INTRODUCTION

Reinforced concrete (RC) flat slab floors lead to architecturally pleasing buildings and bridges as well as simplify and accelerate site operations. They allow easy and flexible partitioning of space and reduce the overall height of tall buildings. However, flat slab construction can lead to high shear stresses around supporting columns, which can cause abrupt punching shear failures at loads well below the slab flexural strength.

At the design stage, there are several ways of avoiding punching shear failure, such as:

1) reducing the applied loads,
2) reducing the effective length of the slab,
3) increasing the overall thickness of the slab,
4) increasing the thickness of the slab locally with a drop panel or an inverted cone,
5) increasing the column head dimensions and
6) providing some kind of shear reinforcement.

The first five solutions either increase the overall floor height, are impractical, architecturally unacceptable or expensive. Consequently, very often, to achieve an elegant thin flat slab, shear reinforcement is required. Properly designed shear reinforcement can prevent brittle punching failure and increase the strength and ductility of the slab-column connection.

Traditionally, to prevent shear failure, reinforcement is provided either at an angle or perpendicular to the main flexural reinforcement, as shown in figure 1. However, in thin structural elements, such as slabs, anchoring short lengths of perpendicular reinforcement is difficult, because full anchorage should be provided to develop the shear reinforcement yield strength within the depth of the slab. This problem is further aggravated by the fact that conventional shear reinforcement, due to its cross-sectional dimensions, cannot be placed in the cover region above the top layer of flexural reinforcement without reducing durability or the efficiency of flexural reinforcement.

Stirrup shear reinforcement was first investigated by Graf (1938) and later by Elstner and Hognestad (1956), Kinnunen and Nylander (1960), Franz (1963), Wantur (1969), Carpenter, et al (1970) and others. Much of the experimental work reported until now led to the conclusion that shear reinforcement consisting of bars is not fully effective, because it does not reach its yield strength before slab failure. Broms (1990) tested eight specimens with different types of shear reinforcement from ordinary stirrup and bent bars. The test results showed that ordinary shear reinforcement in the form of open links (see figure 1) enclosing only the tension flexural reinforcement were not effective enough to give flat slabs the desired ductility. The authors (Li and Pilakoutas, 1995) tested one specimen with similar U-shaped shear reinforcement from ordinary stirrup reinforcement. The results confirmed that this kind of shear reinforcement is not fully effective, because it does not develop high enough strain to reach yield.

Closed hoops, as shown in figure 1, are best suited for shear reinforcement; however, their integration involves building up reinforcement cages from individual bars. This increases the costs and time required for fixing reinforcement. In the UK, the conventional solution is "sausage" links (shown in figure 1) since they are relatively easy to install. However, they are labor intensive and there are to doubts regarding their effectiveness. Inclined reinforcement, though effective, is difficult to design, manufacture and place. It is also not suited for earthquake resistant design, where a reversal of moment may occur.

Cages of prefabricated reinforcement in all forms and shapes are available commercially and depend on factory bending and welding operations. The solutions tend to be more economical and practical than conventional reinforcement. In the UK, one such prefabricated stirrup system, developed by the BCA (1990), is the Shearhoop System.

Other solutions

Due to the difficulties with conventional reinforcement, there have been many attempts to develop easy to place and effective shear reinforcement for flat slabs. Hawkins (1974) summarized most of the different types of shear reinforcement. All current types of shear reinforcement have advantages and disadvantages concerning ease of detailing, anchorage effectiveness, cost effectiveness and ease of placement. However, many of the existing systems increase not only the shear capacity of the connection, but also the flexural capacity. Increasing the flexural capacity may not be undesirable in most construction away from earthquake regions, but it may mean that if failure does occur it could be of a brittle nature. Two types of non-conventional reinforcement are worth mentioning.

Steel section shearhead reinforcement. Shearhead reinforcement includes the use of I Sections, Channel sections, D-collars or Steel Plates. Shearhead cages were also used by Moe (1961) and Anderson (1963). Shearheads consisting of structural steel plates or structural steel sections, or both, were investigated by Tasker and Wyatt (1963) and Corley and Hawkins (1968).

Shearheads provide an effective, but cumbersome way of reinforcing against shear. Steel sections tend to be heavy, expensive relative to conventional reinforcement and normally require welding at the intersection right above the column. They can be difficult to integrate with conventional reinforcement and may obstruct passage of the column bars through the connection. Due to their high stiffness, they attract extra moment to the connection, which can lead to problems at the ends of the steel sections.

Stud type reinforcement. This is an efficient solution used in North America and Germany and involves the use of shear studs welded on a metal strip (as shown in figure 2). Extensive tests in Germany (Andrä, Dilger and Ghali, (1979), Andrä, (1981)) and Canada (Dilger, Walter, and Ghali (1981), Mokhtar, Ghali, and Dilger, (1985), Elgabry and Ghali (1987)) on full-size slab-column connections verified that stud type reinforcement substantially increases the strength and ductility of slabs.

SHEARBAND REINFORCEMENT

Research undertaken at the “Centre for Cement and Concrete” of the University of Sheffield, in the field of punching shear in flat slabs (Pilakoutas and Li, 1995), has led to a new concept in shear reinforcement, called the "Shearband" system. The Shearband system is now patented in many regions internationally, including North America, Europe and Japan.

This new shear reinforcement system is made of steel strip of high ductility. The strip is punched with holes, because this has been demonstrated experimentally to increase its anchoring characteristics over short lengths (Li and Pilakoutas, 1995). The strip can be bent to a variety of shapes, and for the initial tests the strip was punched and bent, as shown in figure 3.

This new shear reinforcement system differs from all existing systems. Due to its small thickness the reinforcement can be placed from the top, after all flexural reinforcement is in place, with minimal loss of cover. This leads to major benefits because the shear reinforcement:

1) anchors above the outermost layer of reinforcement,
2) acts over the entire concrete core, maximizing its effectiveness in resisting shear,
3) does not decrease the effective depth of flexural reinforcement,
4) is very simple and efficient to place and
5) does not increase the slab flexural capacity.

This system is adaptable and can accommodate greater tolerances in placement and enables quick addition of extra reinforcement where required at a later stage. In addition:

1) it can enable the construction of thinner slabs (below what is currently acceptable by the codes),
2) it can be designed by using existing design codes,
3) it can be used in addition to other systems.

On the practical side, since it is light it is easy to store and transport.

EXPERIMENTAL VALIDATION OF SHEARBAND SYSTEM

The experimental validation of the system was undertaken through several series of flat slab testing, beam testing and pullout tests. This paper will discuss and present the results from the first series of flat slab testing.

Slab details

Four slabs were designed and tested in the heavy structures laboratory of the University of Sheffield. The slabs were given codes PSS- (Punching Shear Slabs) A to D. PSS-A was a control specimen without shear reinforcement. Slabs PSS-B, PSS-C and PSS-D were reinforced with different forms of the alternative shear reinforcement system, as will be discussed later.

The dimensions of the slabs were chosen to be similar to those in tests undertaken previously by Li and Pilakoutas (1995) at Sheffield and Chana at BCA (1990). The four slabs were 175 mm thick and 2000 mm square with a 200 x 200 x 200 mm column stub located centrally below the plate. The slabs were reinforced with bottom and top flexural reinforcement mats, as shown in figure 4.

The slab dimensions represent 1:1.5-scaled models of full-scale slabs. As a result, the depth of the slab was below the minimum (200 mm) allowed by the British Standard BS 8110 (1985). However, in the opinion of the authors, the reason design standards limit the slab depth is the inability of conventional shear reinforcement to anchor effectively over short lengths. This implies that such a test is more severe than for deeper slabs and if punching shear failure is avoided in this slab, then the results are applicable for all slabs deeper than 175 mm.

Shear reinforcement

For comparison and control purposes, PSS-A was left without shear reinforcement. Slabs PSS-B, PSS-C and PSS-D were reinforced with the new shear reinforcement system. The new shear reinforcement was made of a high strength, high ductility flat steel strip of width 25.4 mm and thickness 0.8 mm.

For PSS-B, the strip was punched with 8 mm diameter circular holes along the central line of the strip and with semi-circular holes at both edges. Preliminary tests by the authors (Li, 1995) showed that the holes in the strip are necessary to maintain good anchorage characteristics in the strip over short embedment lengths. The strip was bent in the inclined form shown in figure 3a. This was done, because it was felt that shear reinforcement should be placed perpendicularly to the potential shear crack. The effectiveness of inclined compared to vertical reinforcement was one of the parameters studied in this series of testing, as well as in subsequent series.

The amount of shear reinforcement required was determined in accordance with the requirements of BS 8110 (1985) to avoid punching shear failure with the distribution shown in figure 5. The minimum spacing of shear reinforcement was one of the main parameters governing this design. Rather than distributing the reinforcement evenly over the critical zone, which was thought to be excessive, the American practice of providing reinforcement in the two principal directions was adopted. Figure 6 shows a photograph of the slab reinforcement.

The shear reinforcement for PSS-C was similar to that of PSS-B, as shown in figure 5, but without any holes in the strip. The only difference with slab PSS-B was that each length of Shearband reinforcement in PSS-C had two additional legs, one at each end. Extra legs of shear reinforcement were demonstrated in earlier pullout tests to be sufficient for anchorage purposes. Clearly, this test aimed at examining the performance of the shear reinforcement without holes.

The shear reinforcement for slab PSS-D was made by using the steel strip with holes, but this time it was bent with vertical legs as shown in figure 7. The shear reinforcement was distributed more evenly in the required perimeters. Because the length of reinforcement spanned only in one direction, only one depth of shear legs was required. This is thought to simplify matters on site, by eliminating the possibility of placement errors. It is also more efficient, because it anchors on the top layer of reinforcement.

The shear reinforcement in PSS-B and PSS-C was placed above the two top layers of flexural reinforcement only. However, the reinforcement for PSS-D, though placed from the top, was also bent manually during fixing around the bottom flexural reinforcement, as shown in figure 7. This was done so as to provide additional anchorage to the reinforcement in the compression zone.

The shear reinforcement details are shown in table 1. The characteristics of the steel strip and flexural reinforcement are given in table 2.

Concrete

Normal weight ready-mix concrete of nominal strength C35 (35 MPa) was used. The specification for the concrete was: nominal maximum aggregate size 20 mm; slump 50 mm. Cubes, cylinders and prisms of concrete were taken for control purposes. The four slabs were cast at the same time in specially prepared wooden forms, from the same batch of concrete to eliminate variations in concrete quality. Concrete was compacted by poker-vibrators. After casting, the slabs were cured for one week. For control purposes, sets of four cubes, split cylinders and prismatic specimens were cast along side each of the slabs. The results from control specimens tested on the same day as the slabs are summarized in table 3.

Instrumentation

Extensive measurements were made of strains at key locations on the flexural and shear reinforcement. Electrical transducers were also used to measure the vertical and horizontal deflections, as well as concrete strains. The width of cracking was not measured during the test, but the development of the cracks was marked on the top surface.

The positions of strain gauges on the flexural reinforcement is shown in figure 8, whilst the strain gauges on the vertical legs of the shear reinforcement are shown in figures 5 and 7.

The positions of the LVDT transducers are shown in figure 8. Transducers D1 to D11 were positioned to measure the vertical displacement of the slab. Transducer D1 was intended to measure the column movement. Transducers RS1 and TS1, RS2 and TS2 were positioned perpendicular to each other to measure the concrete strains in the radial and tangential directions at the column edge and on the bottom soffit of the slab. Transducers RS3 to RS6 were placed in pairs on the top and bottom surfaces of the slab to measure the radial strains on the concrete.

Testing Arrangement

Traditionally, in punching shear tests, the slabs are supported on their four edges, or through tie rods, and loaded through the column stub in the center. This arrangement imposes the support deformation of the test specimens. However, real slabs are not truly symmetric about the diagonal. The stiffness of slabs is different in the two orthogonal directions, due to the different lever arm offered by the two layers of top reinforcement and, hence, the moment of resistance will differ according to the boundary conditions. Furthermore, specimens supported along four edges are expected to lift up at the four corners and this further distorts the boundary.

The testing arrangement was similar to that used in previous tests by the authors (Li and Pilakoutas, 1995). Symmetric point loads were applied at eight locations on a circle of diameter 1.7 meters, as shown diagrammatically in figure 9 and in the photograph of figure 10. These points correspond roughly to the points of contraflexure of a 4 meter span, or a 6 meter prototype span. Eight hydraulic jacks of 100 kN capacity were used and connected to the same pressured oil supply so that each jack applied the same load. The jacks were restrained against horizontal movement, but the slab at the loading points was free to rotate and displace laterally.

The test-rig developed does not define the deformation of the specimens either in the horizontal or in the vertical direction. Each loading point can deflect independently, but the load at each loading point is kept the same. This provides the best simulation of uniform loading conditions.

Experimental Procedure

The applied load was manually controlled and increased incrementally. The service load (assuming an average load factor of 1.6) and design capacity were calculated by flexural design using the equations of BS 8110 (1985) (yield capacity of flexural bars 460 MPa). The loading procedure followed consisted of several load cycles as follows:

Phase A: Two cycles were applied up to the service load, 150 kN.

Phase B: Two cycles were applied up to the design capacity 250 kN.

Phase C: One cycle was applied up to 450 kN, just below the expected punching shear capacity.

Phase D: The slab was loaded to failure.

TEST RESULTS

Test observations

The initial crack development in all four slabs followed a similar pattern. The first cracks opened up on the top surface in the form of flexural cracks near the column, in the weaker direction, at around 100 kN. The weaker direction was the one with the smaller effective depth. After 100 kN, the cracks propagated from the middle outwards and reached the slab edges. Subsequently, with increasing loads, more cracks developed that advanced radially along the four axes of symmetry of the slab from the column faces towards the slab edges. Cracks parallel to the X and Y axes opened up at a load less than 200 kN, while cracks parallel to the diagonal axes opened up at a load greater than 200 kN. By the time the applied load reached 300 kN, only a few new diagonal cracks developed. After that, with increasing load, the width of the cracks close to the column increased substantially.

PSS-A, which served as the control specimen, failed abruptly in punching shear at a load of 454 kN. The crack pattern through the section at failure can be seen in figure 11. The zone of concrete punched out was similar to the expected truncated cone and surfaced at the top of the slab through inclined cracks. Penetration of the column into the slab was observed and at the end of the test, as the column was found to have punched into the slab by about 23 mm. The widths of the flexural cracks near the column were considerable, exceeding 1mm. No concrete compressive crushing was observed near the column.

PSS-B and PSS-C failed gradually after reaching the maximum flexural load and undergoing substantial deflections. Penetration of the column into the slab was also noticed after the end of the tests. However, no clear evidence of the punching shear cracks was found on the top surface of the slabs, as in PSS-A. The width of the flexural cracks directly above the column was considerable, greater than 5 mm in several locations. The maximum recorded load for these two slabs was 560 kN. During testing, it was observed that the flexural cracks on the top surface followed the expected hogging yield lines, indicating that the slabs reached their full flexural potential before failure. This conclusion is also supported by strain readings (from high elongation strain gauges) on the flexural reinforcement in excess of 20,000 $\mu\varepsilon$. The failure surfaces for PSS-B and PSS-C are illustrated in figure 11 by sections through the slab.

PSS-D failed eventually in punching shear failure, after reaching the maximum flexural load and undergoing substantial deflections. However, in this case the column did not penetrate the slab, but the punching shear failure resulted due to failure outside the shear reinforced area around the four loading jacks located on the diagonals. The maximum recorded load for PSS-D was also 560 kN. The failure pattern through a cross section of PSS-D is shown in figure 11. The failure of this slab highlights the huge potential of strip reinforcement in preventing failure in the desired zone.

Instrumental results

Full details of the test results are presented elsewhere (Pilakoutas and Li, 1995, Li 1997). The load history versus deflection in position D11 is shown in figure 12. The load history versus strain on the flexural reinforcement is shown in figure 13 for locations S1 and S7. The load history versus microstrain for the shear reinforcement is shown in figure 14 for locations S3, S4, S5 and S6.

DISCUSSION OF TEST RESULTS

Deflections and Ultimate Punching Shear Capacity

Figure 12 indicates a significant increase in the ductility of PSS-B, PSS-C and PSS-D reinforced with shearband reinforcement, as compared to control slab PSS-A. The deflections in slabs with shearband reinforcement are more than two times the deflection of the control slab, as shown in Table 4. In addition, because all three slabs reinforced in shear reached their full flexural capacity, a 23 % load enhancement was achieved.

Strain on flexural reinforcement

As shown by the large plastic strains in strain gauges S1 (higher top bar) and S7 (lower top bar) of figure 13, in slabs with shear reinforcement, the flexural reinforcement close to the column yielded significantly before failure. This indicates that plastic hinge regions, formed close to the column face, enabling moment redistribution within the slab, leading to yield lines radiating from the center outwards in between the loading points. Much larger strains were recorded by S1 than by S7 and S1 appears to have yielded first.

Effectiveness of Shear Reinforcement in PSS-B and PSS-C

As shown in figure 14, prior to the initiation of cracking, the strains in the shearband reinforcement, with or without holes, are relatively small. After that, however, strain on the shear reinforcement with holes is seen to be smaller than strain on reinforcement without holes. This can be attributed to the better anchorage of the reinforcement with holes, which arrests the cracking from opening over a shorter length.

The internal inclined shear cracks in these two slabs are shown in figure 11. More shear cracks can be noticed in these slabs than for PSS-A, for which there is evidence of only a single crack that defines the failure surface. This means that the shear reinforcement was effective in stopping failure along this first potential surface of weakness. In fact, the figure shows that the reinforcement was also effective at least along a second surface of weakness, as indicated by the large shear cracks further out. It is interesting to note that none of these shear cracks reached the top surface of the slabs, which explains why spalling of concrete has not been observed on the top surface at failure.

The first set of inclined shear cracks appears to have started from near the top reinforcement and ended near the column. Even though at the end of testing, it was noticed that the column penetrated into the slab, the first two legs of shear reinforcement were actually avoided by the first inclined shear cracks, which propagated inside the cover. These cracks were more inclined away from the column and appeared to span at least two legs of shear reinforcement. It can be concluded that substantial yielding and rigid body rotations must be responsible for the apparent penetration of the column into the slab, which explains why the load was maintained during this process. Because there was shear failure surface through the concrete in compression, the shear resistance of concrete along that surface can be considered to have been adversely affected by shear cracking. This assumption appears to be supported by the ACI code of practice, which adopts a reduced concrete shear contribution when the slab requires shear reinforcement (V_c in equation 11-37 is twice as much as allowed in 11.12.3.1), but not by the British Standards which assume the same concrete shear resistance before and after the initiation of shear cracks.

Load versus strain relationship of shear reinforcement in slab PSS-D

The load versus strain relationship of shear reinforcement in PSS-D is shown in figure 15. Strains in S2 are much smaller than in S6 and S8 and show no yielding took place at that location. Strains S6 and S8 show massive strains indicating that the shearband was well anchored and fully utilized. This can give an explanation why the inclined failure surface of slab PSS-D is different from that of the previous two slabs, as seen in figure 11.

The inclined failure crack of PSS-D differed from that of PSS-B and PSS-C, because more shear reinforcement was spread all round the column and enclosed both the top and bottom flexural reinforcement. Hence, the slab was more rigid in the punching zone of the column and, hence, eventual failure was obtained just at the edges of the shear reinforced zone. The better performance near the column indicates that the European approach of distributing reinforcement over a larger area may have some merit.

Comparisons between arrangements of shear reinforcement

The amount of shear reinforcement used was almost the same in all three slabs reinforced in shear, as shown in table 1. Nevertheless, due to the fact that failure in PSS-D was outside the reinforced zone, it appears that the vertical shear reinforcement in slab PSS-D performed better than the shear reinforcement in slabs PSS-B and PSS-C. This cannot be concluded clearly, because the reinforcement was better distributed and also anchored under the bottom reinforcement. However, as there are practical advantages in having the shear reinforcement vertical, this was a subject of further investigation during a second phase of testing.

It is clear that because all slabs with shear reinforcement reached the same maximum load of 560 kN, the ultimate capacity of these slabs was determined by their flexural capacity.

Effectiveness of shear reinforcement

The shear reinforcement has little effect before the initiation of the inclined shear cracks inside the slab, as shown by the load versus strain curves for shear reinforcement. After the development of the inclined shear cracks, shear reinforcement transfers much of the shear force across the shear crack and delays the further widening of the shear crack, thus increasing the punching shear capacity (in the case of PSSB, C and D beyond the flexural capacity) and ductility of the slab. To achieve this, the reinforcement needs to be well anchored and have enough ductility to allow the mobilization of many legs of reinforcement. For this task, the shear reinforcement with holes was considered to be more efficient and it was thus adopted during the subsequent stages of development testing.

Comparisons of flexural Capacity with BS 8110

The flexural capacity of the slabs, calculated using BS 8110 (1985) and assuming a yield-line pattern radiating outwards form the center towards the middle of loading points is given below:

- the serviceability load was found to be around 270 kN (load factors of 1 used),
- the design flexural capacity was found to be 418.3 kN (using material safety factors),
- the expected (unfactored) flexural capacity was found to be 498 kN (without using material safety factors).

The predicted upper bound flexural load for the slabs by using the true material characteristics (steel strength as obtained from tests) and a section analysis computer programme (Pilakoutas, 1990) is 571 kN. This was very close to the experimentally obtained value of 560 kN, which shows that the three shear reinforced slabs reached their flexural capacity. Similar results are expected when using the American standards. It should be pointed out that the actual steel mechanical characteristics include strain hardening and the measured strength is higher than the characteristic yield strength by up to 20%.

Comparisons of Shear Capacity with BS 8110 and ACI 318-99

The failure loads are summarized in Tables 5 and 6, where they are compared with characteristic strengths predicted by BS 8110 (1985) and ACI 318-99 (1999), respectively. The partial safety factors or strength reduction factors are set equal to 1.0 and the mean compressive strength of the concrete obtained from the control specimens is used in the calculations. The limitation of 40 MPa in the BS 8110 equation has been ignored, to avoid making the code results appear even more conservative.

Two types of punching shear failure are considered for predicting the failure loads.

Type 1 Punching shear failure through the links in the first failure zone

For BS 8110, this is calculated as the concrete shear resistance at 1.5 d plus the contribution of all links within the first failure zone, i.e. 1.5 d from the column face. For ACI 318-99, this is calculated as the concrete shear resistance at 0.5 d plus the contribution of all links within the first failure zone, i.e. 0.5 d from the column face.

Type 2 Punching shear failure outside the shear reinforcement region

For the BS 8110 calculations for PSS-B and PSS-C this relates to the concrete shear resistance at (3.75 + 0.75) d from the column face. For slab PSS-D this is calculated as the concrete shear resistance at (1.45 + 0.75) d plus the contribution of all shear links within this failure zone. For ACI 318-99, calculations for PSS-B and PSS-C this is calculated as the concrete shear resistance at (3.75 + 0.5) d from the column face; for slab PSS-D this is calculated as the concrete shear resistance at (1.45 + 0.5) d plus the contribution of all shear links within this failure zone. For PSS-D, the perimeter was chosen to reflect the observed mode of failure.

From table 5, it can be seen that the punching shear capacity predicted for the control slab by the British Standard is well below the experimental value. In addition, Slab B, which failed in flexure, is predicted to fail in shear Type 1. Slab D was not expected to have any shear problems due to the large amount of shear reinforcement present. Hence, it can be concluded that the British Standard is in general conservative (though less conservative than the ACI code) and that the shear reinforcement behaved at least as well as expected.

From table 6, it can be seen that the ACI code is very conservative, predicting shear failures for all slabs. It underestimates both the concrete shear contribution and the additional resistance provided by the reinforcement. In fact, all the shear reinforced slabs exceed even the upper limit of the maximum capacity expected from a section with the maximum amount possible of shear reinforcement. Again, by using the ACI approach it can be concluded that the shear reinforcement behaved extremely well.

DESIGN RECOMMENDATIONS

It is clear from the above that the design codes examined are very conservative and underestimate the contribution of concrete and shearband reinforcement. Hence, for the design of slabs with the current shear reinforcement system, no extra or specific design rules are required. However, it is recommended limiting the characteristic yield capacity of the strip to 460 MPa or 500 MPa, depending on the maximum stress allowed for reinforcement in each code of practice. This limitation will ensure that the width of shear cracks is controlled during design loads to the same strains as for conventional reinforcement.

CONCLUSIONS

The present shear reinforcement system provides an effective and simple system against brittle punching shear failure. The system leads to improved structural performance, when compared with other shear reinforcement systems, because it:

- prevents brittle punching shear failure and improves greatly the ductility of flat slabs,
- increases the ultimate design capacity,
- allows yielding of the flexural reinforcement and moment redistribution,
- can be used in thin slabs,
- can be designed using existing design codes.

Both BS8110 and ACI318 codes of practice appear to be conservative in predicting the resistance of unreinforced concrete slabs and both underestimate the shear reinforcement contribution.

The ACI318 code is more conservative when it comes to calculating the resistance of shear reinforced slabs, because it reduces to half the contribution of the concrete shear resistance. This practice is not adopted in Europe, where the punching shear reinforcement is distributed in all the punching shear zone.

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Table 1 Shear reinforcement details

Slab	Links within 1.5d	Links within 2.5d	Links Total	Total effective area of links
PSS-A	0	0	None	0
PSS-B	32	56	80	1114
PSS-C	32	56	88	1788
PSS-D	64	84	94	1609

Table 2 Reinforcement Characteristics

Type of reinforcement	Cross section mm	Yield Stress MPa	Strength MPa
High Tensile Strip	25.4 x 0.8	1100	1200
Deformed Type II	12 ϕ	490	600
Deformed Type II	16 ϕ	500	600

Table 3 Concrete Cube, Cylinder and Prism Strength

Slab	Cube Strength MPa	Prism Bending MPa	Split Cylinder MPa
PSS-A	32.3	3.42	2.71
PSS-B	39.0	4.19	3.36
PSS-C	41.2	4.46	3.66
PSS-D	42.2	4.23	3.10

Table 4 Maximum Load and Deflection

Slab	PSS-A	PSS-B	PSS-C	PSS-D
Max. Load (kN)	454	560	560	560
Max. deflection(mm)	11.4	25.4	24.1	21.4

Table 5 Comparisons of Failure Loads and Modes of Test Results Using BS8110 (1985)

Slab	TYPE 1 First Perimeter		TYPE 2 Outside Region					$\frac{V_{test}}{V_{calc}}$	Mode of failure	
	$V_k^{(1)}$	$V_{max}^{(2)}$	$V_{ck}^{(3)}$	$V_{ck}^{(4)}$	$V_{max}^{(5)}$	V_{test}	$V_{calc}^{(6)}$		Predicted	Actual
PSSA	<u>345</u>	-	-	-	632	454	345	1.32	Punching	
PSSB	<u>557</u>	738	967	749	694	560	557	1.00	Type1	Flexural
PSSC	649	745	1099	843	695	560	586	0.96	Flexural	Flexural
PSSD	872	755	867	680	695	560	586	0.96	Flexural	Flexural

(1) $V_k = A_{sv} f_{yv} + v_{ck}$ Ud, $v_{ck} = 0.79 (100A_s/bd)^{1/3} (400/d_{eff})^{1/4} (f_{cu}/25)^{1/3}$, U=4c + 12d
(2) $V_{max} = 2V_c$ considering a similar slab without shear reinforcement at critical section
(3) $V_{ck} = v_{ck}Ud$, using square perimeter 1.5d from the last layer
(4) $V_{ck} = V_{ck}^{(3)}$ but using chamfered perimeter, plus 1.5d for slabs PSSB to PSSE
(5) Maximum shear capacity at column face, shear stress less than 5 x 1.25=6.25 MPa or $\sqrt{f_{cu}}$
(6) V_{calc} is the lesser of $V_k^{(1)}$, $V_{max}^{(2)}$, $V_{ck}^{(3)}$, $V_{ck}^{(4)}$, $V_{max}^{(5)}$ or V_{flex}, where, V_{flex} is the expected flexural capacity
All loads in kN

Table 6 Comparisons of Failure Loads and Modes of Test Results with ACI318-99

Slab	TYPE 1 First Perimeter			TYPE 2 Outside			$\frac{V_{test}}{V_{calc}}$	MODE OF FAILURE	
	$V_n^{(1)}$	$V_{max}^{(2)}$	$V_n^{(3)}$	$V_c^{(4)}$	$V_{calc}^{(5)}$	V_{test}		Predicted	Actual
PSSA	-	-	318	-	318	454	1.43	Punching	Punching
PSSB	289	526	351	1058	351	560	1.60	Type 1	Flexural
PSSC	346	540	360	1235	360	560	1.55	Type 1	Flexural
PSSD	398	546	364	946	398	560	1.41	Type 1	Flexural

(1) $V_n = V_c + V_s$, $V_c = \sqrt{f_c} b_o d / 6$, $b_o = 4(c+d)$ (All loads in kN)

(2) $V_{\max} = \sqrt{f_c} b_o d / 2$ (*d* is 139 mm)

(3) $V_n = V_c$, $V_c = \sqrt{f_c} b_o d / 3$, $b_o = 4(c+d)$ (considering similar slab without shear reinforcement)

(4) $V_c = \sqrt{f_c} b_o d / 3$, (considering perimeter chamfered d/2 from the outermost shear reinforcement)

(5) $V_{calc} \leq \left[V_n^{(1)} or V_{\max}^{(2)} or V_c^{(4)} \right]$, but $\geq V_n^{(3)}$

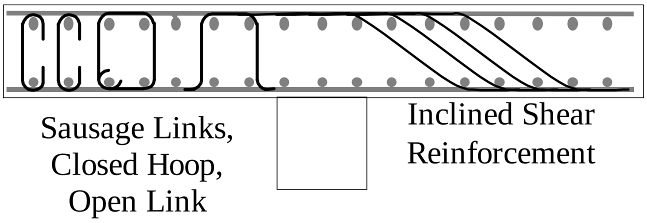


Fig. 1 Typical types of conventional shear reinforcement

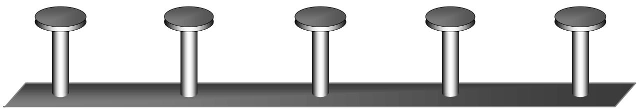
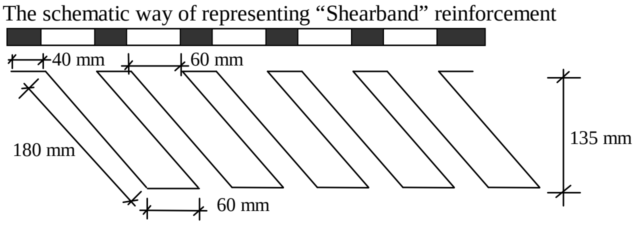


Fig. 2 Stud Rail shear reinforcement

The schematic way of representing “Shearband” reinforcement



a) The inclined form of shear reinforcement

b) Flat steel strip punched with holes

Fig. 3 Inclined Shearband reinforcement

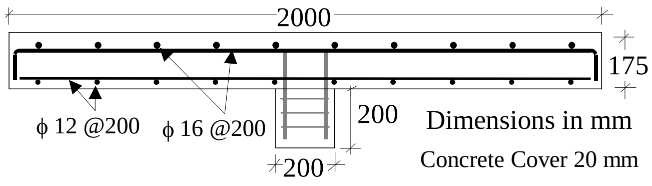


Fig. 4 Detail of flexural reinforcement and column reinforcement

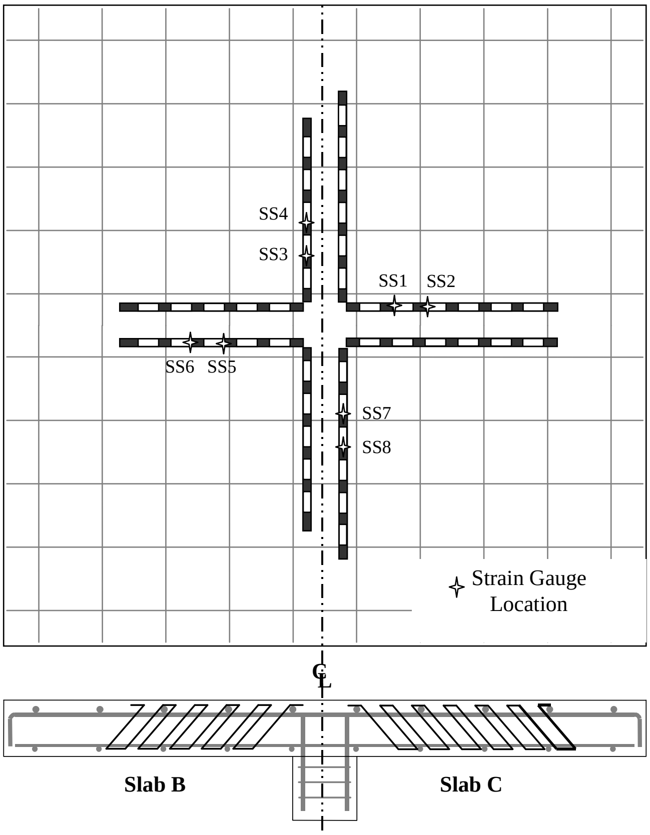


Fig.5 Shear reinforcement of slabs PSSB (left) and PSSC (right)

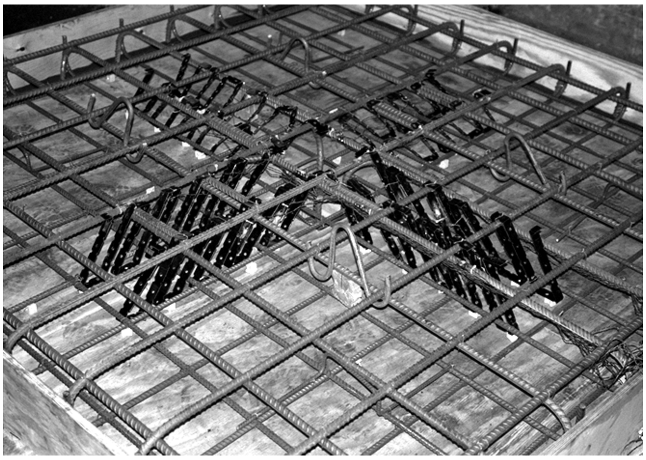


Fig. 6 Reinforcement in slab PSS-B

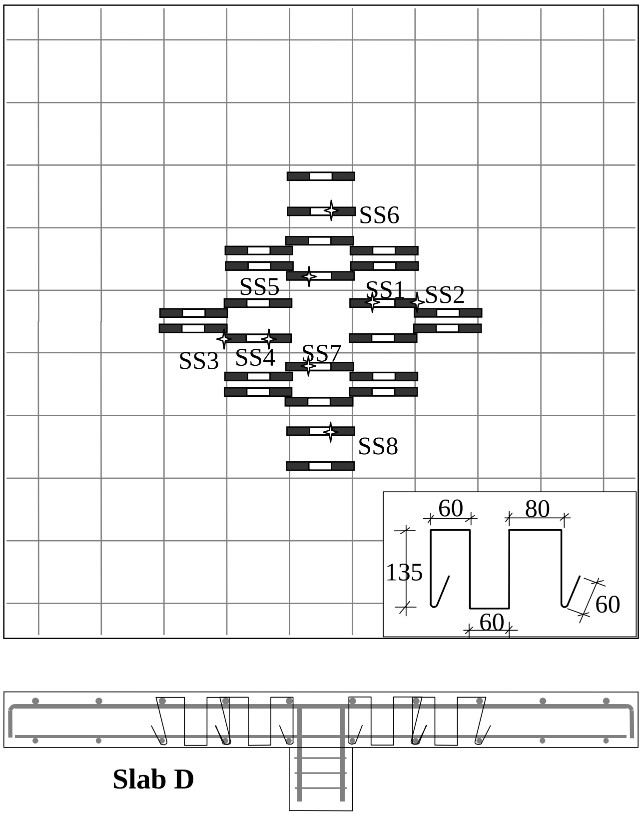


Fig.7 Shear Reinforcement in Slab PSSD

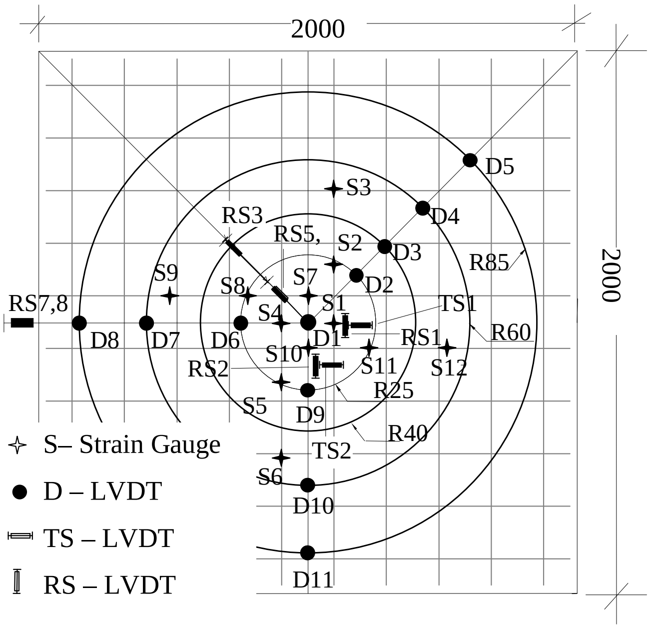


Fig. 8 Positions of instrumentation

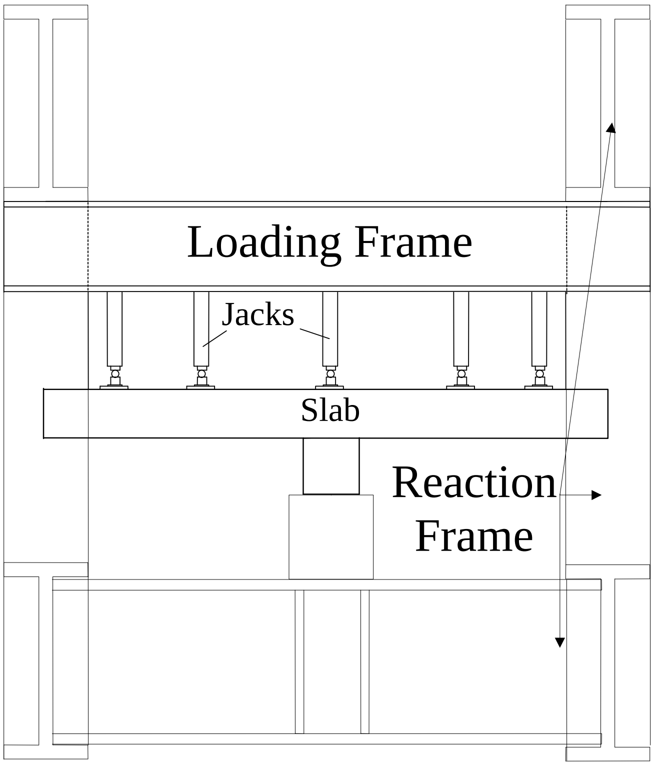


Fig. 9 Schematic representation of test set-up

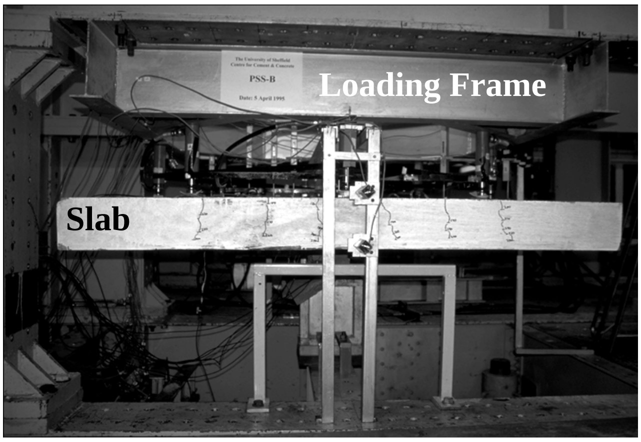


Fig. 10 Slab PSS-B under load

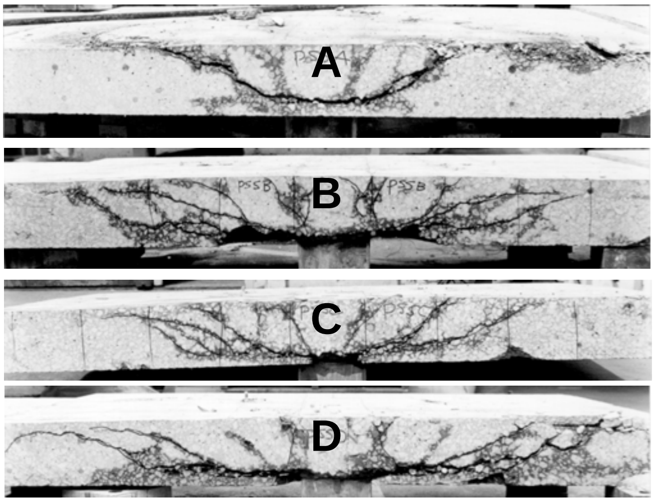


Fig.11 Section through the slabs at failure

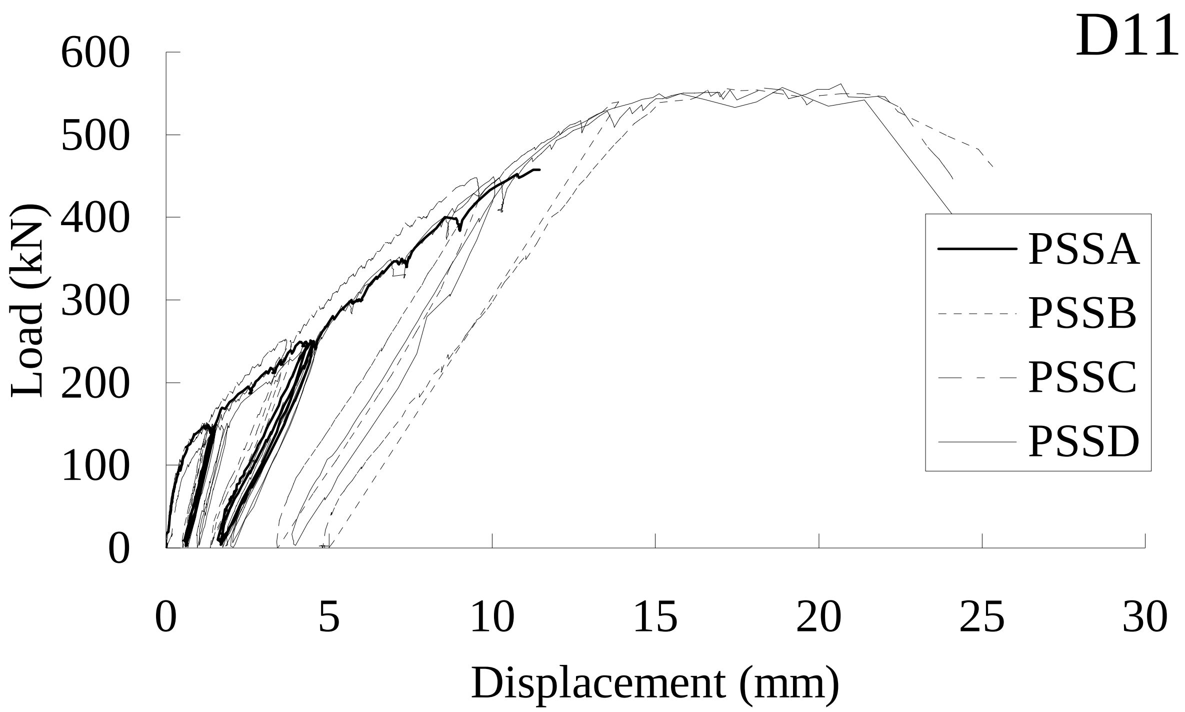


Fig. 12 Load-displacement relationship for locations D10 and D11

(Actual size)

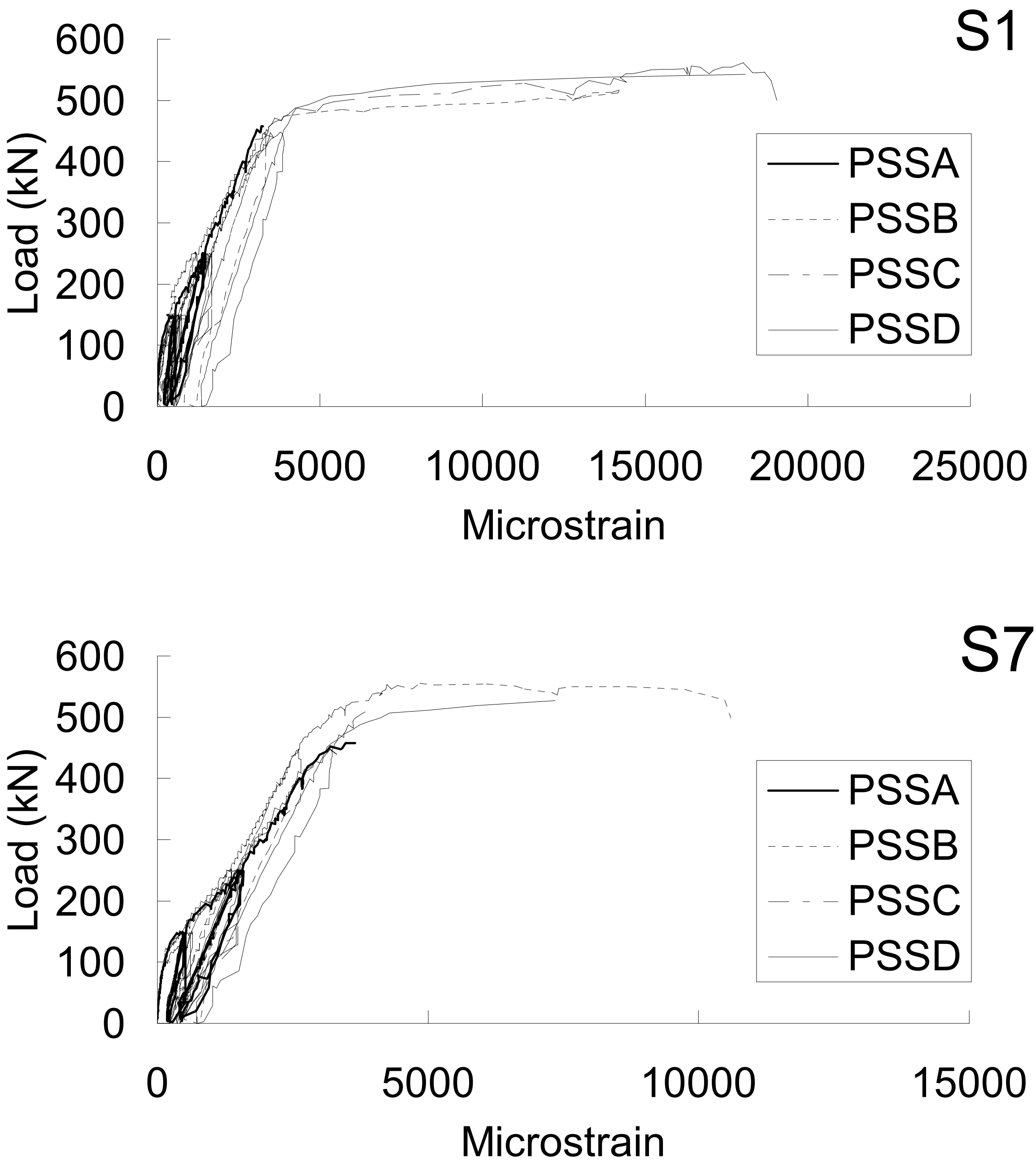


Fig.13 Load-strain relationship for strain gauges S1 and S7 on the flexural reinforcement

(Actual size)

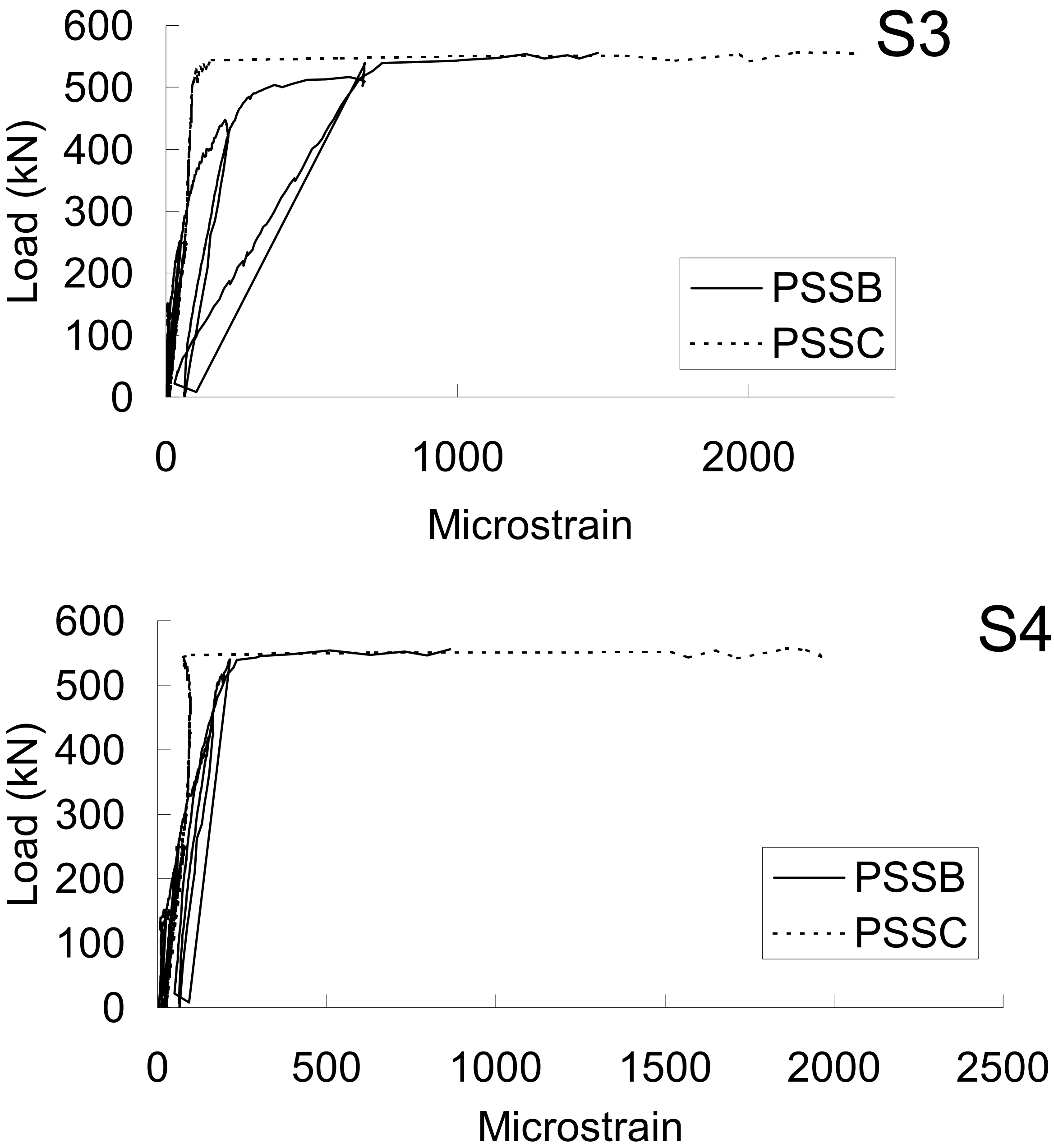
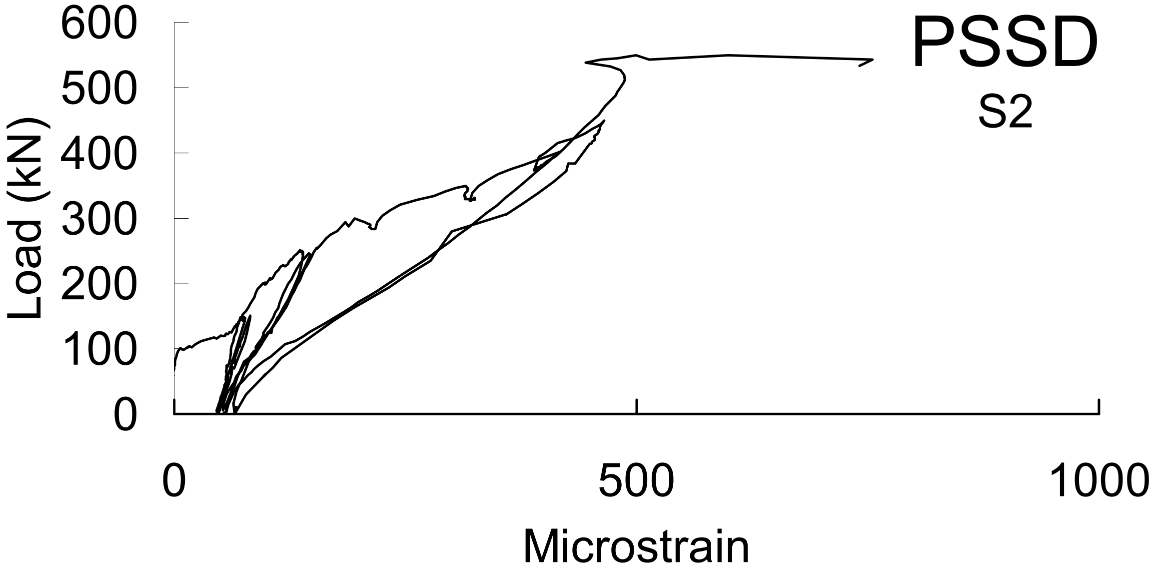


Fig. 14 Load-strain relationship on shear reinforcement on locations S3 and S4 in PSSB and PSSC

(Actual size)



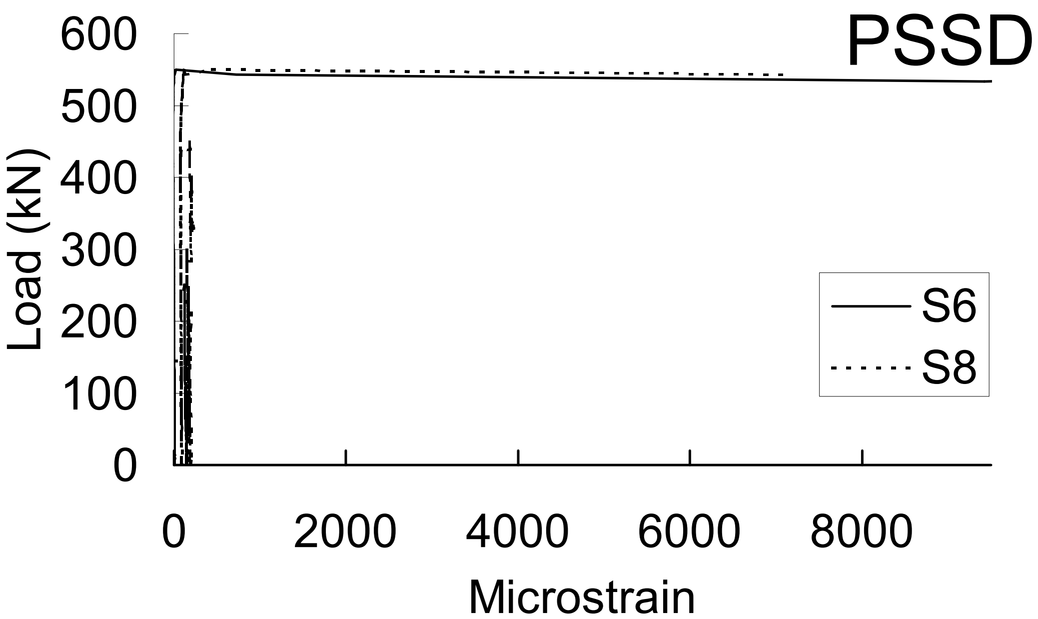


Fig. 15 Typical load-strain relationship in shear reinforcement in PSSD

(Actual size)

LIST OF FIGURE CAPTIONS