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EXPERIMENTAL STUDY OF THE BEHAVIOUR OF SHEAR WALL-COUPLING SLAB JUNCTION OF LATERALLY LOADED HIGH-RISE BUILDING

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Abstract

In this paper the authors present full account of an experimental study conducted on a model of relatively large size R.C model consisting of single shear wall and a portion of coupling slab subjected to the effect of lateral load, which was simulated by uniform displacement of the line of contra flexure. The research work carried here is aimed at studying the flexural behavior, particularly the extent of redistribution of stress in post elastic range and the suitability of Direct Ultimate Load design method for the design of such slab. Due to high concentration of bending stresses, shear and torsion caused by both the gravity as well as wind loading, possibility of failure of these buildings because of punching of walls through the slabs at their inner faces is profound and therefore it is an area of active research for last three decades. end. Results show that fibre reinforcement up to 1.5% has pronounced favorable effect on the strength attempt has been made to improve the strength of wall – slab junction by addition of circular steel fibre reinforcement the junction. Beyond this the effect is negligible.

Keywords: junction, sensitive area, load bearing walls, punching, circular steel, favorable

01. Introduction.

Increase in population, shortage of space to build and consequently high cost of land in urban
has led to the construction of high rise buildings. One major structural characteristic of tall buildings is that the effect of wind and seismic loads becomes more pronounced with the increase in the height of the building. The traditional system of providing lateral stiffness to buildings is essentially one of an extension of rigid structural frames with in-fills serving the purpose of dividing the space. This form, of construction suffers from the following disadvantages. (i) It is an inefficient form of providing lateral stiffness. (ii) As the lateral forces: vary, with the height of the building, the member, sizes also need to be varied. This, apart from preventing the exploitation of repetition, architecturally restricts flexibility of internal planning. (iii) Leads to complication at the joints.
(iv) The deep beams at lower levels can only be concealed with the use of a false ceiling which adds considerably to the volume and cost of the building.

Search for more economical methods of construction and types of structures resulted in the development of a type of tall buildings called shear wall structures.

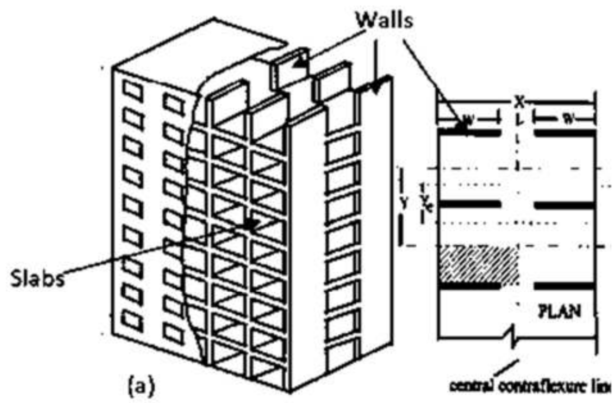

Figure. 1(a): Perspective view of a shear wall building
1(b): Plan of a typical shear wall building

These buildings consist of load bearing walls and slabs. A structure in which the walls (which includes core walls, facade walls or combination of these) carry both gravity as well as the lateral loading, is called a shear wall structure. In this structural form the floor slabs act as diaphragms distributing the horizontal loads to the vertical shear walls. Since no falls ceiling is required to hide the beams etc. The story height can be kept to a minimum. A perspective view of a typical building is shown in Fig. 1(a) .Ideal situation occurs when shear walls not only have the structural function of carrying vertical and horizontal loads but also the non-structural function of dividing and enclosing the space as well. This leads to the system of cross-wall construction as shown in Fig.1 (b). Shear walls are also used to enclose lift shafts and stairwells to form partially open section box structures. Thus, in practice, shear walls of various shapes, planar, flanged or box shaped, may be coupled together in cross-wall structures. Different wall configurations are shown in Fig. 2.

Figure 2: Different wall configurations

In designing tall buildings special consideration must be given to providing sufficient stability in all directions against lateral forces due to wind, earthquake or blast. These forces produce critical stresses in the structure, set up vibrations in the structure land, cause lateral-sway of the building which could reach a point of discomfort to the occupants. The shear walls resist the lateral loads on the structure by cantilever bending action, which results in rotations of the wall cross-sections. The free bending of a pair of shear walls is resisted by the floor slab, which is forced to rotate and bend out of plane where it connects rigidly to the walls as shown in Fig. 3.Due to the large depth of the wall, considerable differential shearing action is imposed on the connecting slab, which develops transverse reactions to resist the wall deformations as shown in Fig. 4, and induces axial forces, (tensile and compressive) in the walls.

A lot of research work has been carried out during last forty years on various aspects of the analysis and design of shear wall buildings. During the earlier days the shear wall buildings were analyzed using equivalent frame method where shear walls were replaced by idealized rigid frames. It was realized that full width of floor slab was not effective in resisting the lateral force because the shear induced by the lateral force was uneven with maximum intensity along the perpendicular line joining the inner faces of a pair of cross shear walls.

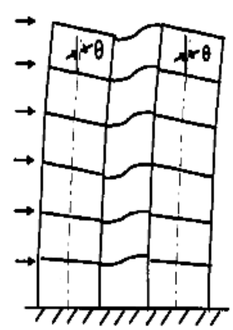

Figure 3: Deflected shape of a typical shear wall - slab structure subjected to lateral loads

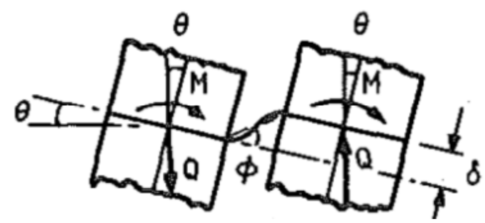

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Figure 4: Deflected shape of the connecting slab in shear wall structure.

Similarly, the bending moment due to lateral forces induced in the slab was maximum at the inner face of the wall and its intensity reduced drastically with the distance from this point. Therefore, the concept of effective width of slab which took part in resisting the effect of lateral forces was put forward by Qadeer & Stafford (1969). They used the numerical method of Finite element. Differences to evaluate the effective width of coupling slabs. Coupled shear wall with two and three bands of openings were pursued actively by Coul & Suedi (1972). Simplified analysis of coupled shear walls of variable cross-sections was presented by Pisanty & Traum (1972). Hago (1982) proposed the Direct Ultimate Load design moments. Ying Wood and Armer equations based on Yield Line Theory. The suitability of this method for coupling slabs was later checked by Mahmood (1984). One of the major problems is the huge concentration of shear, bending and torsional stresses in the slab around the wall periphery near its inner face. This may lead to punching failure of slab, which could be sudden, brittle, and without impending warning causing great loss of life and property. Therefore, Mahmood (1984) carried out systematic research on the behavior of wall-slab junction by testing real reinforced concrete models of relatively large size. Based on theoretical as well as experimental research he proposed the method to estimate the strength of wall-slab junction. Bari (1987) made use of vertical stirrups as shear reinforcement in the coupling slabs along the wall periphery to enhance its strength. Results of their findings about Stiffened Countered Shear walls were presented by Coull et al (1991). Farrar (1992) devised a method to *measure* the stiffness of concrete shear walls. Johnson (1993) pressed his research work regarding the static and dynamic analysis of coupled shear walls. The flexural behavior of coupling slabs was investigated by Muhammad Ayoob (1995). Dynamic analysis of a RCC shear wall with strain rate effect was investigated by Kazushi & Akira (1998). The problem pertaining to tension flange effective width in reinforced concrete shear wall was studied by Mohammad Hassan & sheriff (2003). Vinoth et al (2009), presented results of experimental behavior of wall-slab joint in a laterally loaded shear wall building. Greeshman et al (2011), studied the response of shear wall – floor slab connection containing various types of shear reinforcement when subjected to gravity and lateral cyclic loading. Since the thickness of the slab is quite commonly very small as compared with common beams, it is difficult to accommodate vertical stirrups in heavily reinforced coupling slab. wall periphery to strengthen the wall-slab connection so that Thus it was deemed imperative to provide a different type of reinforcement around punching failure could be avoided. Noor Ahmed (2003) made use of steel fibre consisting of twisted twins of steel wire.

02. Parameters governing the strength of the junction

Referring to Figure 5(a) and (b), the parameters which govern the strength of the junction can be divided into three categories as follows;

a. Load parameters: The force at the junction which effect the strength are

i. Shear force, bending and twisting moment due to gravity and lateral load at the junction at a section perpendicular to the wall

ii. Shear on at a section parallel to the wall.

b. *Geom*force, bending and twisting moments at the juncti*etrical* parameters *parameters*: The main geometrical which effect the strength are

i. Length of the wall web,

ii. Width of the wall flange,

iii. Width of the floor slab,

iv. Thickness of the slab.

c. *Material parameters*: The main material parameters are;

i. Amount of flexural steel.

ii. Amount of shear steel.

iii. Strength of concrete and steel.

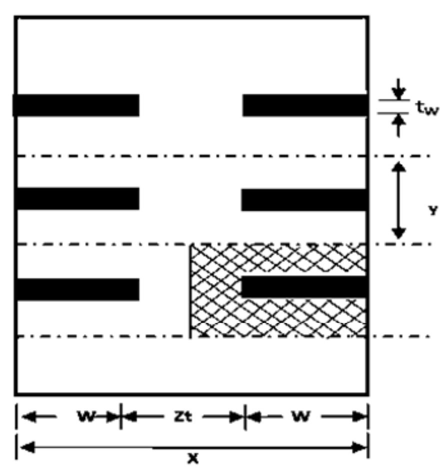

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Figure 5 (a): Plan of typical shear wall-slab building

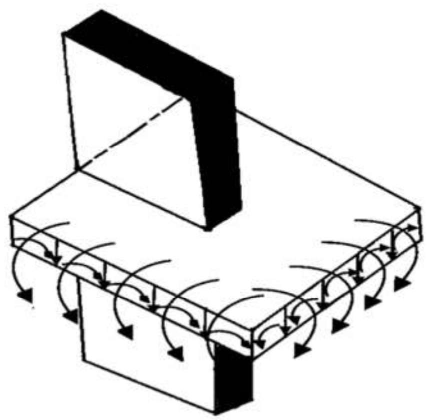


Figure 5 (b): Forces acting on a local region of the slab

03. Parameters chosen for study

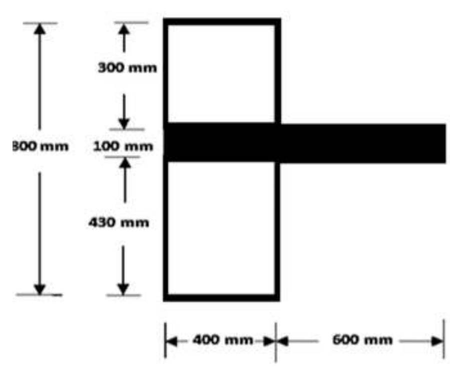
The major parameter of study was the quantity of circular steel fibre by weight. Here a ratio ranging from 0% to 2% with an increment of 0.5% was adopted. The fibre reinforcement was in the form of circular steel so that the bond problems could be overcome. The fibre was added during the mixing and the mixer was run for approximately ten minutes, the fibre was added bit by bit randomly till thorough might have affected the behavior more profoundly mixing was achieved. Proper alignment of fibre but since this could have become another parameter of study, deliberately this was avoided. The major reason is that the placing of fibre reinforcement in proper alignment on mass scale in actual tall buildings during construction would require a lot of labour leading to a very expensive and slow process which can be rated as next to impossible. In fact it has already been established that critical section exists at an average distance of d/2 from the wall periphery. Therefore, the maximum concentration of steel fibre in that region in vertical position might cause lot of improvement but practically this would prove difficult to achieve. Steel being a ductile material would help to absorb and dissipate energy induced due to dynamic lateral forces more efficiently. This particular aspect would determine the overall affordability of a project.

04. Size and Shape of Models

Since it was deemed best to test models of real reinforced concrete of relatively large size so that the effect of size on the results could be minimized and let the behavior of models during the experimental study correspond to that of actual structure as close as possible, the dimensions as shown in figure 6 were adopted for models. For these models the non-dimensional structural parametric ratios are as follows.

- $L_w/X_w = 0.6$
- $Y_w/X_w = 0.5$
- $t_w/W_w = 0.25$
- $t_w/Y_w = 0.1$

For the sake of economy only one shear wall and a portion of slab cut along the central line of corridor opening and the central lines of bay on both the sides of the wall was taken as specimen for this experimental study. Fig 7 shows the typical shape of the model. The thickness of wall, the thickness of slab, bay width, corridor opening width and wall width were selected in such a way that the non-dimensional structural parametric ratios remained within the practical range of such building. Although Mahmood M (1984). adopted the slab thickness of 150 mm for his main test series (MT), depending upon several factors the slab thickness for this experimental study was restricted to only 100 mm. The factors affecting this decision included the capacity of the supporting arrangement of models and other practical difficulties, which might have manifested themselves due to excessive rigid body



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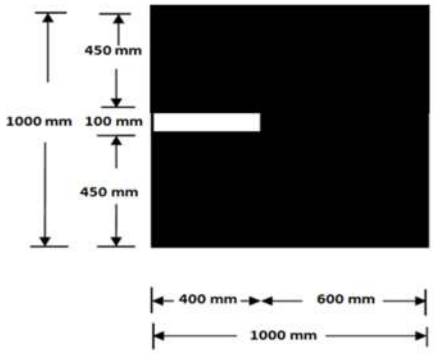

Fig 6: **Plan and elevation of the specimens showing dimension**

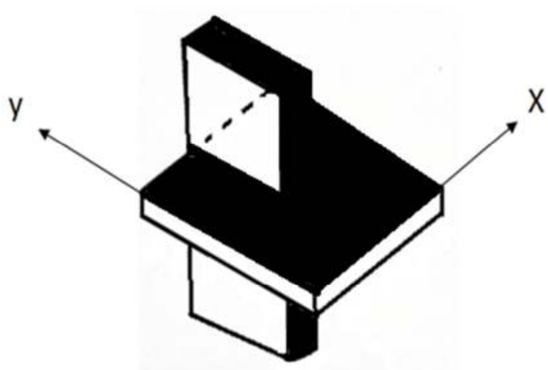

Fig.7: Isometric view of the specimen

rotations caused by relatively more loads, which would have been required because of enhanced flexural strength of the slab if thickness of slab were to be 150 mm. However the wall thickness, which was also 100 mm in this case as compared to 125 mm for (MT) series, was found to be enough, particularly from wall-slab junction strength point of view.

05. Supporting arrangement

Since for the sake of economy no base slab was cast, the model had to be positioned up-rightly at a proper location and held firmly to avoid rigid body rotation of model itself about its own wall edge and consequently creating stress concentration in these regions of wall leading to premature failure due to damage to the wall itself before reaching the ultimate failure load of the wall-slab connection, as desired.
Therefore a supporting arrangement (as shown in Fig. 8) consisting of rolled steel sections with a vertical column and a cantilever supporting arm was fabricated with thick square plate at base. A deep foundation of reinforced concrete consisting of 25mm diameter plain steel vertical bars threaded at the top, were driven in the ground, in order to fix the base supporting arrangement to the ground which would have otherwise got dislocated and its rotation and its rotation would have occurred.

During the test it was observed that strangely the supporting system was showing more deformation than expected as per the set of calculations based on elastic theory and standard values of geometrical and material properties. This could be attribute to the uncertainty about the very origin of the raw material used by the rolling mills for producing the steel sections available in the market. Consequently the total rigid body rotation was alarming too high and therefore unloading of the model was restored to, so that the supporting system could be strengthened. For this purpose, a hollow square steel section was used to act as a beam in the transverse direction near the inner face of the wall of the model.
This steel beam was manufactured by welding two 70 x 70 mm angles together. The beam thus formed was tugged at its proper position with the help of two vertical bars threaded at both the ends and firmly held at their bottom by nuts welded with 10 mm thick steel plate which in its turn was welded to the reinforcing steel bars of two separate foundations on both the sides, of the model.

06 Testing arrangement

Since the effect of lateral load was to be simulated by the uniform displacement of the line of contra flexure, a hollow square steel section was got fabricated by welding 70 x 70 mm rolled steel angle sections all the way through along their length. Two loading points were identified and 100 mm thick plate was welded there to avoid the local deformation because of concentration of load. Two manually operated hydraulic jacks each of ten tons capacity with a maximum extension of 140 mm were used for applying the load. Since in the laboratory there was no reaction floor hence beneath the jacks the ground was strengthened by constructing special foundation of RCC with only thin layer of concrete and plastered with thin coat of cement mortar, so that the possibility of sinking of the jacks in to the floor under load could be avoided. Although the uniform displacement of the line of contra flexure was achieved, for the sake of convenience, instead of imposing equal increment of displacements, the equal increments of load were applied gradually. Dial gauges as well as transducer were used to measure the displacement at the central point of the loading beam of each model. The testing arrangement of present study is shown in Fig. 08.

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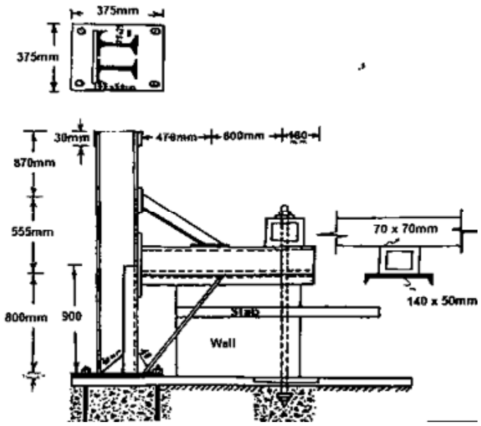

Fig.8: Dimensional details of supporting arrangement.

07. Test Procedure

Initially 5% of expected ultimate load was applied for a little time period and the model was unloaded. Readings were taken of all the dial-gauges, transducer, load cell and strain for each of the models with no load before starting the actual test. Care was taken to see that the applied load was not causing any eccentricity and consequent twisting of the model. To allow for the overall deformation, creep etc., the reading were taken five minutes after application of each load increment and the possibility of crack formation if any was observed. If there were any cracks formed, they were marked with a line and numbered at the tip by drawing a short cross line. The total time for each test was in the range of four to six hours. The strength properties of the concrete were obtained by testing the cubes and cylinders on the day of testing each model

Table 01: Comprehensive table showing all value of loads with a average crack location of the specimens.

Specimen No.	%age of C.S.Fibre	Experimental Ultimate Load	Ratio of Ultimate Load w.r.t. Specimen-01	Design Load	%age difference	Revised design Load	%age différence	Average Crack location
SCSFWSJ-01	0.00	42.10	1.22	40.8	8.1	39.9	10.5	79
SCSFWSJ-02	0.25	54.30	1.30	39.8	35.4	42.8	25.9	110
SCSFWSJ-03	0.50	58.70	1.44	39.7	43.8	45.4	25.8	132
SCSFWSJ-04	0.75	64.61	1.52	43.4	46.6	48.1	32.3	148
SCSFWSJ-05	1.00	69.32	1.57	52.5	28.2	41.9	60.7	146

08. Behaviour of Specimens.

08.1 Specimen SCSFWSJ - 01 (0% C.S. Fiber)

Let it be mentioned here that the abbreviation of sample "SCSFWSJ" is the short form of "Strength of Circular Steel Fibre Wall Slab Junction. For the first time a hair-crack visible by naked eye appeared at a load, which was 50% of ultimate load. As the loading progressed 01% to 80 this crack widened and more crack developed. Clearly this was a case of junction failure by which the wall punched through the slab. The failure was sudden, brittle and without impending warning. The failure occurred at the load of 42.10 kN.

08.2 Specimen SCSFWSJ - 02 (0.25% C.S. Fiber)

The behavior of this specimen was more or less similar to first model. The crack appeared at 40% of the ultimate load. The cracking progressed as the load increased. Several cracks appeared when the load reached 70% of the ultimate load. The cracks were quite wide when the load reached at a level of 80% of the ultimate load. Clearly this was also the case of punching of wall through the slab at junction. The failure occurred at a load of 54.30 kN.

08.3 Specimen SCSFWSJ - 03 (0.50% C.S. Fiber)

Obviously it was expected that the load bearing capacity of this specimen in terms of strength of junction would be higher than the previous specimen due to increase the quantity circular steel fiber. The ultimate load increased by only 7.7% than the

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previous one. However as expected the crack pattern and mode of failure resembled with the previous specimen. For the first time hair crack visible by naked eye at the bottom of the slab appeared at 37% of the ultimate load. Some more cracks appeared when the load reached at the 50% of the ultimate load. Several cracks at the top and bottom developed at 82% of the ultimate load. This was also the case of junction failure. The load at failure was 58.70 kN.

08.4 Specimen SCSFWSJ - 04 (0.75% C.S. Fiber)

By providing large quantity of circular steel fiber reinforcement, consequently higher ultimate load was expected to increase more flexural strain in the slab. The cracks just started at the at the bottom of the slab at 48% of the ultimate load. However, as expected the mode of failure resembled with those of the previous specimens, but cracking at the bottom showed different pattern. Several cracks radiated in various direction from the inner face of the wall. However, the sample behaved more or less like previous sample. This was also the case of junction failure. The load at failure was 64.61 kN.

08.5 Specimen SCSFWSJ - 05 (1.0% C.S. Fiber)

The mode of failure did not show any improvement with the increase of the quantity of this circular steel fiber. The cracking of this sample at the bottom was approximately similar *SCSFWSJ - 04.* The load at failure was 69.32 *kN.*

09. General Discussion

The most important observation regarding the behavior of the specimen is that the cracks causing

failure of the specimens. It appears from the experimental evidence of this study that critical shear perimeter shift away from the sides of wall due to increasing the special form of circular steel fibre reinforcement. Although the mode of failure was the same i.e. punching of wall through the slab which is the case of junction failure. Complete account of the test results are present in table 01.

Based on the test results it can be deduced that by increasing special form of circular steel fibre reinforcement would give better estimation of strength of wall slab junction.

Here the actual load at failure is compared with design load and the revised design load. From table 01 it can be observed that in case of all the specimens containing special form of circular steel fibre reinforcement the revised design load is closer to the experimental ultimate load than design load except specimen containing 1%. Thus it can be deduced that the shift of critical shear perimeter should be taken into consideration and this would give better estimation of the strength of wall slab junction in case of the special form of circular steel fibre reinforcement. Generally the estimated loads are somewhat lower than the experimental load giving a potential factor of safety of **24% and 13%** respectively. Since the calculations for estimation of the strength of wall slab junction are quite complicated. It must be mentioned here that the compressive strength of concrete improves with addition of special form of circular steel fibre reinforcement.

As expected, the ultimate load of the specimen with no special form of circular steel fibre reinforcement was the lowest amongst all the specimens tested i.e. 42.1 kN.

It is apparent that ultimate load as well as the displacement increase as the ratio of special form of circular steel fibre reinforcement increases. The deformation becomes even more than 60% of the thickness of the slab itself at failure of specimen SWSJWNR-05, containing maximum shear reinforcement. Clearly this indicates that there would be excessive deflection of the slab in real structured giving a warming that the failure would imminent. Since the junction failures both in case of flat slabs and coupling slabs are sudden and brittle.

The maximum deflection of the line of contra flexure simulating the effect of lateral load was hardly 30% of the thickness of slab, which is too small, and a manifestation of britlity leading to disastrous failure. It may be mentioned here that the design ultimate load i.e. 40.1 kN of this model was taken as the base value and the applied load was non-dimensionalized by dividing it with this value. Similarly, the deflection / displacement of line of contra flexure was non-dimensionalized by dividing it with thickness of slab. however, the ultimate load as well as maximum displacement go on increasing with the increase of special form of circular steel fibre reinforcement up to 1.5%. Never the less it is astonishing to note that with further increase of fibre

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ratio the load increases slightly but the deflection is even lesser. From this it can be concluded that the use of special form of circular steel fibre reinforcement beyond 1.5% is of no use. Infact the deflection is a measure of ductility, which is a sort of indication that the failure is imminent. This serve as a fore-warning so that catastrophic effect and consequent damage in terms of loss of human life and property could be avoided

As we already know that the failure of junction with no special form of circular steel fibre reinforcement is sudden brittle and without impending warning, this particular effect of wide-open crack and large deflections are a positive point in terms of the use of fibre reinforcement.

From table 02, it can be observed that an addition of 0.5% of special form of circular steel fibre reinforcement caused an increase of 18% in the strength of wall slab junction as compared with that of the specimens containing no circular steel fibre reinforcement. This enhancement is up to 29% when circular steel fibre ratio was 1% and attained an enhancement up to 45.4% when the fibre was 1.5%. Further increase of fibre to 2.0% caused an improvement of strength being as high as 51.7%. however, if we compare the enhancement for 2% with that of 1.5%, the increase is only the order of 7.3%. Therefore, it can be concluded that increasing the ratio of circular steel fibre reinforcement beyond 11.5% would prove be uneconomical.

The most important observation regarding the behavior of specimens is the crack causing failure of the specimens. It appears from the experimental evidence of this study that critical shear perimeter shifts away from the sides of wall due to increasing the special form of circular steel fibre. Although the mode of failure was the same i.e. punching of wall through the slab which is the case of junction failure.

Based on the test results it can be deduced that by increasing special form of circular steel fibre reinforcement would give better estimation of the strength of wall-slab junction.

It is apparent that ultimate load as well as the displacement increase as the ratio of special form of circular steel fibre reinforcement increases. The deformation becomes even more than 60% of the thickness of the slab itself at failure of specimen SWSJWNR-05, containing maximum shear reinforcement. Clearly this indicates that there would be excessive deflection of the slab in real structures giving a warming that the failure would imminent. Since the junction failures both in case of flat slabs and coupling slabs are sudden and brittle. This excessive deflections before failure is a positive point.

Based on the results, the method to estimate the strength of wall-slab junction originally proposed by Mahmood. (1984) has been modified to take into account the additional component of strength imparted by this new form of circular steel fibre reinforcement.
Here as expected strain is quite considerable when no circular steel fibre reinforcement is added. Nevertheless it decreases to its lowest value when the ratio of this special form of circular steel fibre reinforcement is 0.88%.
Similarly, situation is encountered in case of compressive strain measured at various other locations in the slab. It is obvious that in all the specimens the strain is the lowest in the slab along the critical section at the central point near the inner face of the wall.

Table 2: Percentage increase of experimental ultimate load

S.#	Model	%age of Fiber	Experimental Ultimate Load (kN)	%age increase w.r.t SERWST-01
01	SERWST-01	0.0	42.10	-----
02	SERWST-02	0.5	54.30	18..1
03	SERWST-03	1.0	58.70	29.5
04	SERWST-04	1.5	64.61	44.4
05	SERWST-05	2.0	69.32	51.7

10. Conclusions

1. Addition of circular steel fibre reinforcement considerably reduces the strain in concrete presumably because the

steel fibre itself resists bulk of stresses.

2. Addition of circular steel fibre reinforcement beyond 1.5% has no drastic favorable effects on the strength of wall-slab junction.
3. The addition of circular steel fibre reinforcement in addition to flexural steel causes the shifting of cracks somewhat further away from the wall periphery than those containing no fibres in the model consisting of one planar shear wall and a portion of coupling slab implying the shifting of critical shear perimeter itself.
4. Modulus of elasticity was also improved with the addition of circular steel fibre rei**nf**orcement

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Experimental studies the details of which are presented in this research paper, was carried out in the structures laboratory at QUEST, Nawabshah, Sindh. Pakistan.

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