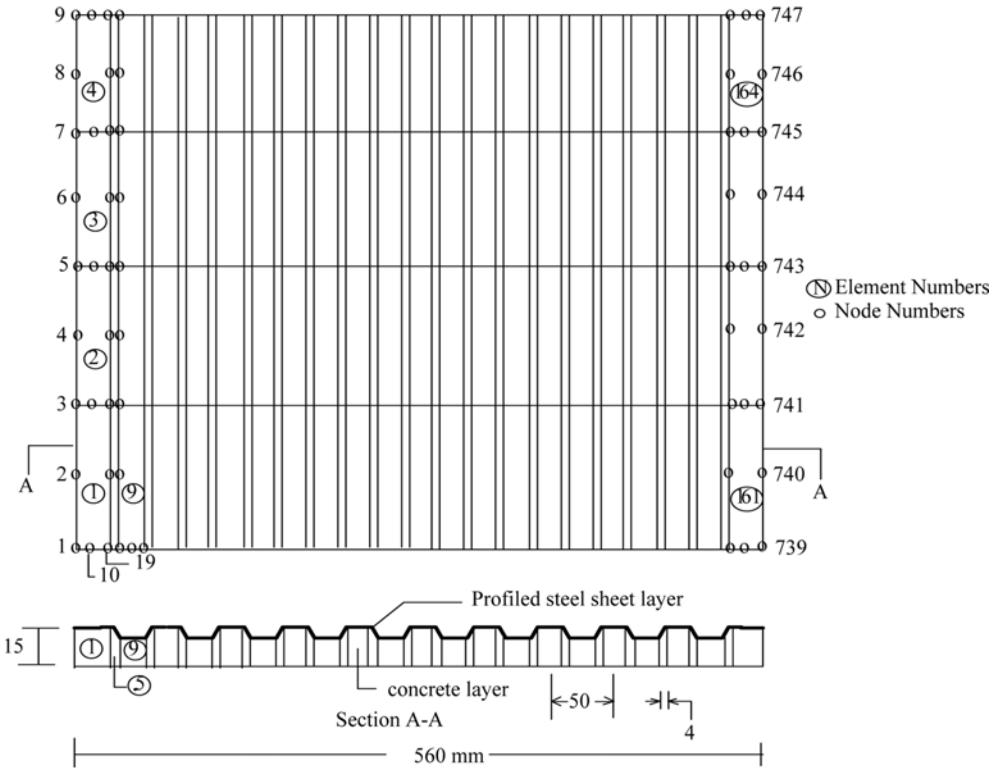

Fig. 11 Descretization of a composite wall (Model 1)

4.3 Model 1: Full composite without interface

The 3-D, 8-noded QSL8 semi-loof shell elements (having three degrees of freedom at corner nodes) with the provision of different layers and material properties in each layer were used in the FE idealisation (Fig. 11) of the composite wall. The QSL8 element configuration allowed only symmetric half thickness of the wall to be modelled and steel and concrete were represented as different layers of the element. QSL8 semi-loof shell elements allowed materially non-linear analysis utilizing elastoplastic constitutive laws (for steel, in this study) and the non-linear concrete model described earlier. Elasto-plastic model for steel was based on von-Mises yield criterion.

Non-linear steel-concrete interaction as observed in actual tests could not be simulated with this model as full composite action was assumed between adjacent steel and concrete layers. The simulation of the boundary frame was not possible with the compatible semi-loof beam (3D- three noded semi-loof beam elements ‘BSL3’ with six degrees of freedom at corner nodes and 5 degrees of freedom at mid nodes) and joint elements (JL43 joints with three degrees of freedom or JL46 with six degrees of freedom at corner nodes and JSL4 joints with five degrees of freedom at mid nodes), as they had no provisions for composite properties. Due to this miss-matching of material properties, the modelling of the wall was performed by simulating P-Δ response without boundary frame as in Case 3.

The load was applied as prescribed displacement and non-linear FE analysis provided diagonal load, P, and corresponding diagonal deformation, Δ, at each load increment. The FE analysis

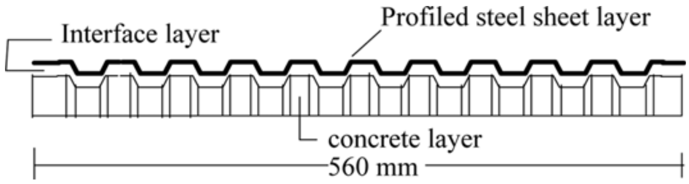

Fig. 12 Use of interface layer in Model 2

provided data on stress-strain characteristics including cracking and yielding of concrete and steel layers, respectively. The non-linear run was carried out up to the yielding of steel to generate complete load-deformation response.

4.4 Modelling of composite wall with joint and interface elements

Composite walls were modelled with various interface and joint elements that could simulate actual non-linear and non-composite steel sheet-concrete interaction as observed in the model tests. The effect of interface properties on the shear stiffness, shear strength and strain characteristics was studied.

4.4.1 Model 2: Layered analysis with interface

An additional interface layer was provided in between steel and concrete as shown in Fig. 12 whose material properties could be changed (like modulus of elasticity, Poisson's ratio, uni-axial yield strength etc.) to study the effects on the stiffness, strength and strain condition in steel and concrete layers. This model ensured non-linear analysis by using elasto-plastic model based on von-Mises yield criterion for both steel and interface layers while using the non-linear concrete model as described earlier for concrete layer. The FE mesh using QSL8 semi-loof shell elements for steel, interface and concrete layers simulating symmetric half thickness of wall was similar to Model 1 as shown in Fig. 11. The modelling of the wall was performed by simulating P-Δ response without boundary frame as illustrated in Case 3.

Extensive parametric studies revealed that the modulus of elasticity and Poisson's ratio of interface layer had little influence on the composite action. The rigidity or flexibility of the steel-concrete interface was found to be sensitive on the variation of yield strength of the interface layer. A low value could simulate flexible interface while a high value could simulate rigid interface. Parametric studies were conducted to model composite walls with interface layers having variable flexibility to simulate a pure shear condition. The stiffness of composite wall reduced with flexible interface layers. The load-deformation response also showed an increase in shear deformation in the case of flexible interface. However, cracking and yielding loads were not affected.

Model test results were used to find an optimum value of yield strength so that the yielding of interface layer could occur at a specified loading stage simulating a non-linear steel-concrete composite action during the later part of the loading history. This model had a limitation. As the layers of steel, interface and concrete were represented by the same elements with the same nodes, it was not possible to simulate physical separation and sliding between steel and concrete as observed in the actual tests.

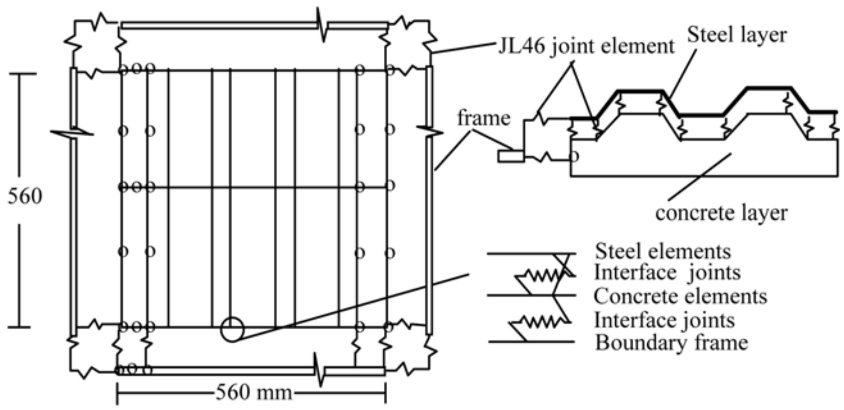

Fig. 13 Finite element descretization of wall in Model 3

4.4.2 Model 3: Simulation with joint interface

Model 3 used separate 3-D, QSL8 semi-loof shell elements to represent steel and concrete layers with different elemental node numbering but the nodes had the same co-ordinates. This model also simulated symmetric half-thickness of walls as in Model 1 and Model 2. The steel and concrete layers were connected to each other by 3-D joint elements (JL46) with 4 nodes where 3rd and 4th nodes were used to define local x-axis and y-axis, respectively. End nodes (active nodes: 1 and 2) had six degrees of freedom (3-transational and 3-rotational). The boundary frames were simulated with 3-noded semi-loof beam elements (BSL3) with six (three rotational and three translational) and five (three translational and two rotational) degrees of freedom at end and mid nodes, respectively. Beam elements were connected to both steel and concrete layers by JL46 joint elements. JL46 joints were associated with a material model that incorporated non-linear elasto-plasticity with isotropic hardening. The detail FE idealisation of the model is presented in Fig. 13.

This model had the limitation of geometric arrangements of sheeting and concrete core layers. Diagonal (P) or shear (V) load was applied (both Case 1 and Case 2 simulations were conducted) through the frame and corresponding diagonal (Δ) or shear (δ) deformation was obtained. Elasto-plastic model based on von-Mises yield criterion was used for steel and the bi-axial non-linear concrete model described earlier was used for concrete.

The steel-concrete interface was the most uncertain portion of the model due to the lack of accurate interfacial response data. JL46 joint elements connecting steel sheet and concrete layers simulated vertical steel-concrete separation as well as transverse sliding of the steel sheeting over profiled ribs of the concrete layer. The general load versus displacements (vertical separation or horizontal slip) relationship was derived from push-off tests and incorporated in the numerical JL46 joint interface elements. The data from transverse push-off tests were used as a basis for a parametric study to identify the joint properties (elastic spring stiffness and strain hardening stiffness) that can reasonably simulate the tested walls. Although push-off tests provided load-horizontal slip/vertical separation relation based on symmetric half thickness of the wall, it provided useful information. However, optimum values of interface joint properties were determined from the parametric simulation of actual wall but influenced by the general behaviour of push-off tests. Extreme cases of full connection (providing very rigid joints) and no-connection at the steel sheet-concrete interface were also studied. The connections between the frame and the wall were made

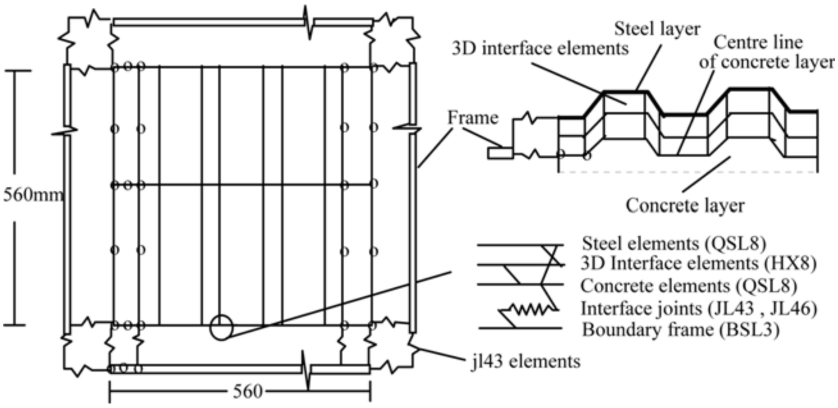

Fig. 14(a) Finite element representation of Model 4

very rigid. The pre-cracking stiffness of walls was found to be not affected by the steel sheet-concrete connections provided that they both had full connection to the boundary frame.

4.4.3 Model 4: Simulation with 3-D interface

In this non-linear model, steel and concrete layers were represented by 3-D semi-loof shell elements (QSL8) as before but elemental nodes had different co-ordinates. The steel and concrete layers were, therefore, completely separate as the nodes were specified at the centre lines of each of the concrete and steel layer as shown in Fig. 14(a). In between the two layers (between the centre lines of the layers), 3-D 8 noded interface solid continuum elements (HX8) having 3 translational degrees of freedom (u, v, w) at each node were used as interface elements (Figs. 14a-b). The HX8 interface elements connected steel-concrete layers (QSL8 elements) at corner nodes as these nodes had three translational degrees of freedom (u, v, w) like HX8 elements. Therefore, compatibility of two different types of elements at nodal points was satisfied.

This model simulated test conditions (Case 1) to achieve P-Δ response. The boundary frame was modelled with semi-loof beam elements (BSL3). Connections between frame-steel sheet-concrete were provided by 3-D, four noded (two active end nodes and two nodes to define local axes) JSL4 joint elements (three translational and two rotational degrees of freedom at two active nodes) for

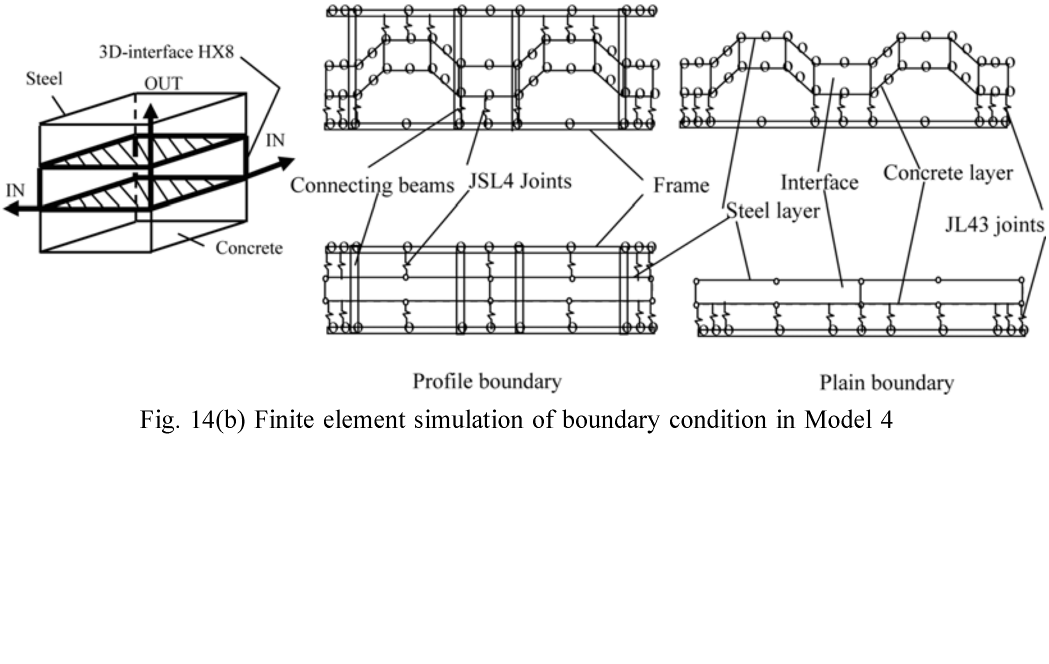

Fig. 14(b) Finite element simulation of boundary condition in Model 4 middle nodes and 3-D, four noded (two active end nodes and two nodes to define local axes) JL43 joint elements (three translational degrees of freedom at two active nodes) for corner nodes to satisfy the compatibility of nodal degrees of freedom. As joint elements should be used to connect nodes having same co-ordinates, it was necessary to use two frame beams connected together by connecting beam for joining both steel and concrete layers to the frame when load was applied through both steel and concrete. The detail connections in profiled and plain boundaries are shown in Fig. 14(b).

JSL4 and JL43 joints had the same material model as JSL46 that incorporated non-linear elasto-plasticity with isotropic hardening. Elasto-plastic model based on von-Mises yield criterion was used for steel sheeting and the bi-axial non-linear concrete model described earlier was used for concrete. HX8 elements were incorporated with elasto-plastic material properties to model the friction-contact relationship between steel and concrete layers. The elastic material properties were defined in the local basis, permitting different values to be specified in the normal (out of plane) and tangential (in-plane) to the plane of the interface (Fig. 14b). The non-linear behaviour was governed by an elasto-plastic constitutive law that incorporated a limited tension criterion normal to the interface plane and a Mohr-Coulomb criterion tangential to the interface plane.

The non-linear 3-D, HX8 interface elements had the following main properties: in and out-off plane modulus of elasticity (E_{in}, E_{out}), in and out-off plane shear modulus (G_{in}, G_{out}), Poisson's ratio, cohesion, friction angle and uni-axial yield stress. Extensive parametric studies had been carried out to study the influence of all these properties that led to the identification of critical parameters controlling the interface behaviour. The distortion of steel layer was found to be influenced by G_{out} while the stiffness seemed to be not affected by the in-plane (G_{in}) and out-of plane (G_{out}) shear modulus. The optimum values of these parameters to achieve reliable simulation were determined from parametric simulation of experimental walls.

5. Performance of FE models

The load-deformation responses from different FE models are superimposed on a typical test response in Fig. 15(a). The test load-deformation response shows a large displacement following the cracking in concrete compared with FE responses. Model 3 with joint elements simulates better flexible response than the other models. Model 2 is better than Model 1 due to the use of interface elements but both of them fail to simulate load-deformation response because of not simulating physical steel sheet-concrete separation. Model 3 is better than Model 4 as joint elements can simulate vertical and lateral separation at steel-concrete interface better than solid interface elements. Model 3 is better although full matching of load-deformation response is not possible. Better correlation of the results of push-off tests with joint properties will definitely lead to much better simulation.

Pre-cracking stiffness of walls (Table 2) from all FE models are in good agreement with the test results as the ratio of experimental to FE values ranges between 1.01 and 1.05. Model 3 and Model 4 with joint/interface elements (ratio ranges between 0.95 and 0.97) are able to predict the cracking loads better than Model 1 and Model 2 (ratio ranges between 0.8 and 0.9). All FE models fail to simulate buckling of sheeting but the first yield loads are found close (ratio ranges between 1.01 and 1.16) to the experimental failure loads.

The typical variation of principal strain at a central rosette location from a typical model test is

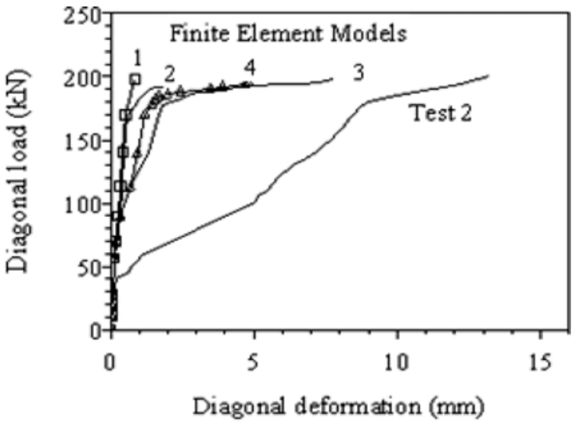

Fig. 15(a) Simulation of load-deformation response

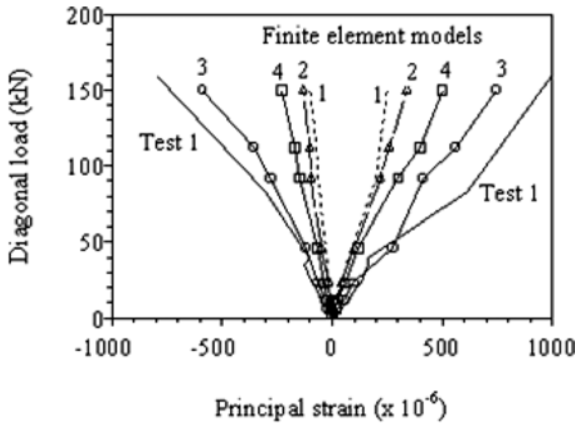

Fig. 15(b) Simulation of strain

Table 2 Comparative study of FE models and experiments

		Finite Element Models				Experimental Model
		1	2	3	4	
Stiffness (kN/mm)	Diagonal	536(1.05)	568(1.01)	532(1.05)	536(1.05)	561
	Shear	268(1.04)	284(1.01)	266(1.05)	268(1.05)	280
Cracking load (kN)	Diagonal	40(0.9)	45(0.8)	38(0.95)	37(0.97)	36
	Shear	28(0.9)	32(0.8)	27(0.95)	27(0.97)	26
Yield load* (kN)	Diagonal	170(1.16)	190(1.04)	194 (1.02)	196(1.01)	198
	Shear	125(1.16)	135(1.04)	144 (1.02)	145(1.01)	146

Values in the brackets represent ratio of experimental to FE models
*Ultimate load for experimental models

compared with those from FE modelling in Fig. 15(b). Performance of FE Model 3 is better compared to other models. The strains from FE models are found to be less than those from model tests. This is reasonable because of the full composite action in Model 1 and difficulty associated with the interface and joint elements to model non-lniear steel-concrete interaction in Models 2, 3 and 4.

5.1 Discussion

FE models are found to be good in predicting the shear stiffness and the ultimate shear strength of composite walls. For strength and stiffness prediction under the test load condition, it is good to use Model 1 with full composite action. Model 1 is relatively simpler and time efficient. Other models, especially Model 3 and Model 4 with joint or interface elements are complex and need much longer computational time.

In general, Model 3 is recommended as it can simulate steel-concrete interface characteristics and provides better geometric compatibility of the composite wall. Model 3 will be good to model the behaviour in practical structural application as variable modes of connection (only clamped condition was applied in model tests) between wall and frame (as frequently encountered in practice) can be incorporated in this model with variation of joint properties to reflect the effect of actual steel-concrete interaction. In practice, this model can also be equipped with actual steel-concrete interface characteristics from push-off test conducted on specimens manufactured from different types of commercial profiled steel sheeting available in the market.

6. Conclusions

This paper describes the finite element (FE) simulation of the shear behaviour of double skin profiled composite walls. Four FE models are developed and their performance is validated through experimental models of composite walls. FE modelling of composite walling should reflect the complex steel sheet-concrete interaction and wall-frame boundary connections. FE model with joint elements simulating proper steel-concrete interaction performs better than full composite (without interface or joints) model or other models with interface elements. The FE models presented in this study can predict strength and stiffness of a composite wall. Proposed model with joint elements connecting steel and concrete can be used to simulate the behaviour of composite walls in practical structures with various modes of steel-concrete and wall-frame connections.

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