Axial load behaviour of pierced profiled composite walls

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This paper compares the behaviour of a novel form of pierced and non-pierced profiled composite walls under axial loading. Previous studies concentrating on non-pierced walls confirmed their potential to be used as vertical and lateral load-resisting structural elements. The presence of the holes and their location in pierced walls can significantly affect the behaviour of the walls. Eight pilot tests on walls manufactured from two different types of profiled steel sheeting were carried out to provide information on the effect of holes on load-deformation response, strength, stiffness, stress-strain condition, buckling and failure modes. This investigation explores the potential application of composite walling as shear or core walls in buildings allowing openings for doors and windows. The strength of the pierced composite wall is reviewed in relation to existing Codes of practice and other available design formulations to develop design guidelines for such walls.

Keywords: *composite wall, profiled steel sheeting, pierced concrete, load deformation response, buckling*

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1. Introduction

Composite walling refers to a new building system consisting of two outer skins of profiled steel sheeting with an infill of concrete. Its development has come about as an extension of the now well known composite flooring system used worldwide. As with composite flooring, the advantages of composite walling lie in the speed and convenience of construction [1,2]. Similar systems have been used as missile- and blast-resistant walls, although these have normally been built on loose sand or stone rather than concrete infill. The composite walling described herein was originally conceived for use as a shear or core wall to stabilise steel frame building structures, although it has potential in concrete buildings, basements and blast resistant structures. It can be noted that the steel sheeting will act to stabilise the building frame [1] as soon as it is fixed, and provides permanent formwork for the infill concrete. Once the concrete has hardened, axial load, lateral load and in-plane loads will be carried through both the steel and concrete.

Previous studies on non-pierced composite walls under axial [2] and in-plane shear [1,3] loading have shown that adequate load transfer devices in the form embossments or other mechanical connections between sheeting and concrete are necessary to fully mobilise the composite action and to improve wall performance. They also confirmed their potential use as viable alternative to reinforced concrete and masonry walls in frame structures. Fig. 1 shows a schematic diagram of the wall, allowing openings for doors and windows in a steel frame building. This paper describes the axial behaviour of a pierced wall compared with a non-pierced wall, based on eight pilot tests. The strength of the currently tested composite walls will be compared with those based on BS8110 and other available formulations with discussion on the effect of holes in the walls.

2. Experimental investigation

Eight pilot tests on walls using two different types of profiled sheeting such as Spandek (0.42 mm thickness) and Trimdek (0.4 mm thickness) were carried out. Both Spandek and Trimdek are used as roofing material in Papua New Guinea and have profiled dimensions (Fig. 2) smaller than those normally used for composite slabs. Various parameters such as geometry of profiled steel sheeting, size-shape-position of the holes, different mode of connections and load transfer devices were considered in the investigation.

2.1 Wall details

The dimensions and connection details of the walls are presented in Fig. 2. The overall dimension was 900 x 830 mm for Trimdek walls and 900 x 680 mm for Spandek

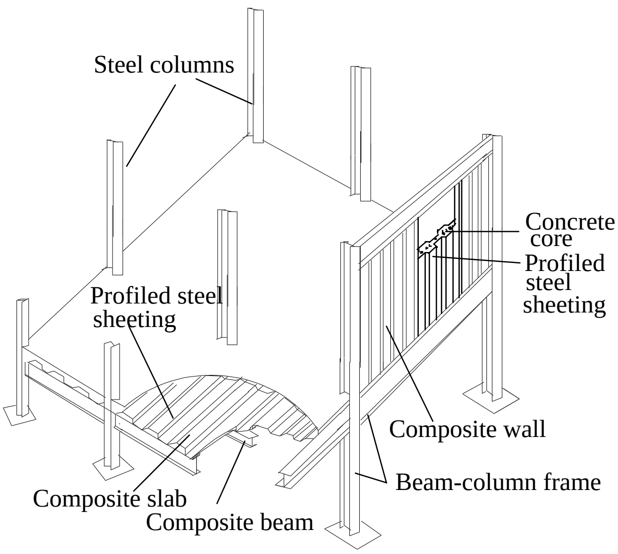

FIGURE 1: Schematic of composite wall in a building.

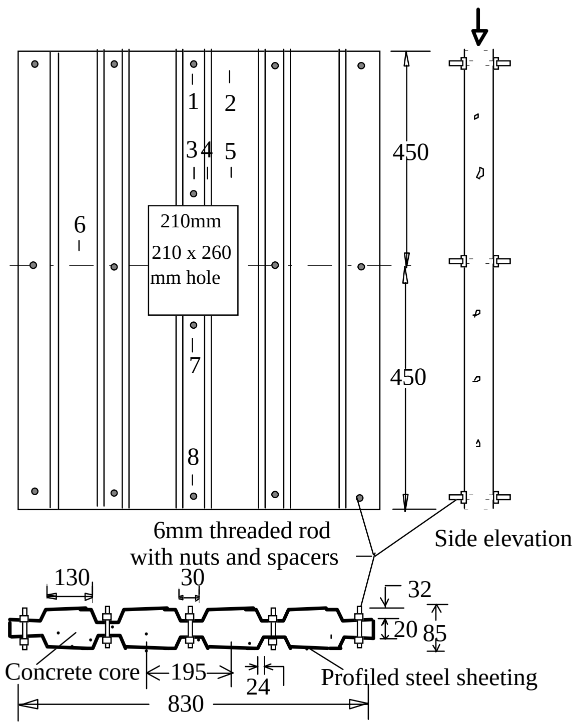

FIGURE 2(a): Detailed description and instrumentation of Trimdek pilot walls

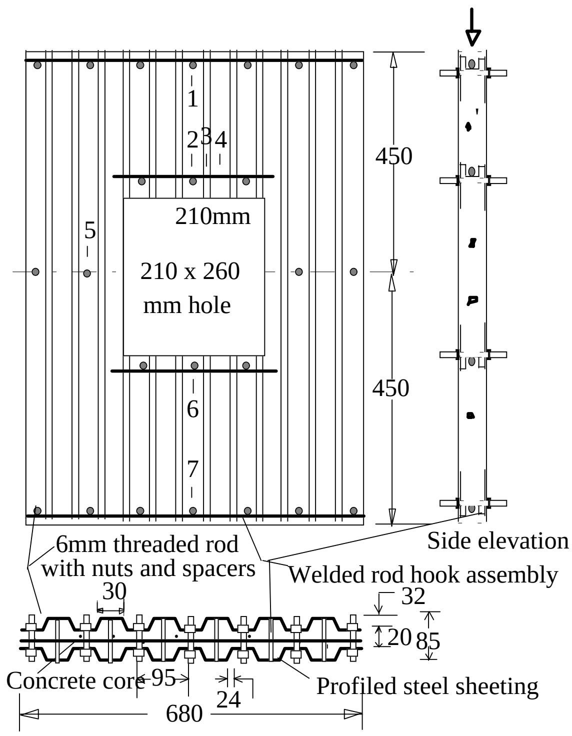

FIGURE 2(b): Detailed description and instrumentation of Spandek pilot walls

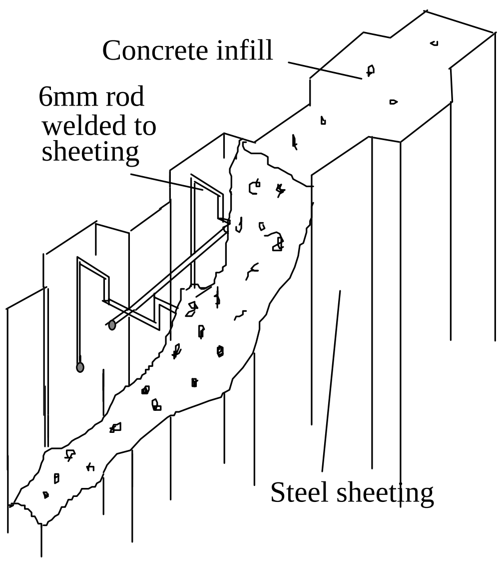

FIGURE 2(c): Welded rod and hook assembly

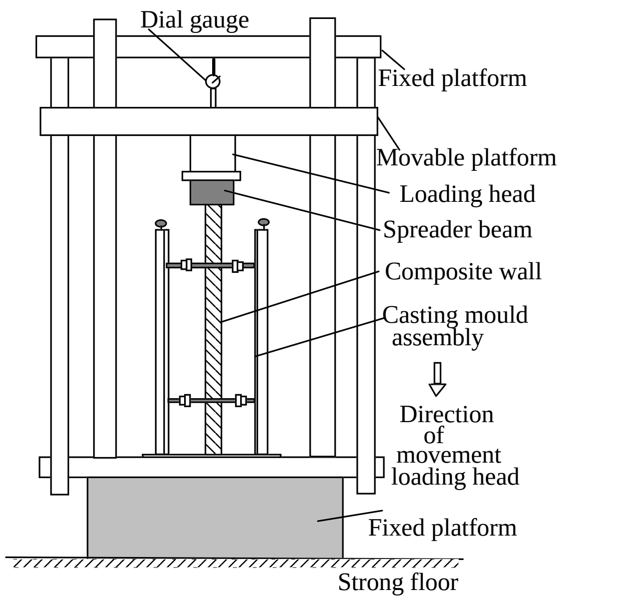

FIGURE 2(d): Schematic of experimental set-up

walls. Two Trimdek walls were fabricated without a hole and the other two with a 210 x 260 mm hole at the centre. The connection between the pair of Trimdek sheets was provided by 6 mm threaded rods with spacers and nut arrangements as shown in Fig. 2(a) for both pierced and non-pierced walls.

It was revealed from Trimdek wall tests that special connection devices at the loaded ends and around the hole should be necessary to improve the wall performnace. As a result, in Spandek walls (Fig. 2b-c), additional connection in the form of welded rod and hook assemblies (Fig. 2c) were installed at the top and bottom of the wall in addition to the threaded rod-spacers-nut arrangement. For two non-pierced walls, a welded rod and hook assembly was installed at the top and bottom of wall. This allows more effective transfer of load from the top of the wall to the bottom and mobilises the interaction between sheeting and concrete. The top and bottom of the holes in two pierced Spandek walls were also strengthened by the installation of a welded rod and hook assembly (Fig. 2b-c) so that they could act as a lintel beam to support the concrete and ensure the proper connection between the pair of sheets.

2.2 Casting, curing, instrumentation and testing of walls

After assembly of the pair of sheeting with connection devices, they were installed in a casting assembly specially fabricated for efficient casting of concrete and transportation of the wall to the testing machine. The walls were cast with machine-mixed concrete placed vertically parallel to the profile in three layers, each layer being compacted by poker vibrator and were air-cured at room temperature until testing.

Strain gauges were installed at similar key locations of both pierced and non-pierced walls. The details of strain gauges and their numberings are shown in Fig. 2(a-b). After instrumentation, the walls were transported to the compression-testing machine with casting assembly by using a crane. An schematic of the experimental setup with casting assembly [4] is shown in Fig. 2(d). Before application of axial loading, verticality of the wall was checked to avoid eccentric loading and to ensure the uniform distribution of load to the wall through the top spreader beam. The loads were applied incrementally until failure of the walls. At each load increment, axial deformation and strains were recorded using dial gauges and a manual electronic strain measuring equipment, respectively. During loading, the overall behaviour of the walls, including failure modes was observed.

3. Analysis of failure modes

3.1 Trimdek walls

These walls failed by initial crushing of concrete at the top, followed by cracking along the trough of the specimens. Fig.3 shows typical crack propagation in pierced and non-pierced Trimdek wall concrete cores. For pierced walls, diagonal cracks formed from the top corner of the hole and propagated towards the main cracks in the nearest trough. Local buckling of the profiled sheeting was observed at the top, followed by tearing and peeling at the location of the spacers. Failure of the walls was due to crushing of concrete and local buckling of sheeting at the top, with associated buckling of sheeting at the ends of the wall. However, for pierced walls, the unsupported weak concrete at the top of the hole was crushed and the overall load-carrying capacity was reduced compared with non-pierced walls.

3.2 Spandek walls

Failure of these walls was due to crushing of concrete at the top with subsequent buckling of sheeting. The crushing of concrete and buckling of sheeting were more severe in these walls than those at Trimdek walls. Extra welded rod and hook assembly enhanced and mobilised the interaction between sheeting and concrete. This assembly at the top and bottom of the hole acted as a lintel beam to support the core concrete and improved the performance.

The zone of local buckling of sheeting and crushing of concrete in these walls was extended to the bottom of the welded hooks rather than at or around the locations of the spacers as observed in Trimdek walls. Fig. 4 shows the buckling of sheeting in pierced and non-pierced Spandek walls. For non-pierced walls, local buckling formed at the top and at the middle of the wall in line with spacers. Tearing of the sheeting was also observed at spacers. At the end, the sheet separated from the concrete and buckled outward as shown in Fig. 4(a). For pierced walls, buckling and tearing of the sheeting were observed at the top of the wall and at the top corners of the hole. The buckled

(a) (b)

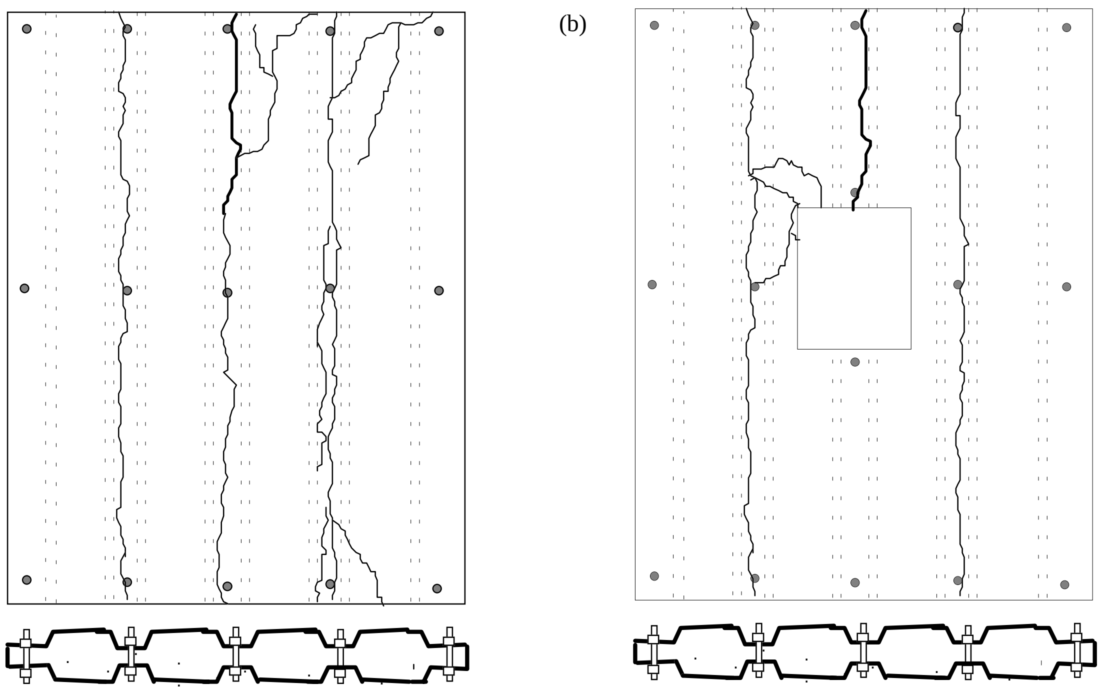

FIGURE 3: Crack propagation in Trimdek walls. (a) non-pierced wall; (b) pierced wall.

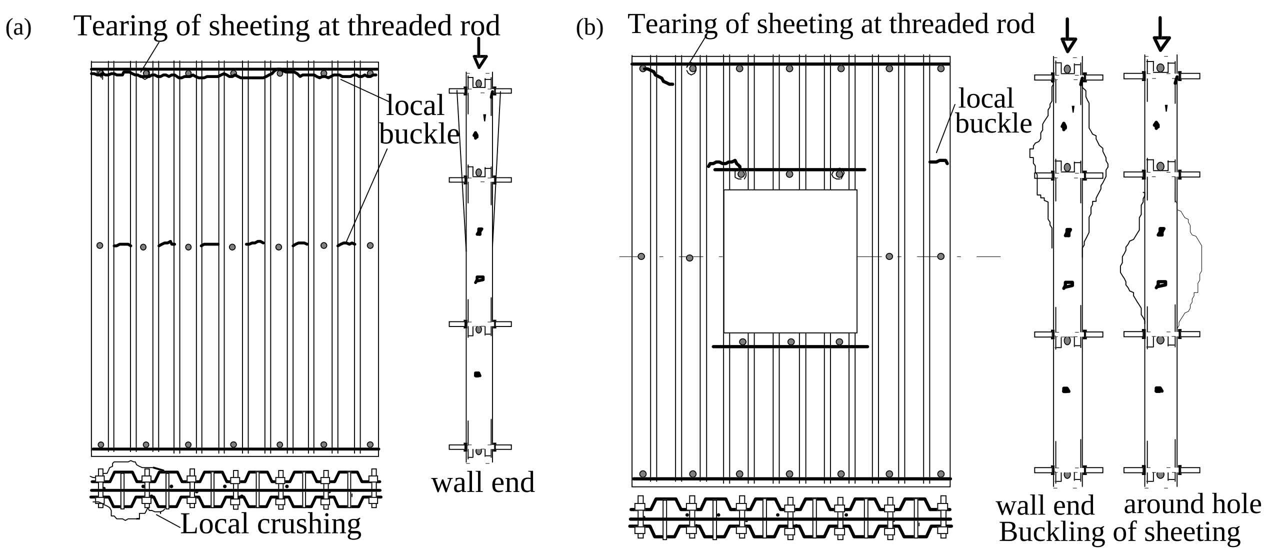

FIGURE 4: Buckling of sheeting in Spandek walls: (a) non-pierced wall; (b) pierced wall.

pattern of sheeting at the end of the wall and at holes was as shown in Fig. 4(b). For pierced walls severity of crushing of concrete around the hole was much greater than at the top of the wall.

4. Load deformation response

The load deformation responses for both pierced and non-pierced walls are shown in Fig. 5. The non-pierced walls were found to have greater axial load capacity than pierced walls. Pierced walls registered greater axial deformation than non-pierced walls. Although the axial load capacity per unit length of the Trimdek walls was greater than the Spandek walls, the latter had higher strength when compared in terms of net concrete area.

5. Strain characteristics

Fig. 6(a) shows a typical variation of axial strain along the height of the pierced Spandek wall. The compressive strains were observed near the top (gauge 1) and bottom (gauge 7) of the wall as expected, while tensile strains were observed at the top (gauge 2) and bottom (gauge 6) of the hole. Strains at gauges 2, 3 and 4 near the top of the hole showed tensile strains (Fig. 6b) compared with the predominantly compressive strains (Fig. 6c) in similar gauges (2, 3, and 4) in non-pierced walls. The existence of compressive strain at the top of both pierced and non-pierced Spandek walls was also confirmed. The strain (Fig. 6d) at the side of the hole (gauge 5) was more or less compressive for non-pierced walls, but in the case of pierced walls, it initially showed compression and then suddenly changed to tension at about 40% of the ultimate load. Similar behaviour was observed in Trimdek walls.

The strain conditions within the walls were affected by the presence of holes. The presence of holes in both steel sheet and concrete constituted a descontinuity, through which the stress could not transfer. This suggested a stress redistribution in pierced walls causing tensile strains in profiled sheets, crushing of concrete and buckling of sheeting around the holes. In all tests, lower strain was recorded at the bottom (gauge 7) of the wall than that at the loaded top (gauge 1). This indicated that full load did not transfer from the loaded end of the wall to the bottom

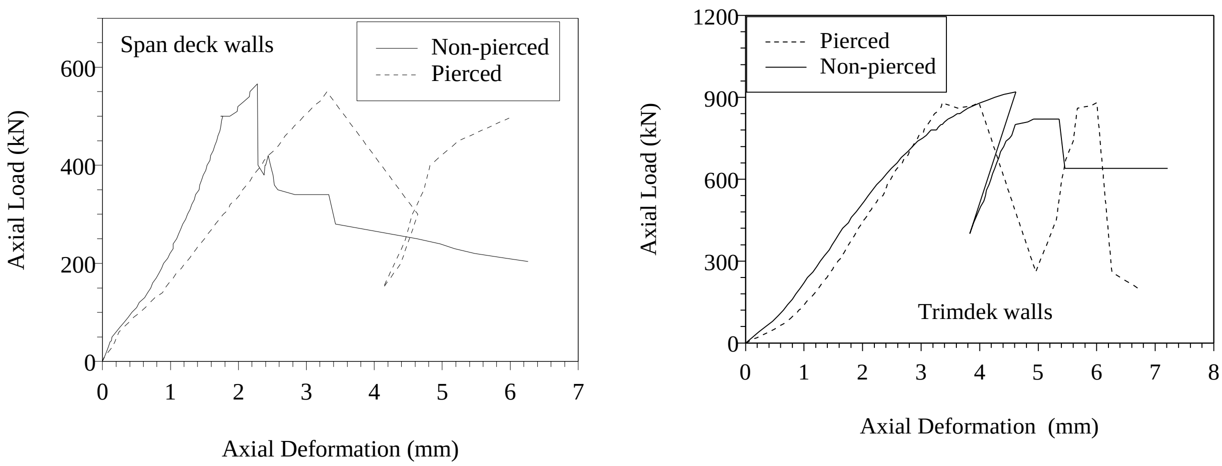

FIGURE 5: Axial load deformation response for walls:(a) Spandek walls; (b) Trimdek walls.

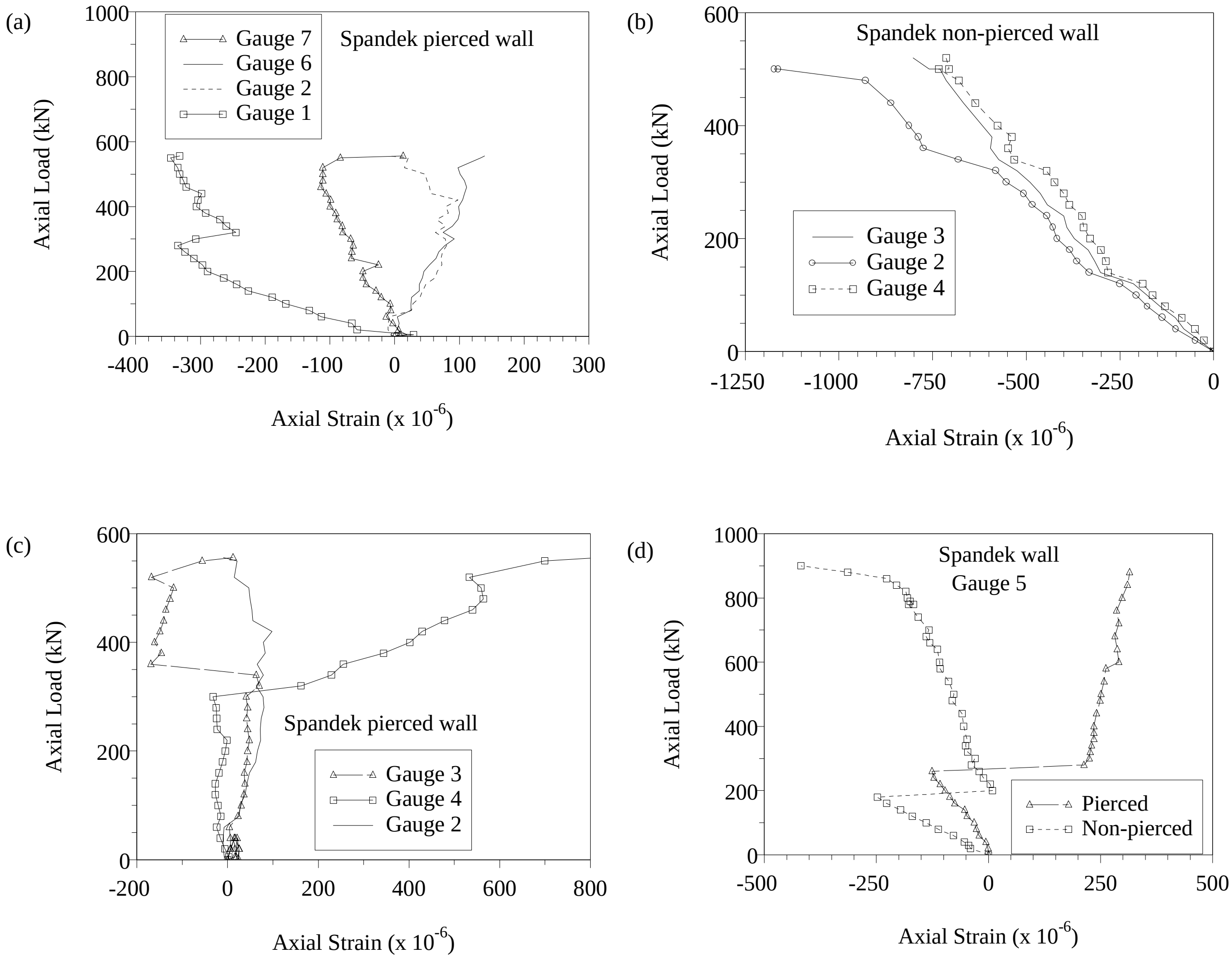

FIGURE 6: Variation of axial strains in the Spandek pierced and non-pierced walls.

and full composite action was not achieved in both pierced and non-pierced walls.

6. Comparative study and design formulations

The axial capacities of the experimental composite walls are compared with those obtained from different methods based on Codes of practice in Table 1. The axial capacity can be evaluated by using the BS 8110 [5] simplified short wall method. This method assumes that the axial capacity is the sum of the steel and concrete capacity, with a reduction of 10% for the possible additional compressive bending stresses created during slight eccentric loading or as a result of imperfections in the wall. Without material safety factors, clause 3.8.4.3 of BS 8110 can be quoted as:

$$N = 0.6 f_{cu} A_c + 0.87 f_y A_s \qquad (1)$$

where N = Axial load capacity of the wall in N,
f_{cu} = Concrete cube strength in N/mm^2,
A_s = Cross-section area of steel in the wall in mm^2,
and f_y = Yield stress of steel in N/mm^2

TABLE 1: Comparative study of axial load capacities.

Types of walls	Concrete strength MPa f_{cu}	Concrete strength MPa f'_c	Ultimate load kN: Simplified BS8110 Eq 1	Ultimate load kN: Modified BS8110 Eq 3a	Ultimate load kN: Modified AS3600 Eq 3b	Test load kN*	Ratio Test/Theory Eq 1	Ratio Test/Theory Eq 3a	Ratio Test/Theory Eq 3b
Trimdek walls: a = 0.87, b = 0.41, b/t = 95, h/w = 10.6, fy = 350 MPa									
Non-pierced	24.2	18.0	1044	549	632	920	0.88	1.68	1.45
Pierced	29.6	22.2	1222	652	754	880 (702)	0.72	1.35	1.17
Spandek walls: a = 0.68, b = 0.80, b/t = 70, h/w = 10.6, fy = 350 MPa									
Non-pierced	37.4	28.0	976	492	555	566	0.58	1.15	1.02
Pierced	40.0	30.3	1029	516	583	560 (507)	0.54	1.08	0.96

* Mean value of ultimate loads (for two identical tests for each type of wall) - the values in the brackets represent experimental loads calculated for similar concrete strengths based on different concrete strengths in pierced and non-pierced pilot walls.

The axial load capacities of all the experimental walls are shown in Table 1 and seem to be lower than those predicted by the Code-based equation 1. This is due to possible additional bending stresses at the extremities of the wall section. In the composite wall tests, it is observed on dismantling the failed specimens (Fig. 3a) that local crushing occurred at the extreme edges of one side of the walls, generally just below the load transfer device. The extreme edges of the walls do not present a solid mass of concrete, and the extra bending stress had to be carried only by the concrete in the ribs of the profile. Consequently, a profiled wall of the same overall width as a solid wall would be less able to carry bending moments caused by eccentric load application or imperfections in the wall. For the case of pierced walls, particularly top corners of the holes could be subjected to extra bending stresses causing reduction in axial capacity. This was considered reasonable, as the crushing of concrete at the top corners (Fig. 3b)of the holes was noted in the tested walls.

An empirical correction to the assumed concrete capacity can be derived assuming the reduction in load capacity to be directly proportional to the extent of void created by the profiling on the compressed edge of the wall. A reduction factor, a, should be applied [2] to the strength of the concrete in the wall and may be expressed as:

$$a = 1 - \text{profiled void on one face}/A_{cp}$$
$$= 1 - ((D*p) - A_{cp})/2A_{cp} \qquad (2a)$$

where D = Overall thickness of the wall in mm,
p = Pitch of profiles in wall in mm,
and A_{cp} = Cross-section area of concrete in one pitch of the wall in mm.

It should be noted, however, that the section area need not be calculated with extreme precision. The stiffening ribs and other details rolled into the profile generally accounts for less than 1% of the concrete area and may be ignored.

The load capacities should also be calculated taking into account the detailed geometry of each profile type. The breadth of the component steel plates were such that significant reductions in steel strength were assumed to occur due to buckling, as confirmed from the pilot tests. Purpose-designed profile geometries with larger stiffeners and smaller plates could avoid this problem. This would also true for the reduction due to profiling of the concrete cross-section. Reduction or profiling in Trimdek walls was about 21% compared to 40% in the case of Spandek walls.

Free outer flange plates at the edges of the walls were observed to buckle before the other interior plates of the profle and this was thought to precipitate collapse. Considering the significant restraint provided by the in-fill concrete, elastic buckling stress of the edge plates was calculated conservatively assuming the logintudinal edges of the pates are half-way between being simply supported and fixed. A reduction factor, b, was applied to the yield stress of the steel plate to take into account the effect of buckling of plates in contact with concrete. The factor, b, is the ratio of buckling stress to yield stress and can be related to the *b/t* (width to thickness ratio of outer flange plate) ratio of the steel plates as:

$$b = 3.9846 \times 10^{-0.01(b/t)}$$

or $$b/t = 751b^4 - 2005b^3 + 1994b^2 - 941b + 265 \qquad (2b)$$

and can also be found from Wright [2]. The equation 2(b) is derived for steel plates having a yield stress of 350 MPa. For steel plates with yield stress of f_y MPa, the value of b obtained from equation 2b, should be multiplied by the factor, $f_y/350$.

Using clause 3.8.4.3 of BS8110 and including the factors a and b, the modified formula for axial load capacity of the walls can be derived as :

$$N = 0.4f_{cu} A_c a + 0.75 f_y A_s b \qquad (3a)$$

Using clause 10.3.3 of AS 3600[6], the corresponding design formula for axial capacity of the walls using cylinder strength of concrete, f'_c, can be written as:

$$N = 0.63f'_c A_c a + 0.75 f_y A_s b \qquad (3b)$$

All the tested walls can be defined as short (height to maximum width ratio of the wall, *h/w* = 10.6), based on 3.8.1.3, and braced,based on clause 3.8.1.5 of BS 8110. Simplified BS 8110 over-predicts the capacity of the walls for the reasons already explained. Modified BS8110 and AS3600 seem to under-predict the capacity of Trimdek walls, both pierced and non-pierced. On the other hand, it seems to predict reasonably the capacity of Spandek walls. This may be for the following reasons: failure of these walls was due to extensive crushing of concrete at the loading top of the wall as well as around the holes, with significant buckling of sheeting at the end of the walls compared to Trimdek walls. The reduction factors a and b, applied to the modified BS8810 and AS3600 equations, are best suited for the Spandek walls.

The presence of holes in the walls was found to reduce the axial capacity of Trimdek walls by 23.7% compared with 10.4% in Spandek walls. The special load transfer device in the form of a welded rod and hook assembly used at the top and bottom of the hole improved Spandek wall performance and was the cause for lower percentage of strength reduction.

7. Conclusions

The experimental study of pierced composite walls validated the proposed construction method and highlighted problems not encountered with conventional reinforced concrete construction. The axial load capacity of the wall was found to be less than the fully composite failure load and was influenced by the local buckling of the component plates in the steel sheeting, profiled shape of the concrete cross-section, interface bond strength between sheeting and concrete and the presence of holes. Reduction factors to account for the first two phenomenon were discussed and used in Code based design equations. It was suggested that purpose-designed profiled steel sheeting with additional shear connection devices could take care for the later two phenomenon and composite walls could achieve equal structural efficiency with conventionally reinforced walls. Further experimental works are required and are now in progress to understand various aspects of pierced composite wall behaviour under axial and in-plane loading.

8. Acknowledgement

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