Study of the Behaviour of Wall-Slab Junction with Special Form of Shear Reinforcement

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***Abstract*—Wall-Slab junction is a very sensitive area of high rise buildings consisting of floor slabs directly supported by load bearing walls. The floor slabs act as sole coupling media between pairs of crosses walls. 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. Therefore, attempt was made previously to increase the strength and ductility of wall – slab junction by the addition of steel wire couplets twisted together along the periphery of wall-slab junction. However, use of this type of fiber beyond 1.5% by weight is rather uneconomical and with no further appreciable improvement of the performance of junction. Shear reinforcement in the form of vertical stirrups was tried but since the slabs are thin, it is difficult to accommodate such type of reinforcement. Therefore, systematic experimental research has been carried out and based on the results details are presented about the effect of special form of reinforcement consisting of ½ inch wide "I" sections placed at various locations within the slab around the wall periphery. This type of reinforcement seems to have a very favorable effect and therefore the performance of wall – slab junction could be improved to the desired level.**

Keywords—sensitive area; load bearing walls; punching; ductility; fiber;

I. INTRODUCTION

Initially the tall buildings were designed as skeletal structure comprising of beams and columns of rolled steel sections encased in concrete and RCC floor slabs. For tall buildings it is not the gravity load but the lateral force due to wind and earthquake, which becomes the controlling factor in the analysis and design of such buildings. The effect of wind becomes more pronounced with the increase in the height of the building and it acts like a cantilever fixed at base. The skeletal structure particularly proved inadequate to provide resistance against lateral forces and therefore considerable lateral sway of these buildings caused discomfort and sense of insecurity amongst the occupants of these buildings. Attempts were made to provide lateral stability to those buildings with the help of wide base and inclined / curved bracing beams as shown in Fig: 1 & 2. However, search for more efficient form of construction led to the concept of shear wall buildings consisting of load bearing cross RCC walls solely connected by floor slabs, called coupling slabs (with no beams or columns). Typical Plan of a slab panel with pair of coupling walls is presented in Fig 3, while perspective view is presented in Fig 4.

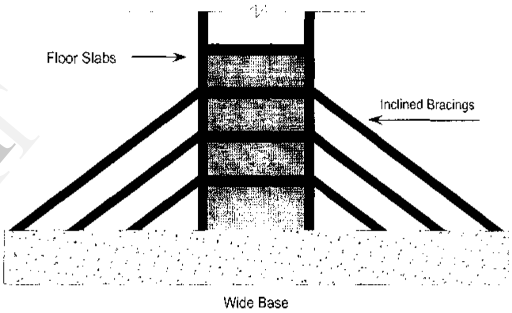

Fig. 1. Typical structures frame consisting of coloumns-beams along with inclind bracing provided for lateral stability.

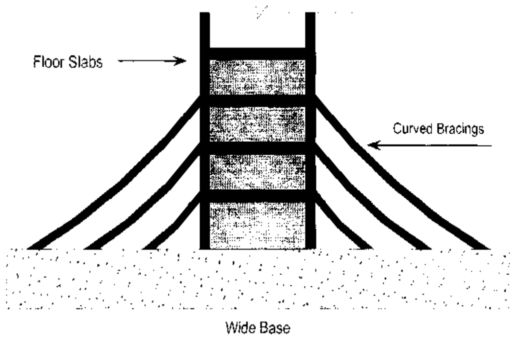

Fig. 2. Typical structures frame consisting of columns-beams along with curved bracing for lateral stability

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A lot of research work has been carried out during last

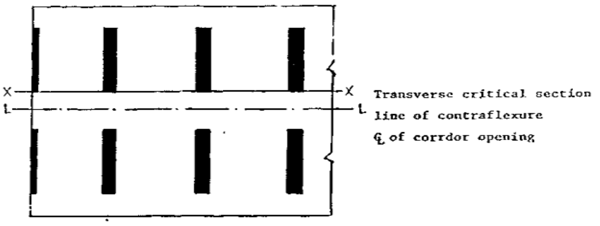

Fig. 3. Plan of typical shear wall building showing transvers critical section

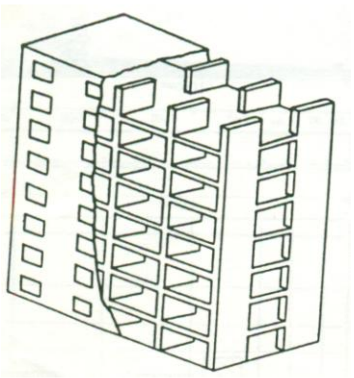

Fig. 4. Perspective view of a shear wall building.

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.

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 [1]. They used the numerical method of Finite 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 [2]. Simplified analysis of coupled shear walls of variable cross-sections was presented by Pisanty & Traum [3]. Coull & Wong investigated the matter Design Method (DULDM) for flexural Design of common slabs where he found moment triads with the help of Elastic Finite Element analysis for the ultimate loads but found further by using Finite Element Method [4]. Hago proposed the Direct Ultimate Load design moments from these triads by applying Wood and Armer equations based on Yield Line Theory [5]. The suitability of this method for coupling slabs was later checked by Mahmood [6]. 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 [6] carried out systematic research on the behaviour 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. Ghassan Elnounu [7] extended the study of the strength of wall-slab junction to other wall configurations instead of planar cross walls considered by Mahmood [6] as shown in fig. 5. Bari made use of vertical stirrups as shear reinforcement in the coupling slabs along the wall periphery to enhance its strength [8]. Results of their findings about Stiffened Coupled Shear walls were presented by Coull et al [9]. Farrar devised a method to measure the stiffness of concerte shear walls [10]. Johnson presented his research work regarding the static and dynamic analysis of coupled shear walls [11]. The flexural behavior of coupling slabs was investigated by Muhammad Ayoob [12]. Challal & Nollet worked on upgrading the degree of coupling of coupled shear wall [13]. Dynamic analysis of a RCC shear wall with strain rate effect was investigated by Kazushi & Akira [14]. The problem pertaining to tension flange effective width in reinforced concrete shear wall was studied by Mohammad Hassan & sheriff [15]. 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. Thus it was deemed imperative to provide a different type of reinforcement around wall periphery to strengthen the wall-slab connection so that punching failure could be avoided. Mahmood Memon assessed the strength of wall-slab junction under various circumstances without any shear reinforcement. Noor Ahmed made use of steel fibre consisting of twisted twins of steel wire 1 mm diameter as shown in fig. 6 [16]. The major parameter was the ratio of fibre reinforcement, which ranged between zero to 2% by weight.

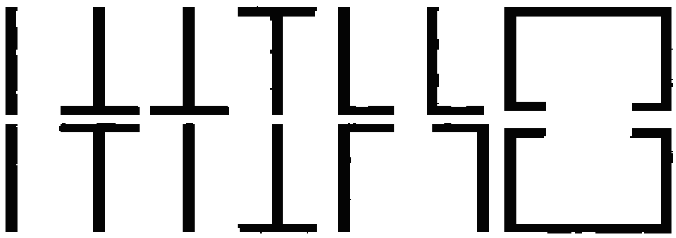

Fig. 5. Different Wall Configurations.

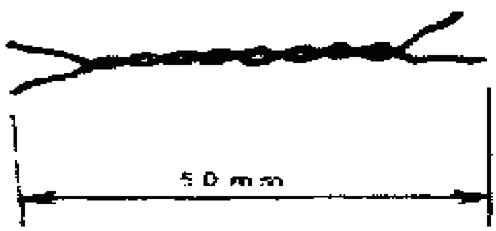

Fig. 6. The sketch showing twisted twin steel fibre used by Noor Ahmed.

II. Present Investigation

The testing arrangement used for this research is presented in fig 7 while dimensional sketch in shown in Fig 8. The I-section used as shear reinforcement around the periphery is shown in Fig 9 and its location is presented in Fig 10. The major parameters of study included the location of I-section,

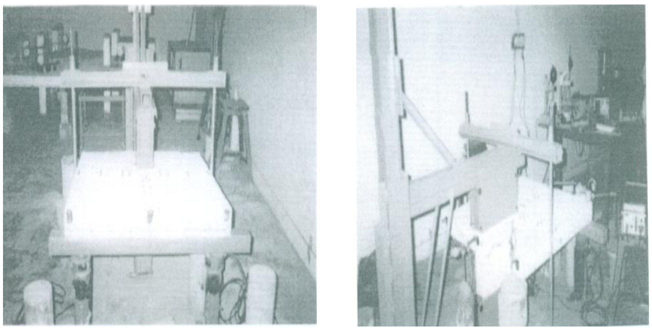

Fig. 7. Photographs showing the pictorial view of testing arrangement & view of model testing

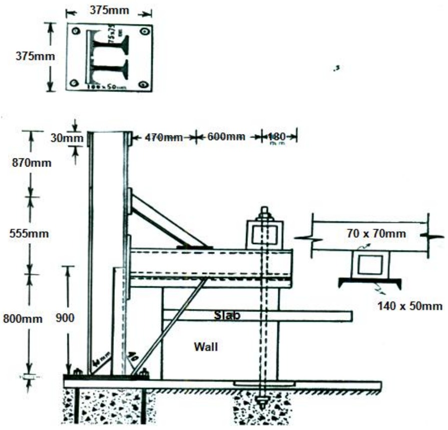

Fig. 8. Dimensional details of supporting arrangements.

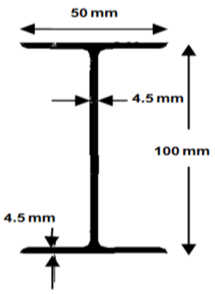

Fig. 9. Cross-Sectional view of I-Section used as special form of shear reinforcement.

which ranged between 0.5 to 0.65d, the spacing of the pieces and the rate of improvement of the ultimate load. Details of flexural reinforcement in one of the model are presented in Fig 10. In all six models were tested. Eleven I-section pieces used as special form of Shear reinforcement at the critical section around the wall periphery at a distance of 0.5d from the sides of walls. Where d is effective depth of slab. The shear reinforcement used was 0.74 % of the critical area around the wall periphery. The cracking appeared at 40% of the ultimate load. The cracking progressed as the load increased. Several cracks appeared when the load reached 70% of ultimate load. The cracks were extended and widened when the load reached at 80% of ultimate load. The crack pattern of this model as seen from top, bottom and the back of the model along with the measurement of distance of cracks is shown in fig 11.

Clearly this was also the shear falure due to punching of

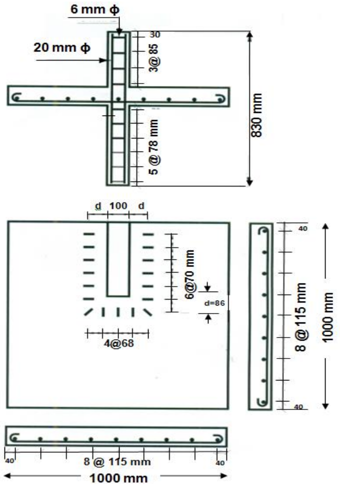

Fig. 10. Arrangement of reinforcement and location of I-section of the model.

wall through the slab. The failure occurred at a load of 53.04 kN. Strain versus stress at various locations and different stages of loading are presented in fig 12.

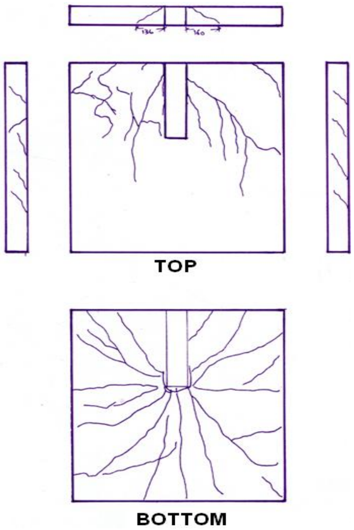

Fig. 11. Crack pattern of the slab at top & bottom with sides and back of the model SWSJNR-03

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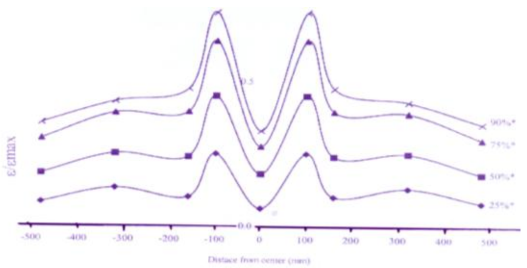

Fig. 12. Variation of compressive strain in concrete along transvers critical section different stages of loading in the slab of SWSJWSR-2

III DISCUSSIONS OF RESULTS

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 shifting of location of I- section piece used as special form of shear reinforcement; 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 the shift of the critical perimeters should be taken into consideration because this would give better estimation of the strength of wall-slab junction in case of special form of shear reinforcement. It is therefore recommended that the new location of critical shear perimeter be taken at a distance of 0.75d instead of 0.5d. Hence the estimation of wall-slab junction of laterally loaded shear wall building should be based

TABLE I. COMPREHENSIVE TABLE SHOWING ALL VALUES OF LOADS WITH AN AVERAGE CRACK LOCATION OF SPECIMENS

Specimen No.	No. of I-section	Experimental Ultimate Load	Design Load	%age difference	Revised design Load	%age difference	Average Crack location
SWSJWNR-01	Nil	44.1	40.8	8.1	39.9	10.5	79
SWSJWNR-02	11	53.90	39.8	35.4	42.8	25.9	110
SWSJWNR-03	14	57.10	39.7	43.8	45.4	25.8	132
SWSJWNR-04	14	63.64	43.4	46.6	48.1	32.3	148
SWSJWNR-05	14	67.32	52.5	28.2	41.9	60.7	146
SWSJWNR-06	17	69.36	52.4	32.4	48.0	44.5	140

TABLE II. PERCENTAGE INCREASE OF EXPERIMENTAL ULTIMATE LOAD

Specimen No.	No. of I-section	Location of I-section	Experimental ultimate load	Ratio of Ultimate Load w.r.t. specimen 01
SWSJWNR-01	Nil	Nil	44.1	----
SWSJWNR-02	11	0.5d	53.90	1.22
SWSJWNR-03	14	0.5d	57.10	1.30
SWSJWNR-04	14	0.65d	63.64	1.44
SWSJWNR-05	14	d	67.32	1.52
SWSJWNR-06	17	d	69.36	1.57

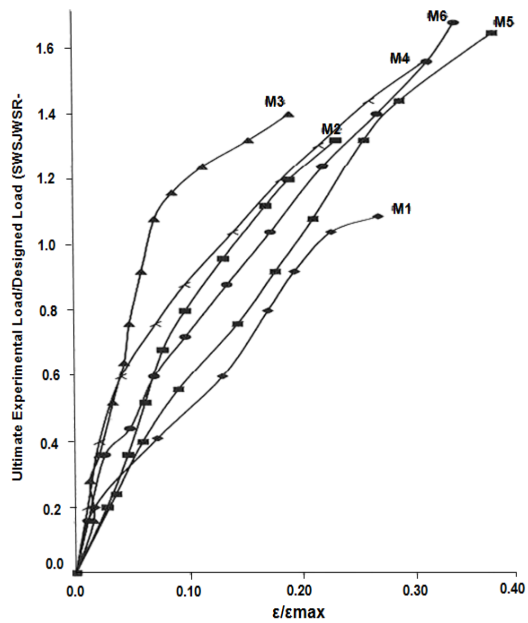

Fig. 13. Load displacement relationship of all the models. i.e. SWSJWNR- 01 to SWSJWNR-06

on 0.75d instead of 0.5d, when this type of special form of shear reinforcement is used. Figure 13 shows a comparison of the load-displacement relationship of all the test specimens. It is apparent from the figure that ultimate load as well as the displacement increases as the ratio of special shear reinforcement increases. The deformation becomes even more than 60% of the thickness of the slab itself at failure of specimen SWSJWNR-06, 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. Increasing of shear reinforcement in specimens showed further shifting of shear perimeter to a distance of d (d is effective depth of slab). However, from the test results which include crack pattern, mode of failure, location of cracks causing failure in the slab and their locations, it is suggested that the critical perimeter for junction failure around the wall periphery preferably be assumed to be at a distance of 0.75d instead of d/2. It may be mentioned here that for slabs the location of the critical shear perimeter is assumed to be at a distance of d/2 by ACI-318 and 1.5h (where h is over all depth) by CP8110. Based on the results, the method to estimate the strength of wall-slab junction originally proposed by Mahmood. M [6] has been modified to take into account the additional component of strength imparted by this special form of shear reinforcement. Here as expected strain is quite considerable when no special form of shear reinforcement is added. Nevertheless it decreases to its lowest value when the ratio of this special shear reinforcement is 0.88%. Obviously this is due to the fact that bulk of stresses is born by the special form of shear reinforcement. 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. Several other investigators have already reported this fact. The value is highest at second points from central locations.

IV. Analysis of Results

The last column of the Table I show the average crack location in millimeters from the side of the wall. During experimental study it was observed that the shear crack causing the failure of first specimen was averagely 79 mm away from the side of the wall. However this distance increased in the subsequent specimen as the special form of shear reinforcement consisting of pieces of I-section were placed and increased in the subsequent specimens. From Table-I, it can be observed that the change of location of I-section pieces was also changed from 0.5d to 0.65d and finally to d. This seems to have affected the location of cracks, which increase to a maximum value of 148mm in case of specimen SWSJWNR-04. Based on the results of this experimental work, it is suggested that this type of shear reinforcement be provided at distance of 0.75d from the wall. Table I, gives the details of experimental ultimate design loads, the initial design loads, the revised designed load and the percentage difference for all the specimens. In all the cases it can be observed that the experimental load is substantially higher than even the revised design loads, a maximum difference of 60.7% is reached in the case of specimen SWSJWNR-05. In table II, a maximum increase of the load (w.r.t specimen SWSJWNR-01 without shear reinforcement) of 1.57 times in case of specimen SWSJWNR-06. From this study it can be inferred that failure was governed by the strength of concrete and the steel could play only limited role rather than controlling the failure of the junction. The increase of I-section pieces from specimens SWSJWNR-05 to SWSJWNR-06 caused only a marginal increase of approximately 1.05 times of the ultimate load. Therefore, further increase of this type of reinforcement may not be useful. The ratio of shear reinforcement as percentage of the area of critical shear perimeter is only 0.97%. It is therefore, recommended that the placement of this type of reinforcement must not be beyond 1%. However, when the location of critical shear perimeter is assumed to be at a location of 0.75d instead of 0.5d, the estimated strength as predicted by M. Memon's [6] method will also be enhanced due to increase area. But substantial factor of safety would be still exist.

V. Conclusion

Based on the extensive study of the behaviour, the rate of improvement of shear strength and the location of I-section pieces of ½ inch size, the following conclusion have been drawn.

01. A maximum improvement of up to 57 percent has been achieved. However, to be on safe side a maximum guaranteed improvement of 50 percent may be assumed for design purpose when special shear reinforcement is provided to the extent of 1%.
02. A ratio of 0.88 percent of critical section for shear along the wall periphery seems optimum.
03. This special form of shear reinforcement may be placed at a distance of 0.75d instead of 0.5d (d/2), where d is effective depth of slab.
04. The deflection at failure of the slab increases by up to 60% when special form of shear reinforcement is provided, thus giving warning of imminent failure, which is a positive point.
05. Although special form of shear reinforcement shows substantial improvement of strength of wall-slab junction, however, full strength of steel shear reinforcement is not utilized.
06. Amendments in the original method have been proposed to take into account the inclusion of special form of shear reinforcement revised method has been presented.
07. From the conclusions and comparison it is observed that the agreement between the actual load at failure and those predicated theoretically using proposed method is reasonable.

VI. SIGNIFICANCE OF RESEARCH

Wall-slab junction is highly sensitive area of tall shear wall buildings where high concentration of stresses due to bending, shear and torsion are caused by lateral and gravity forces. This can lead to a premature failure of tall shear wall buildings. Although a significant amount of work has been carried out in the area under research, more work is required in order to develop a definite design procedure with more convenient and economical type of shear/ torsion reinforcement in terms of ratio v/s economy. This aspect is under taken as part of research program; the details of which are presented in this paper.

VII. ACKNOWLEDGEMENT

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