J. Materials, Conc. Struct. Pavements, JSCE, No. 767／V-64, 285-300, 2004 August

VOLCANIC PUMICE BASED THIN WALLED COMPOSITE FILLED BEAMS WITH INTERFACE CONNECTIONS

Khandaker M. Anwar HOSSAIN

Dr. Eng., Professor, Department of Civil Engineering, Ryerson University
(350 Victoria Street, Toronto, Ontario, M5B 2K3, Canada)
E-mail: ahossain@ryerson.ca

The behaviour of thin walled composite (TWC) filled beams with normal (NC) and lightweight volcanic pumice concrete (VPC), is described. Comprehensive series of tests provided information on the load-deflection response, failure modes, stress-strain characteristics and effect of different modes of interface connections. The strength of the beam is limited by the compression buckling capacity of the steel plate at the top of the open box section. The enhancement of strength of such beams can be possible by stiffening the compression steel plates at the open end of box section with various modes interface connections. The design of such beams should consider local buckling of steel and interface shear bond characteristics.

Key Words: *volcanic pumice concrete, thin walled composite beam, steel buckling, sheet-concrete interface connection, strength*

1. INTRODUCTION

Thin walled composite (TWC) section is a new idea for beams [1), 2), 3), 4)] comprising cold formed steel elements with an infill of concrete that are suitable as replacement for hot-rolled steel or reinforced concrete in small to medium sized building. Typical TWC beams are shown in **Fig.1.** The inherent advantages of this system are derived from its structural configurations [5), 6)]. Open box sections for beams will allow easy casting of in-fill concrete. TWC sections do not require temporary formwork for infill concrete as the steel acts as formwork in the construction stage and as reinforcement in the service stage. They are simple to fabricate and construct compared to conventional reinforced concrete where skilled workers are needed to cut and bent complex forms of reinforcement. The infill concrete is less likely to be affected by adverse temperature and winds as experienced in the case of reinforced concrete. The in-fill concrete is generally cured quickly and in any case, the load capacity of the steel alone may be relied upon for most construction loads. The steel sections can be designed, primarily, for the construction load of wet concrete, workmen and tools. TWC sections are more susceptible to fire, although the thermal mass of the concrete infill provides reasonable protection to most fire loads. The smooth metallic finish of TWC sections is superior to conventional concrete and accepts more paint finish. Concrete filled steel elements as a means of providing aesthetic and economical structural elements attract interests in the construction industry.

Researches [7), 8), 9), 10), 11)] had been conducted in the past to study the behaviour of different forms of composite beams. Oduyemi and Wright [9)] and Wright et al.[10)] investigated sandwiched composite beams with varied plate thickness, stud spacing and length and concrete strength. Tested beams exhibited three types of failure: flexural (stèel yield prior to concrete crushing), horizontal slip (failure of shear connectors) and vertical shear (due to insufficient shear capacity of concrete and studs). However, local buckling of the steel may precede the three failure modes. It was possible to get the buckling of compression plate prior to yield by limiting the stud spacing/steel thickness ratio to 30.

Oehlers [6)] and Oehlers et al. [5)] investigated the flexural behaviour of profiled composite beams. They suggested that the flexural strength of such beams could be adversely affected by the local buckling of the sheeting between the ribs of the profile. Simple design procedures were developed to prevent local buckling of profiled steel sheeting prior to ultimate

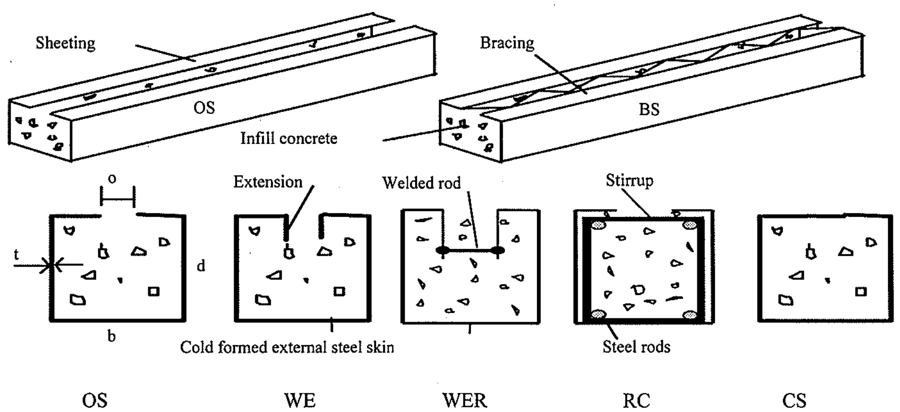


Fig.1 Geometric of composite beams

Table 1 TWC beam Details

Beam	Concrete Steel Type	f'c MPa	f_{sy} MPa	b	d	o	y	t	Configuration
Series A : L=1500mm									
A1	NC	21	375	100	100	20	0	3.2	OS, NMR, NS
A2	NC	21	350	50	100	10	0	2.3	OS, NMR, NS
A3	VPC	20	375	100	100	20	0	3.2	OS, NMR, NS
A4	VPC	20	375	50	100	10	0	2.3	OS, NMR, NS
Series B: L = 600mm									
Filled									
B3Inc	NC	21	375	100	100	20	0	3.2	OS, NMR, NS
B3IInc	NC	21	375	100	100	100	0	3.2	CS, NMR, NS
B3IIvpc	VPC	20	375	100	100	100	0	3.2	CS, NMR, NS
B4IInc	NC	21	350	50	100	50	0	2.3	CS, NMR, NS
B4IIvpc	VPC	20	350	50	100	50	0	2.3	CS, NMR, NS
B3d/4nc	NC	21	375	100	100	20	25	3.2	WE, NMR, NS
B3d/2nc	NC	21	375	100	100	20	50	3.2	WE, NMR, NS
B4Inc	NC	33	350	50	100	10	0	2.3	OS, NMR, NS
B4IIInc	NC	33	350	50	100	10	0	2.3	BS, NMR, NS
Unfilled									
B4III	None	--	350	50	100	10	0	2.3	BS
B4I	None	--	350	50	100	10	0	2.3	OS
Series C: L = 990 mm; fy = 455 MPa									
CB1	NC	21	257	150	250	35	0	1.6	WE, NMR, NS
CB2	NC	21	257	150	250	35	62.5	1.6	WE, NMR, NS
CB3	NC	21	257	150	250	35	62.5	1.6	WER, NMR, NS
CB4	NC	21	257	150	250	35	62.5	1.6	WE, NMR, NS
CB5	NC	33	275	150	250	35	125	1.6	WE, NMR, NS
CB6	NC	33	275	150	250	35	125	1.6	WER, NMR, NS
CB7	NC	33	275	150	250	35	0	1.6	OS, MR, NS
CB8	NC	33	275	150	250	35	0	1.6	OS, MR, S

Remarks

Abbreviations:
b, d, L = Width, depth and span of the beams, respectively; t = thickness of the sheeting; y = length of extension; o = opening width; MR: Main reinforcement, NMR: no main reinforcement; NS: No stirrup; S: with stirrup; W/C: Water to cement ratio.

Other features:
Stirrup spacing: 6 mm @150 mm c/c
Main steel (MR): 4, 6 mm rod - two at the top and two at the bottom;
Welded rod: 6 mm @150mm c/c connecting welded extension
36 mm holes @ 150 mm c/c on the welded extensions for beams in series C only, to allow concrete movement.

Concrete mixture proportions in kg per 1 m^3 of concrete:
NC: Cement = 400; Sand = 870; Crushed stone aggregate = 800; W/C = 0.45
VPC: Cement = 420; Sand = 704; Crushed VP aggregate = 412; W/C = 0.45

Dry density of concrete (kg/m^3):
2500 (NC); 1800 (VPC)

strength. It was confirmed that shear bond failure had only a small effect on the ultimate strength.

Current study [12), 13)] led to the development of novel form of thin walled composite (TWC) beams with recommendation on the use of volcanic pumice concrete (VPC) manufactured from volcanic pumice (VP) aggregate. One aspect of this research was to study the comparative performance of TWC beams with NC and VPC. Comprehensive research had been carried out [14), 15)] on the properties of lightweight VPC. Volcanic aggregates are found in volcanic areas around the world and finding new and improved ways to build with such aggregate is becoming widespread. New sources of volcanic aggregates are being produced steadily. Recently the eruption of volcanoes in Caribbean Island of Monseratt emitted large quantities of such aggregate. Of course these volcanic eruptions are very dangerous catastrophes but they leave very useful materials after the disaster. The meaningful use of such volcanic debris can, not only transform them into natural resources to produce low cost construction materials for sustainable development but also can help to decrease environmental hazard. Investigations suggested that, it is possible to obtain a VPC having strength of 30 MPa and 30% lighter than normal concrete (NC). The improved performance of VPC in a confined environment will allow the use of comparatively low strength VPC in this form of construction. Advantage of such TWC system includes a very light construction weight, excellent surface finish, relatively slender dimensions, enhanced ductility and the potential for semi-rigid connections.

The behaviour of the proposed TWC beams is affected by the sheet-concrete interface and buckling of thin sheets in contact with concrete as observed in previously tested composite beams. On the other hand, composite beams using profiled sheets have the advantages of providing more composite action due to the presence of profiled ribs that can provide mechanical interlocking and improve the buckling capacity of the sheeting. Proposed TWC beams in this study will face problems of interface separation, as it will rely only on chemical bond at the sides of the beam. Additional connection devices are to be used to increase the composite action, otherwise beams may fail due to pre-mature buckling of the sheeting and interface separation. The use of volcanic pumice (VP) in this form of construction will also be an interesting aspect. Comparative performance study of such beams with traditional normal concrete beams will explore the viability of using VP in such construction.

This paper describes the experimental performance of TWC beams with various interface connections and highlights the effectiveness of such connections in enhancing the strength of such beams.

2. EXPERIMENTAL STUDY

Comprehensive series of tests were conducted to study the behaviour of TWC beams. The test specimens were fabricated with varying geometric, material and interface connection parameters. Based on geometric and mode of connections, the beams were classified (detailed shown in **Fig.1**) into open (OS), welded extension (WE), welded extension with rod (WER), braced (BS), closed (CS) and RC filled. The experiments had been conducted in three series: Series A, B and C.

(1) Description of composite beams

a) Series A: Slender open beams (OS)

A total of four beams were tested to study the performance of comparatively slender OS beams (**Fig.1**) with normal (NC) and lightweight pumice concrete (VPC) as in-fill. The beams A1 and A3 were made of 100 mm x 100 mm square hollow section (SHS), while A2 and A4 were made of 50 mm x 100 mm rectangular hollow sections (RHS). The details of the beams are presented in **Table 1**.

b) Series B: TWC beams with various mode of connection

A total of 11 beams classified as OS, CS, BS and WE were tested to study the effect of connections enhancing the sheet-concrete interaction, effect of in-fill VPC and performance compared to unfilled steel skins. The details of such beams are presented **Table 1**. These beams designated as B3 and B4, were made of square and rectangular hollow section respectively.

Three OS beams (designated as B3I and B4I) both filled and unfilled (steel section alone) sections were tested. Two of four CS (designated as B3II and B4II sub series) beams were made of VPC in-fill. Two WE (designated as B3d/4nc and B3d/2nc) beams (**Fig.1**) had steel extension made from the pieces of same hollow section. The welded extensions had full-length weld and the depth of welded extensions (y) used, were quarter (d/4) and half (d/2) of the depth of the beam.

Two BS (designated as B4III sub series) beams both filled and unfilled were fabricated with welded steel braces at the open top as shown in **Fig.1**. The steel pieces used in bracing were made of steel of the same hollow section. The braces were 10 mm wide and formed an angle between 42-45° covering a length 42-45 mm (30 mm wide and 30-33 mm long).

c) Series C beams

A total of 8 beams designated as CB having the dimensions of 150 mm x 250 mm x 1200 mm were made of 1.6 mm thick cold-formed steel plate. Three types of beams including WE, WER and RC as shown in **Fig.1** were fabricated. These beams had an effective span of 990 mm with a span/depth ratio of 3.96. Detail of these beams is presented in **Table 1**.

CB1-2-4-5 (WE) had welded extension plates as shown in **Figs.1** and **2**. The 0.48 mm welded extension plates were tag welded at 150 mm c/c. The welding of such thin plates did not cause any problems from fabrication point of view or unusual deformation due to residual strain but it might have reduced the strength of the plate at weld. However, it is better to take into consideration the effect of residual strain if problem arises, to ensure the strength of the welded part. CB3 and CB6 (WER) had provided with additional restraint to enhance the

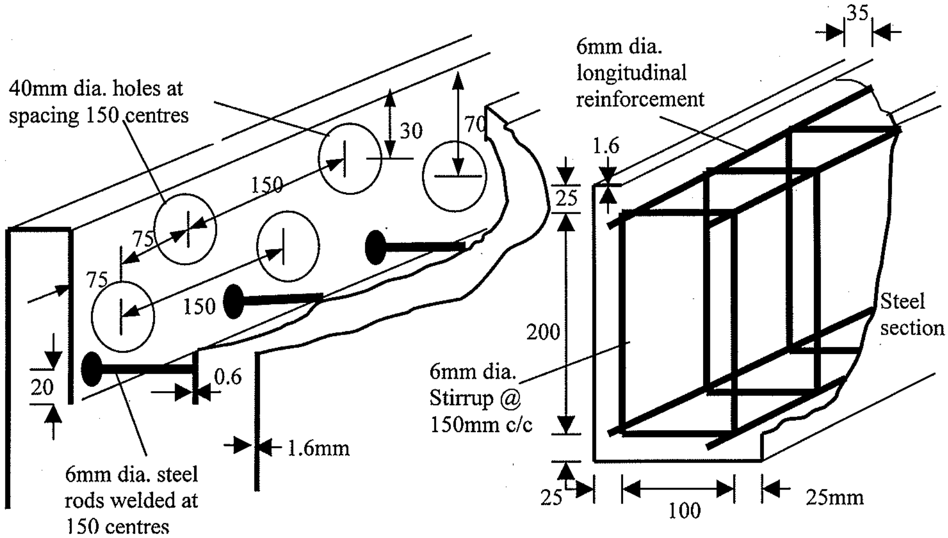
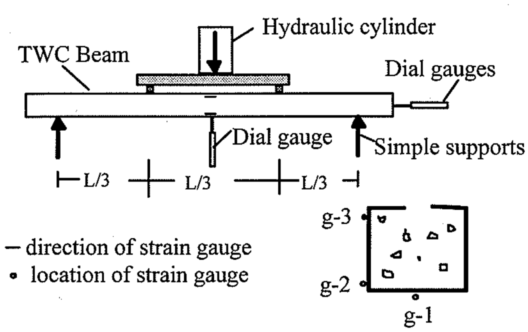


Fig. 2 Details of WE, WER and RC filled beams (C-series)



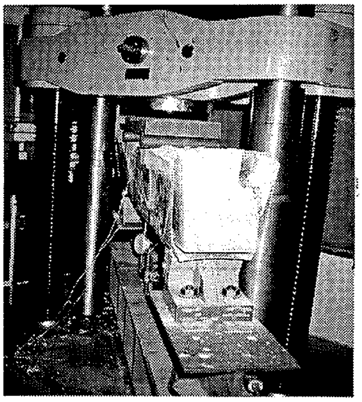


Fig. 3 Test set-up and instrumentation of composite beams

interaction between sheeting and concrete with 6 mm rods welded to the bottom of the extension plate at 150 mm centre to centre as shown in **Fig.2**. CB2-3-4 had welded plate extended to about one quarter of the depth (62.5 mm) while CB5-6 had welded extensions up to half of the depth of the beam (125 mm). To facilitate concrete casting and to ensure proper filling of the box section especially at the top, the extension plates (CB2-3-4-5-6) were punched with 40mm diameter holes as shown in **Fig.2**. There was no other specific reason to choose 40 mm dia holes although the geometry of the holes should affect the mechanical behaviour of the beam. However, the influence of geometry and size of the holes on the behaviour of such beams was not studied. RC beam CB7 was provided with only four 6 mm rods as longitudinal reinforcements while CB8 had similar longitudinal reinforcements but with 6 mm, U stirrups at 150 mm center to center (**Fig.2**).

(2) Casting and Curing of TWC beams

TWC specimens were cast in the Concrete Laboratory of Civil Engineering Department of Papua New Guinea University of Technology. Concrete was machine mixed and poured manually using a spatula. The specimens were laid down horizontally and concrete was poured through the top opening of the box section in an especially fabricated stand. Care had been taken to avoid lateral buckling of thin steel plates especially for series C beams. Concrete was compacted in layers with portable poker vibrator. Control specimens in the form of 100 mm x 200 mm cylinders and 100 mm cubes were also cast to determine the concrete strength. After 24 hours of casting, the control specimens and beams were demoulded and cured in open air until testing.

The infill concrete for TWC beams either VPC or normal concrete (NC) was made from 12.5 mm maximum size aggregates. The behaviour of thin steel sheeting under wet concrete was observed during the casting operation. The open box sections for beams behaved perfectly as formwork with no sign of outward buckling. The concrete casting was also much simpler than reinforced concrete.

However during casting, several problems were encountered in some beams. For beams with long welded extensions, the top part of the beam was not completely filled with concrete leaving void at the top that was later filled with cement paste. The problem was overcome in the later stages with provision of layers of holes in the welded extensions (**Fig.2**).

(3) Material properties

VP used in this study was collected from Tavurvur and Vulcan craters located in the Rabaul area of the East New Britain province of Papua New Guinea. Chemical analysis indicated that the VP is principally composed of silica (about 61%). However, VP has compounds such as calcium oxide, alumina and iron oxide (total about 30%). Mix proportions of NC and VPC are presented in **Table 1**. Both NC and VPC mixes were designed to have a 28-day cylinder compressive strength of 21 MPa. Locally manufactured ASTM Type I Portland cement was used. 12.5-mm maximum size crushed gravel having an oven dry density of 2470 kg/m^3 and 24-hour water absorption of 2.8% was used as coarse aggregate for NC. For VPC, 12.5 mm maximum size VP aggregate (VPA) having an oven dry density of 763 kg/m^3 and 24-hour water absorption of 37% was used as coarse aggregate. The bulk density results suggest that the VPA is much lighter than normal aggregate and also has high water absorption, which indicates high degree of porosity. Local river sand having an oven dry density of 2610 kg/m^3 and 24-hour water absorption of 0.6% was used as fine aggregate for both NC and VPC. Table 1 shows steel and concrete properties of Series A, B and C beams. Cylinder (f'c) strength of concrete as well as yield strength of steel plate (f_{sy}) and rods (f_y) are presented. The 28-day modulus of elasticity of NC and VPC were around 18 kN/mm^2 and 12 kN/mm^2 respectively. The 28-day splitting tensile strength of VPC was around 2.2 MPa compared to 3.1 MPa of NC.

(4) Instrumentation and Testing

The strain gauges were installed at mid span of series A and C beams as shown in **Fig.3** while no strain gauges were used in series B. The strain gauge 1 was installed at the centre of the bottom plate while gauges 2 and 3 were installed on the side steel plate 15 mm from the top and bottom.

Typical test setup for beam tested under two point loading condition is presented in **Fig.3**. The specimens were tested in a hydraulic testing machine. The load was applied at increments and at each load increment strains and central deformations were recorded to get complete load-deformation response. Central deflection was monitored by dial gauge. The strains were recorded by using a manual electronic strain measuring equipment.

Beams in Series A and C were tested under two point loading while those in Series B were tested under central single point loading condition. There was no obvious reason to use single point loading in Series B. The deformation and strain readings were recorded normally at 5 kN interval until the beams failed. The overall behaviour of the beams including failure modes, cracking of concrete and buckling of steel plates was observed during the entire loading history.

(5) Test observations and failure modes

The behaviour of the beams was affected by initial loss of chemical bond between the steel-concrete interface, presence of welded extension with or without welded rod and flexural and shear reinforcement in the beams.

a) Open beams (OS)

After initial stages of debonding in series A beams, the lateral separation between steel and concrete started due to compression (**Fig.4**) in the open top flange where the steel plate was free to deform laterally. The lateral separation between side steel plates and concrete started first with cracking sound and subsequent loss of chemical bond. As load increased, the bond between steel plate at the top open flange and concrete started to deteriorate and eventually increased compression force caused the steel plate especially between loaded points to peel out laterally with sound and finally buckled at about 90% of the ultimate load. The failure mode of these beams is named as "mode 1". The load and deflection steadily increased up to the point of buckling and after that, the continuing outward buckling (as shown in **Fig.4**) of the steel plate caused large deflection and reduced the strength of the beam significantly. The transition between start of buckling and failure was quick and finally the beam failed due to excessive outward buckling and cracking of in-fill concrete.

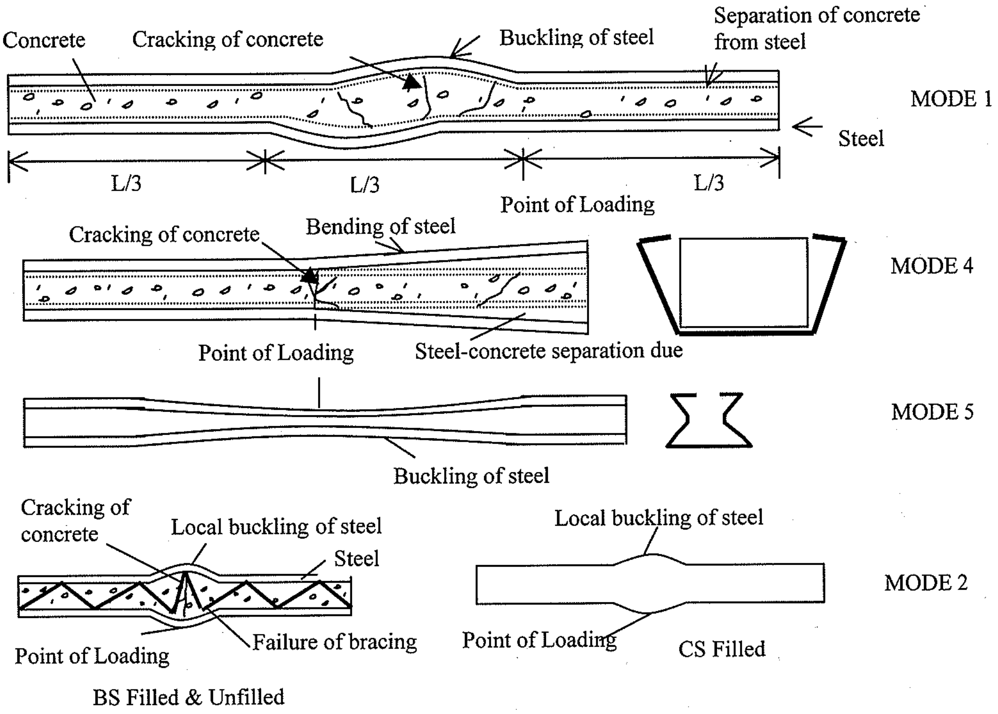


Fig. 4 Failure modes 1, 2, 4 & 5 of TWC beams (Series A & B)

The OS beams B3Inc and B4Inc exhibited different pattern of failure. B3Inc failed due to lateral separation of steel and concrete (mode 4) mainly extended from the centre to the end (support) as shown in **Fig.4**. B4Inc failed similar to beams in series A (mode 1) with buckling of steel plate and concrete cracking at the centre. This beam also showed lateral separation between steel and concrete throughout its length. The strength of such beams was less than braced, welded extension and fully closed beams.

Hence the OS filled beams failed due to lateral separation between sheeting and concrete either concentrated at the centre (Mode 1) or extended to the support (Mode 4). To avoid such lateral separation at the top open compression flange and to enhance better interaction, it was necessary to provide connections between sheeting and concrete in the form of bracing's or extensions as used in series B and C.

Unfilled OS beam B4I (steel section only) showed different mode of failure (mode 5) behaviour than its similar filled (B4Inc) counterpart. With progressive loading, the open side flanges were observed to buckle inwards (converging) forming a bow shape as shown in **Fig.4**. The beam section was badly damaged at the centre while original shape was maintained at the supports.

b) Braced Beams (BS)

BS filled beam B4III exhibited better response and the separation between steel and concrete at interface was negligible compared with OS beams. Typical failure mode of such beam (failure mode 2) is shown in **Fig.4**. Load capacity of braced filled beams was greatly increased and the welded braces were failed near the point of loading only after the ultimate capacity of both filled and unfilled B4III beams. Unfilled B4III braced beams failed in a similar manner to those of braced filled beams (mode 2). The welded braces restricted the outward buckling of the top steel plates and the failure was associated with outward buckling of steel skins and concrete cracking at the point of application of load at the centre.

c) Closed Beams (CS)

The performance of CS beams was better than OS, BS, WE beams. CS beams B3II and B4II beams with NC and VPC showed failure modes (mode 2) similar to those of braced beams as shown in **Fig.4**. However, the strength of such beams was greater than the braced beams.

d) WER Beams (C-series)

The failure (mode 3) WER beams CB3 and CB6 can be described as shown in **Figs. 3** and **5**. During loading process, the beams underwent flexural deflection (stage 1), followed by separation (stage 2)

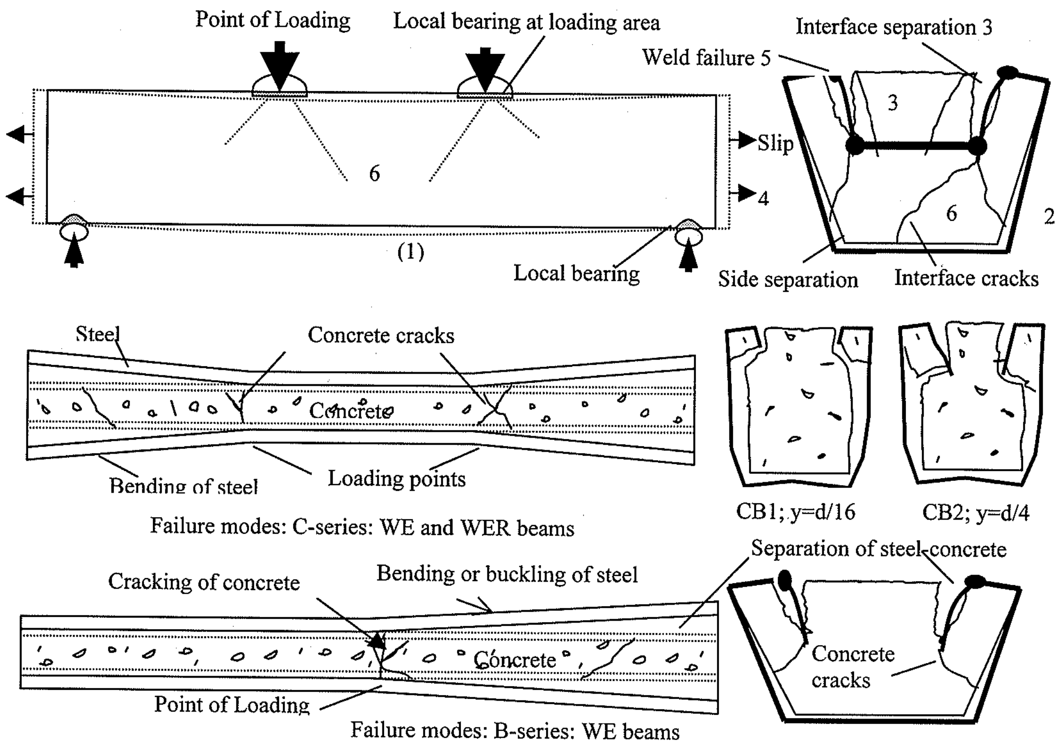


Fig. 5 Failure mode 3 of WER and WE beams

of side steel plates from concrete after initial cracking or debonding.

The welded extensions with rod acted as mechanical interlock and anchored the top steel plate to the concrete. The lateral separation of sheeting from concrete in such beams was delayed. With continued loading, concrete cracked at the interface of extension plate (stage 3) causing interlocking force to deteriorate that initiated the lateral separation (stage 4) of steel at the top. At the same time, slip between steel and concrete (stage 4) was observed at the ends of the beam. As the lateral separation continued, the tag welds between extension plate and main steel skin (stage 5) started to fail. This enhanced the outward separation of steel and widened the gaps between extension plate and concrete core. Before the ultimate load, interface cracks propagated to the sides and deep insight into the concrete core. At the failure stages, side steel plates buckled outward and took the cracked concrete with it. After the ultimate load, the steel skins buckled outwards diagonally (stage 6) at the loaded points, which was followed by local bearing at the supports and the loaded points (stage 7). Lateral separation of the plate started from ends of the beam and propagated towards the loaded point as shown in **Fig.5**.

e) WE beams (B and C series)

The WE beams CB2, 4 and 5 showed similar behaviour (mode 3) to that of WER beams although they had lower strength. The load capacity of such beams was observed to increase with the increase of the depth of welded extension.

WE beams B3d/4 and B3d/2 also showed similar failure mode 3. Stages 5 and 7 of failure mode 3 were absent in these beams. This was due to the presence of continuously welded extension plate (rather tag welded) and shallower nature of beams compared with C beams. The outward buckling normally propagated from one end of the beam to the centre as shown in **Fig.5** while the other end showed no sign of such buckling. This might be due to the initiation of interface crack at one end and subsequent collapse of the affected end. The extension of interface cracks depends on the depth of extension. The typical failure cross-section of WE beams is shown in **Fig.5**. The beam CB1 closely represented an OS beam as WE was only 1/16th of the depth of the beam.

f) Reinforced TWC beams

CB7 and CB8 showed similar behaviour to that of CB1. They produced higher strength compared to CB1 and the presence of stirrups in CB8 enhanced the strength.

Table 2 Comparative study of beams

Parameters	Series A	Series B	Series C
Width to depth	0.5 and 1.0	0.5 and 1	0.6
Span/depth	15	6	4
Depth/sheet thickness	31.2 and 43.5	31.2 and 43.5	156.3
Loading	Double point	Single point	Double point
Type of section	SHS and RHS	SHS and RHS	Made in the laboratory from plain sheet
SHS: Square hollow section; RHS: Rectangular hollow section			

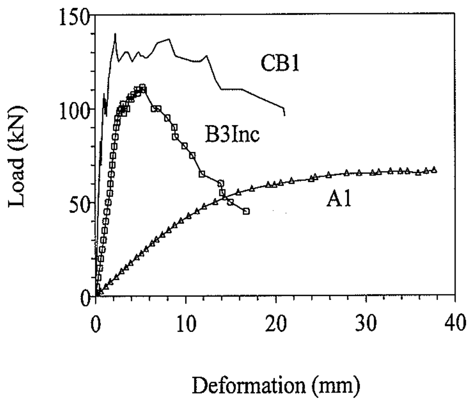


Fig. 6a Comparative study of load-deflection responses

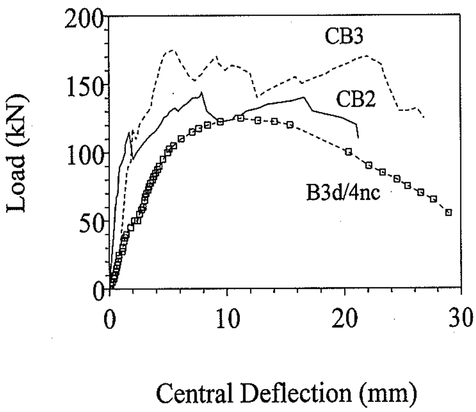


Fig. 6b Comparative study of load-deflection responses

(6) Comparative study of load-deformation responses

Figs.6 show typical load-deflection responses of beams from series A, B and C. The load-deflection responses of series A and B were quite similar while series C showed a response with some distinctive features. For series A and B (**Fig.6a**) the load was increased uniformly to the peak level and then decreased. The side plate debonding, peeling of the steel plate at the top and outward buckling of the steel section had not caused any sudden drop in loads. The series C beams had pronounced drop and rise in loads during the loading history (**Fig.6b**) which could be related to some parametric influence. **Table 2** presents various geometric and loading parameters of all the beams.

Series A and B beams were made of hollow box sections which had high span to depth ratio (L/d) and low width to depth (b/d) and depth to steel thickness (d/t) ratios compared with C beams. This made steel sections in series A and B much stiffer than those of C beams. High (almost 5 times) d/t in series C beams influenced the modes of side plate separation, peeling and failure of welded extensions at the top compression zone and eventually lateral buckling of the steel plates. Buckling load of side steel plates increases with the decrease of d/t and such beams can be designed to ensure failure either by buckling or yielding of steel plates through appropriate selection of d/t. To ensure failure due to yielding of steel plate, the geometry of box section should be chosen with a d/t so that the buckling stress becomes higher than the yield stress of steel plate. The C beams (as shown in **Fig.6b**) showed several peaks during the entire loading history. The first peak indicated the beginning of side plate separation when the load dropped momentarily. After that the beam regained its strength and the load rose up to the second peak. The second peak indicated the start of peeling of extension plates from concrete at the top and cracking of concrete at the extension plate-concrete interface. After the second peak, the load dropped momentarily and then started to rise again. The beams (CB1, CB2 and CB3) showed several peaks with widening and progressive propagation of cracks before finally failed. The several rises in the loads could be associated with the enhancement of strength due to the presence of welded rods and extension plates (beams CB2 and CB3). However, the ultimate load was governed by the second peak although they showed several subsequent peaks. On the other hand, the B beams with welded extension (as shown for B3/4nc) did not show several peaks.

Table 3 Comparative studies of Series A beams

Test beams	Side plate debonding (kN)	Peeling Load (kN)	Buckling Load (kN)	Buckling Deflection (mm)	Ultimate Load (kN)	Ultimate Deflection. (mm)
A1	20.0 (30%)	52.5 (79%)	62.0 (93%)	15.89 (42%)	66.5	37.68
A2*	22.5 (37%)	42.5 (69%)	57.5 (93%)	15.72 (52%)	61.5	30.22
A3	15.0 (50%)	25.0 (83%)	27.5 (92%)	10.05 (58%)	30.0	17.32
A4*	17.5 (56%)	22.5 (73%)	30.0 (97%)	10.52 (59%)	31.0	17.96

* VPC beams Concrete strength : f'c = 21MPa (NC) ; 20 MPa (VPC)
Values in the bracket show the % based on ultimate load and deflection
Mode of failure of all beams: Mode 1

Table 4 Performance of Series B beams

Beam	Buckling Load (kN)	Buckling Deflection (mm)	Peak Load (kN)	Peak Deflection (mm)	% increase in Load Peak	% increase in Load Buckling	% increase in Deflection Peak	% increase in Deflection Buckling	Type & Failure Modes
B3Inc	88 (79)	2.73 (50)	112	5.5	-	-	-	-	OS; 4
B3IInc	130 (82)	3.84 (44)	158	8.7	41	48	58	41	CS; 2
B3IIvpc	110 (77)	3.71 (56)	143	6.7	28	25	21	36	CS; 2
B3d/4nc	110 (88)	6.11 (56)	125	11	11	25	100	124	WE; 3
B3d/2nc	117 (92)	7.38 (49)	127	15	14	33	173	170	WE; 3
B4Inc	67 (95)	3.92 (59)	70.6	6.62	-	-	--	--	OS; 1
B4IInc	100 (97)	6.17 (55)	103	11.27	45	49	70	57	CS;2
B4IIvpc	80 (82)	4.57 (51)	97.5	8.88	38	19	34	27	CS;2
B4IIInc	70 (83)	4.04 (51)	84	7.94	19	4	20	38	BS;2
B4I	34.5 (91)	4.37 (34)	38.1	12.94					OS;5
B4III	37 (97)	4.24 (38)	38.3	11.27					BS;2

Buckling load and Deflection: Start of local buckling or buckling causing lateral separation
OS, WE, CS, BS: open, welded extension, closed, braced section
1, 2, 3, 4, 5: Modes of failure
Values in the bracket show the % based on ultimate load and deflection

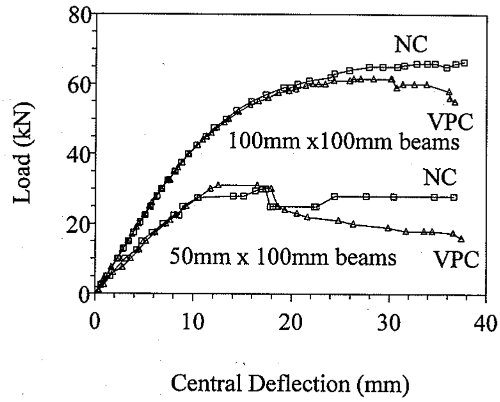


Fig.7a Effect of NC and VPC in A beams

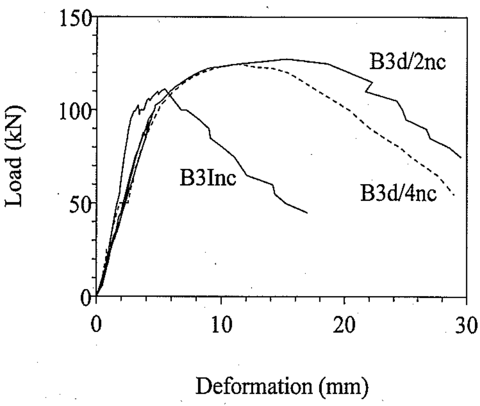


Fig. 7b Effect of welded extension

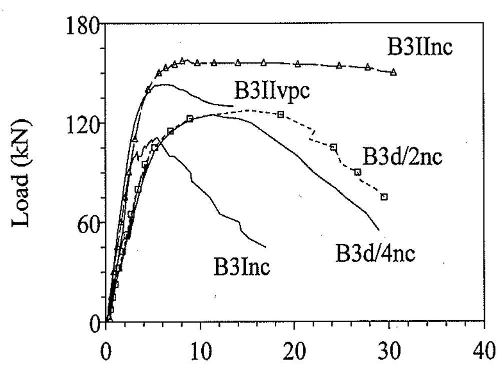


Fig. 7c Comparison of WE and CS beams

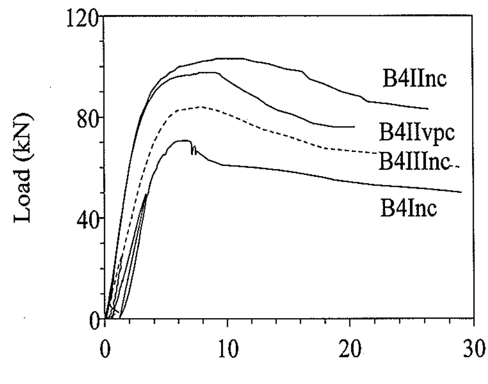


Fig.7d Study of OS, BS and CS beams

3. PERFORMANCE STUDY

(1) Series A

These beams were slender and the variables were cross-sectional geometry and type of concrete. Both VPC and NC beams showed similar load-deflection responses (**Fig.7a**) and failed in the similar manner. **Table 3** shows a comparative study of the beams performance. The side plate debonding seemed to be dependent on the cross-sectional geometry of the beam and found to be increased from 30-37% to 50-60% when geometry was changed from square (100 x 100 mm) to rectangular (50 x 100 mm) section. The peeling (69%-83%; averaging 76%) and buckling (92%-97%; averaging 94%) loads seemed to be not sensitive to the change in cross-sectional geometry. The beams underwent large deformation, about two times that at buckling load, between the transition from buckling (averaging 94%) to ultimate load. The effect of VPC on the peeling, buckling and ultimate loads and deflections of beams was found similar to NC.

(2) Series B

Series B studied the effect of various parameters such as: presence of bracing, type of concrete and welded extension.

a) Effect of welded extension

Fig.7b and **Table 4** show the effect of welded extension on the strength and overall behaviour of TWC beams. The buckling and ultimate (peak) strengths of the beams increased with the increase of the depth of welded extension. The ultimate strengths of the beams with welded extension of d/4 (B3d/4nc) and d/2 (B3d/2nc) were increased by 11.17% and 13.8% respectively compared with OS beam (B3Inc). On the other hand, buckling strengths were also increased by 25% and 33% respectively.

The ductility (measured on the basis of peak deformation) of WE beams (B3d/4nc and B3d/2nc) was also higher than B3I beam (OS) as could be seen from the 100% and 173% increase in the ultimate (peak) deformation. In addition, buckling deformation was also increased by 124% and 170% (**Table 4**). Only 2.6% increase in ultimate strength was observed compared with 73% increase in ductility when the length of the welded extension was increased from d/4 to d/2. The welded extension beams underwent large deformation, about two times that at buckling load, between the transition from buckling (averaging 90%) to ultimate load.

b) Effect of closed and braced sections

The ultimate strengths (**Table 4** & **Fig.7c**) of CS beams (B3IInc and B3IIvpc) were found to be higher than OS (41.3% and 28% respectively) and WE beams (26.4% for d/4 and 24% for d/2). The ductility of CS beam was higher than OS beam (58%) but lower than WE beams (27% compared with d/4 and 73% compared with d/2).

Similar behaviour was observed in series B4 beams and **Fig.7d** compares the load-deflection response of such beams. The ultimate strength of CS beams (B4IInc and B4IIvpc) was higher (45% and 38% respectively) than OS beam (B4Inc). CS beams were also ductile (70% for NC and 34% for VPC) than OS beams. The ultimate strength of BS beam (B4IIInc) was higher than OS beam (B4Inc) by 19% but less than CS beam (B4IInc) by about 23%. The ductility of the BS beam was lower (42%) than CS beam but higher (17%) than OS beam. The buckling load (ranging between 77% and 97% of ultimate load) of CS beams was also increased (48-49% for NC and 19-25% for VPC) compared with OS beams. For BS beams, buckling load (83% of ultimate load) was increased by only 4% while the ductility was increased by 38% compared with OS beams.

Fig.7e compares the performance of CS beams in series B3 and B4. It is revealed that both NC and VPC beams performed in similar manner when compared on the basis of strength and ductility, although the strength of NC beams was 5-10% higher than VPC beams having similar concrete strength.

c) Performance unfilled beams

The unfilled steel sections were also tested in series B4 to study the interaction of steel and concrete in filled sections and also to study the influence of bracing. **Fig.7f** compares the load-deformation responses of such beams. The presence of bracing in unfilled sections did not improve the strength (only 0.5% increase) but the failure mode had been changed from 5 to 2. The buckling load seemed to be increased by 7%. The strength of the filled beams was substantially higher (46% for open and 54% for braced) than the individual strength of the steel section alone.

(3) Series C

Series C studied the behaviour of TWC beams with various strength enhancement devices such as: welded extension, welded extension with rod, longitudinal and shear reinforcements. **Table 5** summarizes the load and deflection characteristics of series C beams.

a) Effect of welded extension and rod

Fig.8a shows the effect of welded extension (of length d/4) with rod on C beams. WE beams (CB2

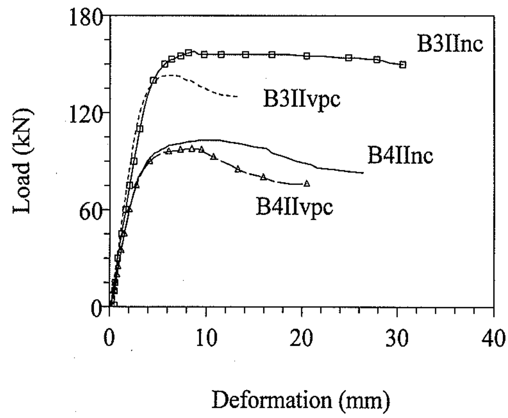


Fig. 7e Effect of NC and VPC (CS beams)

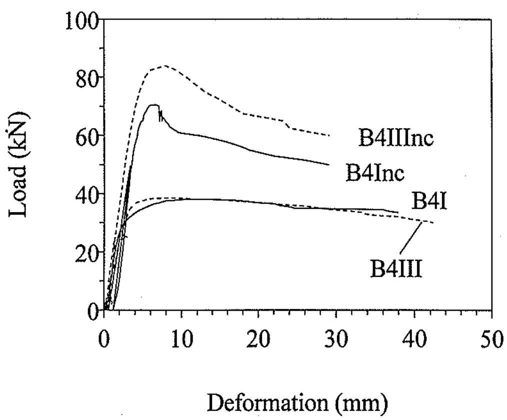


Fig.7f Filled and unfilled beams (OS & BS)

Table 5 Comparative study of series C beams

Beam	Section and failure modes	1st peak load *			2nd peak (ultimate) load **				3rd or 4th peak load***	
		Load kN	Ratio	Deflection mm	Load kN	(1)	Deflection mm	(2)	Load kN	Deflection mm
CB1	OS; 3	100 (71)	--	1.02 (45)	140	--	2.25	--	137(98)	8.23 (366)
CB2	WE;3;d/4	108 (75)	1.08	1.35(17)	144	3%	7.87	250%	140(97)	16.61(211)
CB3	WER;3;d/4	90 (51)	0.90	1.99(40)	177	26%	4.98	121%	173(98)	22.02(442)
CB4	WE;3;d/4	118 (69)	1.18	1.12(19)	170	21%	6.05		163(96)	9.14(151)
CB5	WE;3;d/2	105 (72)	1.05	0.96 (44)	147	5%	2.17	--	128(87)	10.67(492)
CB6	WER;3;d/2	90 (38)	0.90	1.94 (28)	235	69(60)	6.93	219%	215(91)	14.87(215)
CB7	OS;MS;3	87 (45)	0.87	2.47(35)	198	41%	7.12	216%	183(92)	9.42(132)
CB8	OS,MR,SR;3	90 (40)	0.90	2.14(32)	225	61%	6.68	197%	223(99)	6.68(132)

* Side plate debonding
** welded extension failure or peeling of welded extension and crack initiation at the interface
*** due to interaction of crack propagation, welded rods and extension
values in the brackets are expressed as % of 2nd peak load and deflection
percentage values in (1) and (2) represent increase in load and deflection based on CB1

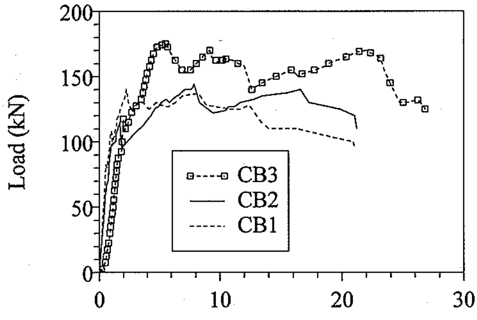


Fig. 8a Effect of welded extension

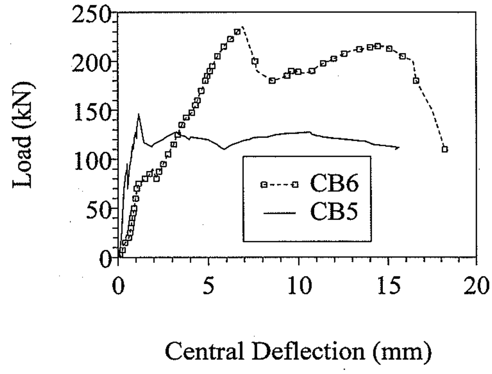


Fig. 8b Effect of welded rod

and CB3) produced higher strength (3% and 26% respectively) than OS beam (CB1) as found in the case of series B. The presence of rods at the end of welded extension enhanced the strength (**Table 5**) of CB3 beam by 23% compared with CB2 beam without welded rod. The ductility up to the ultimate load was increased by about 250% for CB2 (WE) and by about 121% for CB3 (WER) compared with CB1. It is interesting to note that although the use of welded rod enhanced the strength of CB3 (WER) but it seemed to produce lower ductility than CB2 (WE).

Fig.8b compares the performance of CB6 compared with CB5. Both beams had welded extension up to a depth of d/2 but CB6 is provided with welded rod. The strength and ductility of CB6 (WER) were 60% and 219% more than CB5 (WE) respectively (**Table 5**). The enhancement of ductility seemed to be dependent on the location of the welded rod. The presence of welded rod at d/2 enhanced the ductility of CB6, while the presence of rod at d/4 reduced the ductility of CB3. This was due to the fact that the provision of welded rod at greater depth allowed more volume of concrete to be effective in the zone of influence of welded extension-rod assembly.

Fig.8c shows the comparative performance of RC beams CB7 and CB8 with longitudinal and shear reinforcement compared with CB1 without such reinforcement.

The strength of CB7 and CB8 was increased by 41% and 67% respectively while the ductility was also enhanced by 216% and 197% respectively (**Table 5**).

The strength and ductility of CB6 (WER) beam were higher than any other beam in the series C. This implies that it will be possible to design TWC beams with welded extension and rod assembly without using longitudinal and shear reinforcements.

The first peak load associated with the side plate debonding seemed to be not sensitive to the strength

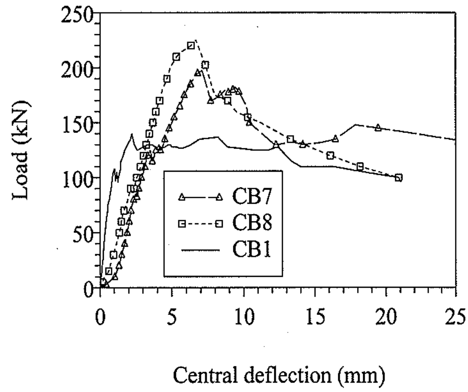


Fig. 8c Effect of main and shear reinforcement

enhancement parameters as can be seen from **Table 5**. The ratio of 1st peak load of all the beams compared with CB1 ranged between 0.87 and 1.18 with a mean value of 0.96. The 3rd or 4th subsequent peak loads were less then 2nd peak load and their percentages with respect to 2nd peak load ranged between 87% and 99%.

4. STRAIN CHARACTERISTICS

(1) A-series

The typical variation of bending strains in series A beams with VPC and NC in-fill is shown in **Fig. 9a-b**. Both VPC and NC beams showed similar pattern of variation. The compressive strain in gauge 1 seemed to be similar in both types of beams although the tensile strain in gauge 3 showed higher values for VPC beams. But it seemed to be that the type of concrete had less influence on the strain. Both gauges 1 and 3 showed attainment of yield strain in the steel at the final stages before ultimate load. **Table 7** summarizes the yielding and ultimate

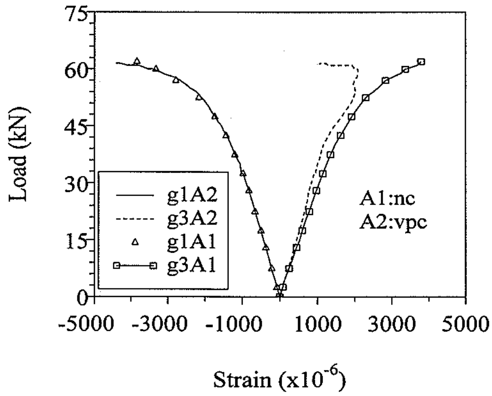


Fig. 9a Strain characteristics of series A beams

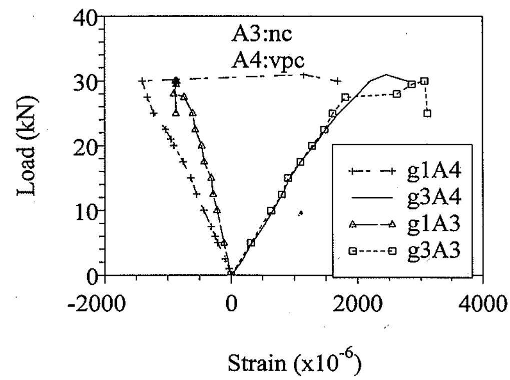


Fig. 9b Strain characteristics of series A beams

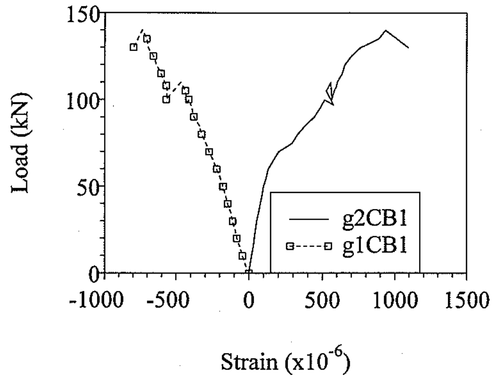


Fig. 9c Strain characteristics of series C beams

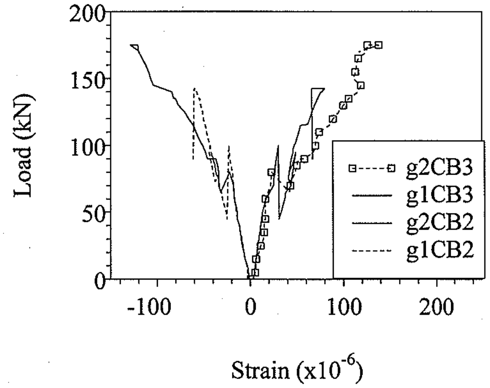


Fig. 9d Strain characteristics of series C beams

load situations in series A beams. The yield load ranged between 75% and 97% of the ultimate load.

(2) Series c

The strain characteristics of three C beams (CB1, CB2, and CB3) are shown in **Figs.9c-d**. The strain patterns in all three beams were similar. Strain gauge 1 at the top of the beam registered compressive strain while gauge 2 at the bottom registered tensile strain. For all the beams, the strain pattern did not change much until side plate debonding was noticed. At this point, change in strain was noticed with a sudden drop and rise in strain.

The strain characteristics of C-beams were affected by the steel-concrete debonding, peeling of steel at the top, concrete cracking, welded extension failures and buckling of steel plates with lateral separation. This could be attributed to the rise and fall exhibiting a saw tooth behaviour in strain pattern after the side plate debonding especially in CB2 and CB3 beams. The presence of welded extension and rod assembly significantly reduced both tensile and compressive bending strains in CB2 and CB3 beams compared with CB1 without welded extension (**Figs. 9c-d**).

(3) Analysis of strain (series A and C)

Table 6 summarises the strain conditions in yielding and failure stages of the beams. Tensile strains in Series A beams exceeded yield strain at about 75 to 96% of ultimate load. Series A beams also registered the attainment of compressive yield strain at about 79 to 98% of ultimate load except for A3 beam that only registered about 45% of yield strain at ultimate load. This indicated that analytical models for these beams could be developed based on the yielding of the tensile steel plate.

Tensile strains in CB1 (OS) exceeded yield strain at about (93%) of the failure load. None of the strain gauges in CB2 (WE) and CB3 (WER) registered yield strain. They registered only 3 to 7% of yield strain of steel (**Table 6**). This might be due to the more composite action between steel and concrete in CB2 and CB3 compared to CB1 that also lead to the higher ultimate load.

Table 6 Strain conditions in yielding and failure stages (Series A and C)

Beams	Yielding stage				Failure stage		
	Compressive		Tensile		Load	Strain	
	Load	Strain	Load	Strain		Tensile	Comp.
A1 : nc	52.5(79%)	2161	50(75%)	2104	66.5	y	y
A2 :vpc	52(85%)	2042	54(88%)	2022	61.5	y	y
A3 :nc		ny	28(93%)	2628	30	y	-878(45%)
A4 :vpc	30.5(98%)	2954	30(96%)	2206	31	y	y
CB1:OS		ny	130(93%)	2141	140	y	-735 (38%)
CB2:WE		ny		ny	144	60 (3%)	-66 (3%)
CB3:WER		ny		ny	177	129 (7%)	-138(7%)

y: yielded ny: not yielded Strains are in micro-strain
Values in the brackets indicates % based on ultimate load and yield strain

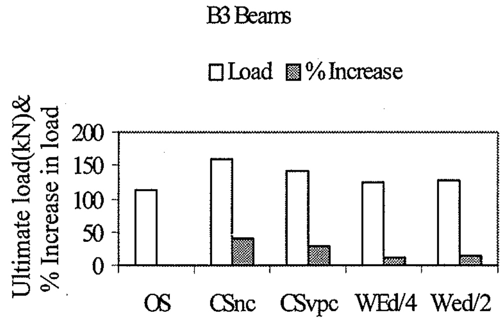


Fig. 10a Effect of mode of connection

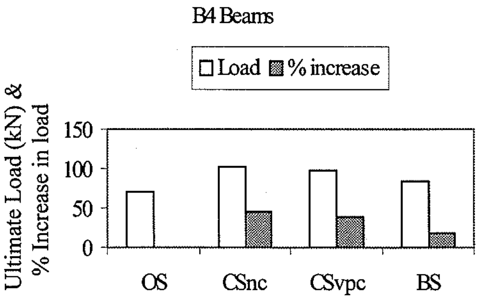


Fig. 10b Effect of mode of connection

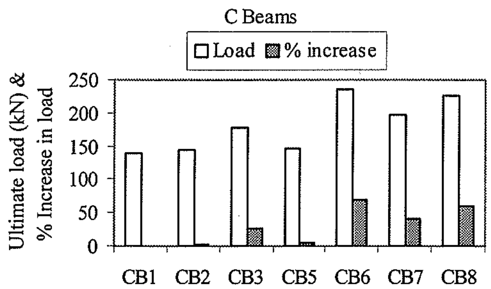


Fig. 10c Effect of mode of connection

5. GENERAL DISCUSSION ON MODES OF CONNECTION

The strength of the beams for various modes of connections and reinforcements are compared in **Fig.10**. **Fig.10a** compares the strength of B3 beams. It is found that the strengths of CS (41% increase for NC and 28% increase for VPC) and WE (increase of 11% for y = d/4 and 14% for y = d/2) beams are higher than the respective OS beams.

From **Fig.10b**, it can be concluded (for B4 beams) that the strengths of CS (45% increase for NC and 38% increase for VPC) and BS (19% increase) beams are higher than the respective OS beams.

The performance of CS beams suggests that it is more effective to use a closed steel skin such as rectangular concrete filled tube (CFT) beams if it is convenient from actual construction point of view. The better performance of CS beams can be attributed to (i) better confinement of in-fill concrete, (ii) better composite action due to enhanced sheet-concrete interaction and (iii) enhanced buckling capacity of steel plate.

On the other hand the performance of OS beams confirmed that the composite action between steel and concrete can not be ensured only by relying on the bond strength of concrete. The better performance of WE and WER beams compared to OS beams confirmed the fact.

Fig. 10c compares the strengths of C beams. It can be concluded that the strengths of WE (increase of 3% for y = d/4 and 5% for y = d/2) and WER (increase of 26% for y = d/4 and 69% for y = d/2) beams are higher than the respective OS beams. RC beams with main and shear reinforcements also registered higher strength with 41% and 61% increase.

6. CONCLUSIONS

Comprehensive series of 23 tests on TWC beams provided information on the load-deformation response, failure modes, stress-strain characteristics, effect of strength enhancement devices and comparative performance with VPC and NC infill. The following conclusions can be drawn from the study:

The behaviour of the beams was affected by initial loss of chemical bond at steel-concrete interface, lateral interface separation (side plate debonding) and local buckling of sheeting.

To avoid lateral separation between sheeting and concrete at the top open compression flange and to enhance interaction and strength, it is necessary to provide additional interface connections such as welded extension with or without rod.

The performance of closed beams was better than open, braced and welded extension beams. The strengths of CS beams are found to be 28% to 45% higher than open and 24% to 26% higher than WE beams. The ductility of CS beams is also higher than OS beams (ranges between 58% and 70%) but lower than WE beams (ranges between 27% and 73%). The strength of the BS beams is higher than OS beam by 19% but less than CS beams by about 23%. The ductility of the BS beams is lower (42%) than CS beams but higher (17%) than OS beam.

Both NC and VPC beams performed in similar manner when compared on the basis of strength and ductility, although the strength of NC beams are 5-10% higher than VPC beams having similar concrete strength.

WE beams produced higher strength than OS beams. It can be concluded that the strengths of WE (increase 3% for $y = d/4$ and 5% for $y = d/2$) and WER (increase of 26% for $y = d/4$ and 69% for $y = d/2$) beams are higher than the respective OS beams. The strength of WER beam is increased by 23% compared with WE beam. The ductility up to the ultimate load is increased by about 250% for WER beam and by about 121% for WE beam compared with OS beam.

It is interesting to note that although the use of welded rod enhances the strength of the beam but it seems to produce lower ductility. The enhancement of ductility seems to be dependent on the location of the welded rod. The presence of welded rod at greater depth enhances the ductility as it allows more volume of concrete to be effective in the zone of influence of welded extension-rod assembly.

Presence of flexural and shear reinforcements in the beams provides better performance. The reinforced TWC beams produced higher strength and the presence of stirrups in such beams also enhanced the strength. RC beams with main and shear reinforcements also registered higher strength with 41% and 61% increase than the beams without reinforcements.

The strength and ductility of WER beams are higher than any other beams, which imply that it will be feasible to design such TWC beams in practical circumstances.

The side plate debonding and buckling are found to be dependent on the cross-sectional geometry of the beam and degree of plate restraint at the edges. To develop theoretical equations, buckling stress of side steel plates should be checked with appropriate boundary conditions. The degree of restraint achieved in beams with various modes of connection devices is to be identified based on good agreement between experimental and theoretical capacities.

The design of such beams should include the following:

- A check whether the design should be based on yielding of steel or buckling of side steel plates.
- Identification of degree of interaction between sheeting and concrete with various mode of connections

TWC beams with VPC exhibited satisfactory performance compared with normal concrete that validated the viability of the use of VPC in such construction.

REFERENCES

1) Hossain, K.M.A.: Behaviour of thin walled composite sections as structural elements, *Proce. of the Australasian Structural Engineering Conference*, Auckland, 30 Sept. - 2 October, Vol. 1, pp. 175-180, 1998.
2) Hossain, K.M.A.: Performance of volcanic pumice concrete with especial reference to high-rise composite construction, Innovation in Concrete Structures: design and construction, Ed. R.K. Dhir and M. R. Jones, Thomas Telford Limited, London, UK, pp. 365-374, September, 1999.
3) Hossain, K.M.A. and Mol, L.: Thin walled composite construction using volcanic pumice concrete, *Proce. of the IEPNG International Conference*, September 25-27, Rabaul, Papua New Guinea (PNG), pp. 17-24, 1998.
4) Hossain, K.M.A., Mol, L. and Wright H.D.: Behaviour of thin walled composite beams and columns, *Proce. of 2nd International Conference on Thin-walled Structures*, Singapore, 2-4 December, pp. 12, 1998.
5) Oehlers, D. J., Wright, H. D. and Burnet, M. J.: Flexural strength of profiled beams, *Journal of Structural Engineering, ASCE*, Vol. 120, No. 2, pp. 378-393, 1994.
6) Oehlers, D. J.: Composite profiled beams, *Journal of Structural Engineering, ASCE*, Vol. 119, No. 4, pp. 1085-1100, 1993.
7) Uy, B. and Bradford, M.A.: Ductility of profiled composite beams: Part 1: Experimental Study, *Journal of Structural Engineering, ASCE*, Vol. 121, No.5, pp. 876-882, 1995.

8) Uy, B. and Bradford, M.A.: Ductility of profiled composite beams, Part 2: Analytical study, *Journal Structural Engineering, ASCE*, Vol. 121, No. 5, pp. 883-889, 1995.
9) Oduyemi, T.O.S and Wright, H. D.: An experimental investigation into the behaviour of double skin sandwiched beams, *Journal of constructional Steel Research*, Vol. 14, No. 3, pp. 197-220, 1989.
10) Wright, H. D, Oduyemi, T. O. S. and Evans, H. R.: The design of double skin composite elements, *Journal of constructional Steel Research*, Vol. 19, No. 2, pp. 111-132, 1991.
11) Uy, B. and Bradford, M. A.: Inelastic local buckling behaviour of thin steel plates in profiled composite beams, *The Structural Engineer*, Vol. 72, No. 16, pp. 259-267, 1994.
12) Hossain K.M.A.: Behaviour of thin walled composite beam, Research Report No. CE-SE00-1, Department of Civil Engineering, The Papua New Guinea University of Technology, Lae, pp. 121, 2000.
13) Mol, L.T.K.: Behaviour of thin walled composite structural elements, MPhil Thesis, Department of Civil Engineering, University of Technology, Private Mail Bag, Lae, Papua New Guinea, pp. 215, 2001.
14) Hossain, K.M.A.: Development of mix-design charts for volcanic ash and pumice concrete, Research Report No. CE-SE97-5, Department of Civil Engineering, PNG University of Technology, Lae, Papua New Guinea, pp. 41, 1997.
15) Hossain, K.M.A.: Properties of volcanic ash and pumice concrete, *IABSE Report*, Vol. 80, 1999, pp. 145-150, 1999.

(Received July 18, 2003)