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Characteristics of deformable vertical joints in prefabricated shear wall assembly

S. C. Chakrabarti, N. N. Bhise and K. N. Sharma

In large panel building systems, structural walls are constructed to room size and storey-high prefabricated wall panels. The vertical joints in the shear wall assembly play an important role in determining the structural response of the shear wall against lateral load. The conventional assumption of vertical joints to be rigid is an oversimplification. In an accurate analysis, the shear deformability characteristics of the vertical joint should be accounted for, and a prefabricated shear wall system should be considered as a multi-panel cantilever wall coupled together through the shear continua of the vertical joints. In the paper, the results of testing joints in twenty-three specimens are reported. An empirical formula for deriving the load carrying capacity of vertical joints has been suggested. It has been found that coefficient of shear deformability at the working load level varies from 50 to 150kg/cm³.

Large panel buildings with storey-high and room-size prefabricated walls are gradually becoming popular in India, although they have been limited to four to six storeys at present. Messrs Hindusthan Prefab Limited, Delhi have been building a number of such four-storeyed buildings at Delhi.

In case of large panel multistoreyed buildings, wall rigidity and structural response of the prefabricated shear wall system with vertical joints in between the assembled prefabricates depends greatly on the shear deformability characteristics of the joints[1]. Generally, in structural design, the joints are considered to be rigid and the whole shear wall assembly as a homogenous cantilever. But, due to shear deformability, vertical joints form shearing surfaces and the statical scheme of a homogenous cantilever is, in reality, changed to that of multi-panel cantilevers coupled through the shear continua of the vertical joints[2]. The rigidity, or in other words, the flexibility of prefabricated shear wall system against lateral load thus depends on the shear deformability characteristics of the vertical joints.

A number of tests has earlier been carried out to study the load bearing capacity and shear deformability characteristics of joints [3,4,5,6]. The mechanism of shear transfer through the joints from one prefabricate to the adjacent one depends on a number of factors[4].

(*i*) Shape of joint which is determined by its geometrical dimensions and profile of the edges of the adjoining prefabricates *e.g.*, plain, grooved, shear key joint, *Fig* 1

(*ii*) ratio of transverse reinforcement to the shear key area of the joint

(*iii*) area of shear keys

(*iv*) characteristic strength of concrete or mortar filled in the joint f_c, and

(*v*) technological factors like homogeneity of concrete in joint and its compaction, control of water cement ratio, prevention of shrinkage, etc.

S. C. Chakrabarti, Scientist, Central Building Research Institute Roorkee, UP
N. N. Bhise, Scientist, Central Building Research Institute, Roorkee, UP
K. N. Sharma, Scientific Assistant, Central Building Research Institute Roorkee, UP

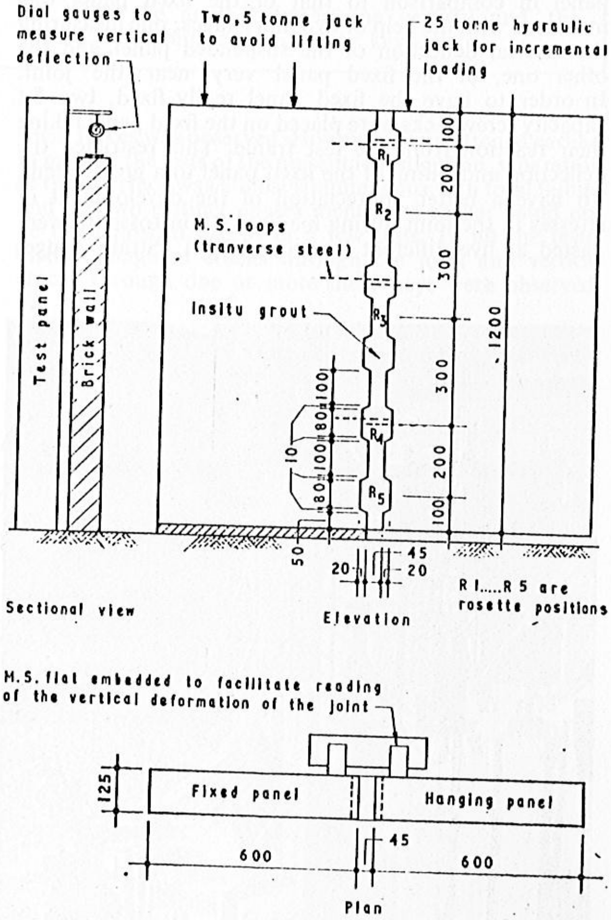

Fig 1 Details of test joints

Experimental work

Considering the important role of shear deformability characteristic of vertical joints in predicting the structural response of prefabricated shear wall system against lateral load, an elaborate test programme was undertaken at the Central Building Research Institute to study the mechanism of failure of vertical joints in shear and to derive the load carrying capacity. The detailed dimensions of a typical test specimen are shown in *Fig.* 1. Different parameters studied in the tests are as follows:

(*i*) compressive strength of concrete filled in the joint

(*ii*) area of shear key

(*iii*) area of transverse reinforcement distributed uniformly along the joint.

In all, test specimens of vertical joints were tested, details of which are given in *Table* 1. While filling the vertical joints with concrete, three cubes were taken for each joint and their strength was determined on the day when the shear deformability test was carried out. Complete experimental set up for a shear deformability test is shown in *Fig* 2. In order to study the shear deformability characteristics; *i.e.*, the variation of shear deformation in the joint with increase in load, deflection of the suspended panel in comparison to that of the fixed panel was measured with the help of two dial gauges; one measuring the vertical deflection of the suspended panel, and the other one, of the fixed panel very near the joint. In order to have the fixed panel really fixed, two 5-t capacity screw jacks were placed on the fixed panel taking their reaction from the test frame. This restricted the deflection and tilting of the fixed panel to a great extent. To have a better appreciation of the development of stresses in the joint during loading, strain rossettes were pasted at five different locations, *Fig* 1. Strain gauges

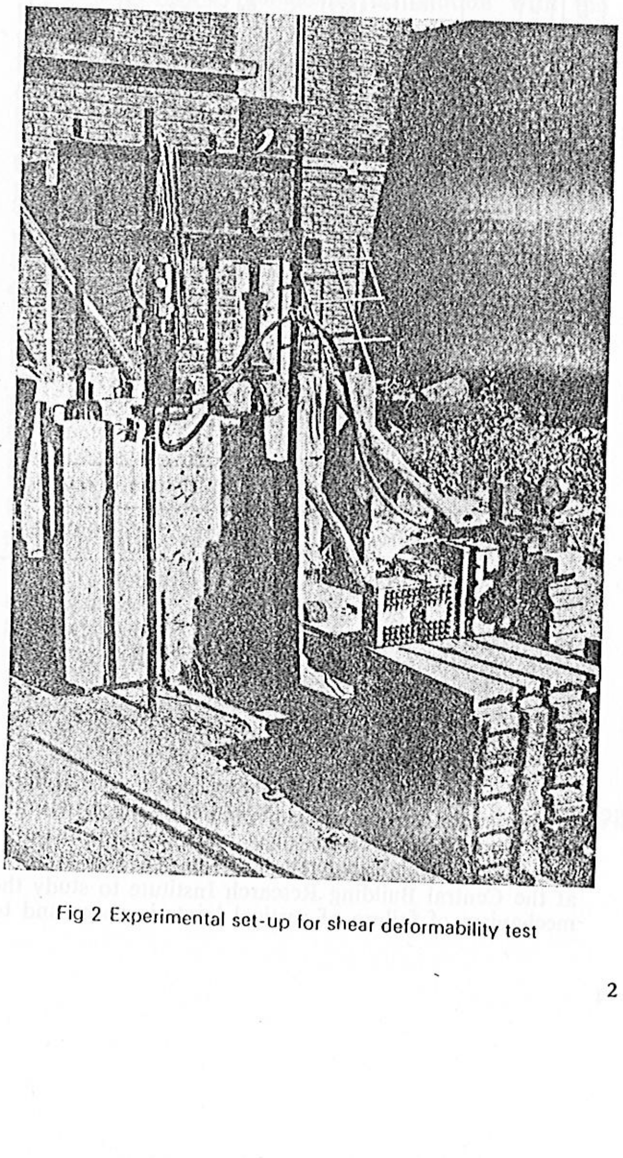

Fig 2 Experimental set-up for shear deformability test

were also fixed in the horizontal reinforcement bars protruding inside the joint to measure the level of stresses during loading. Load on the suspended panel was applied with the help of a 25-capacity hydraulic jack at one tonne interval.

Test results and discussions

The failure loads of the 23 vertical joints tested are given in *Table* 1. In specimen type A in which there was no reinforcement in the joint, the failure of the joint was comparatively sudden; whereas, in specimen type B, C, D and specially types C and D, better ductility before failure was observed.

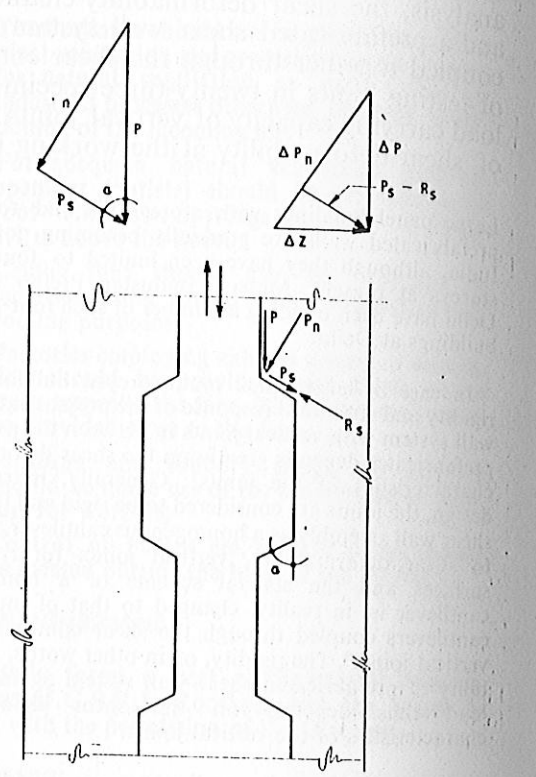

Fig 3 Load transfer mechanism in shear key

Earlier studies on the behaviour of vertical joints have revealed two different working phases of shear key joint[4]. In phase I, the joint and the prefabricates carry the load till such time separation occurs at least along one interface of the joint and prefabricates. In phase II, the separation cracks along the edges of the prefabricates are complete and the load is transferred from one prefabricate to the adjacent one by the mechanical action of shear keys. This has been clearly explained by Cholewicki[4].

Referring to *Fig* 3, slippage will not occur so long as

$$P_s < R_s$$

or, $P \cos\alpha < P \sin\alpha \tan\phi$

or, $\tan\alpha > \dfrac{1}{\tan\phi}$

or $\alpha > 56°$, assuming the coefficient of friction in concrete to concrete as 0.7.

When α is less than 56°, the transverse reinforcement prevents slipping by taking the horizontal component δP_s of the slipping force

NOTATIONS

A = area of shear surface in the joint

A_k = area of all the shear keys, cm²

A_s = area of transverse reinforcement, cm²

A_{sh} = cross sectional area of vertical joints, cm²

a_k = area of one shear key, cm²

f_c = characteristic strength of concrete, kg/cm²

f_s = stress in transverse reinforcement, kg/cm²

f_{sh} = shear strength of concrete

f_y = yield strength of transverse reinforcement, kg/cm²

K = coefficient of shear deformability, kg/cm³

k_c, k_s = empirical constants for concrete and reinforcement contribution to load carrying capacity.

n = total number of shear keys in the joint

P = vertical shear at the joint

P_s = component of P parallel to the sloping surface of shear key

p = $\frac{A_s}{A_k}$

R_s = frictional resistance along the sloping surface of shear key

$\tan\phi$ = coefficient of friction in concrete to concrete

σ_c = cylinder strength of concrete

δP = horizontal component of slipping force, $(P_s - R_s)$

τ = shear stress

Δ = shear deformation

$\delta\tau$ = incremental shear stress

$\delta\Delta$ = incremental shear deformation of the suspended panel relative to the fixed panel

α = the angle between the vertical and the sloping edge of shear key.

$$\delta P_s = \frac{n(P_s - R_s)}{\cos\alpha} = nP(1 - \tan\alpha\tan\phi) \qquad \ldots(1)$$

In most of the tests, destruction of bond occurred slightly before the total failure of the joint; although in a few tests, the two phases mentioned above were distinguishable. Different investigators have reported failure-types of joints differently. Hansen and Olesen observed diagonal cracks across the joint after separation at the interface. In some cases, diagonal cracks did not form at all, but local crushing under the concentrated shear forces was observed. However, as mentioned earlier, in most of the tests of the present investigation, separation at the interface was almost simultaneous with total failure and the two distinct phases mentioned earlier could not be distinguished except in a few tests. In all the tests, distinct diagonal cracks through the joint and vertical cracks through one or more shear keys were observed, *Fig* 4.

From the strain measurements by the rossettes pasted along the height of the joints, no clear indication of the

TABLE 1 Test results

Specimen type	*No of shear keys*	*Area of transverse reinforcement,* cm²	*Value of fc,* kg/cm²	*Observed failure load,* tonne	*Failure load according to equation 3,* tonne	*Predicted failure load,* tonne	*Predicted failure load according Cholewicki's formula*	*Value of K,* kg/cm³
A-1	6	—	90	6.00	6.62	4.80	7.56	—
A-2	6	—	60	7.50	5.41	3.24	6.17	—
A-3	6	—	60	5.50	5.41	3.24	6.17	48
A-4	6	—	60	5.00	5.41	3.24	6.17	55
A-5	6	—	150	9.50	8.54	8.10	9.76	60
A-6	6	—	290	9.00	11.88	15.66	13.57	85
A-7	6	—	290	16.00	11.88	15.66	13.57	102
A-8	6	—	180	5.50	9.36	9.72	10.69	69
B-1	6	1.68	130	10.00	10.05	10.51	12.88	110
B-2	6	1.68	180	11.50	11.46	13.21	14.48	58
B-3	6	1.68	130	13.25	10.05	10.51	12.88	78
B-4	6	1.68	90	7.50	8.72	8.35	11.35	66
B-5	6	1.68	120	8.50	9.74	9.97	12.52	65
B-6	6	1.68	120	3.50	9.74	9.97	12.52	78
B-7	6	1.68	120	9.00	9.74	9.97	12.52	—
B-8	6	1.68	180	12.50	11.46	13.21	14.48	126
C-1	6	4 71	45	8.90	10.57	12.23	15 97	137
C-2	6	4.71	60	12.00	11.24	13.04	16.80	148
C-3	6	4.71	50	11.40	10.82	12.56	16.26	152
C-4	6	4.71	90	13.30	12.51	14.66	18.19	155
D-1	3	4.71	90	8.90	9.20	12.23	14.41	66
D-2	3	4.71	90	11.00	9.26	12.23	14.41	—
D-3	3	4.71	120	8.80	9.71	13.04	14.99	—

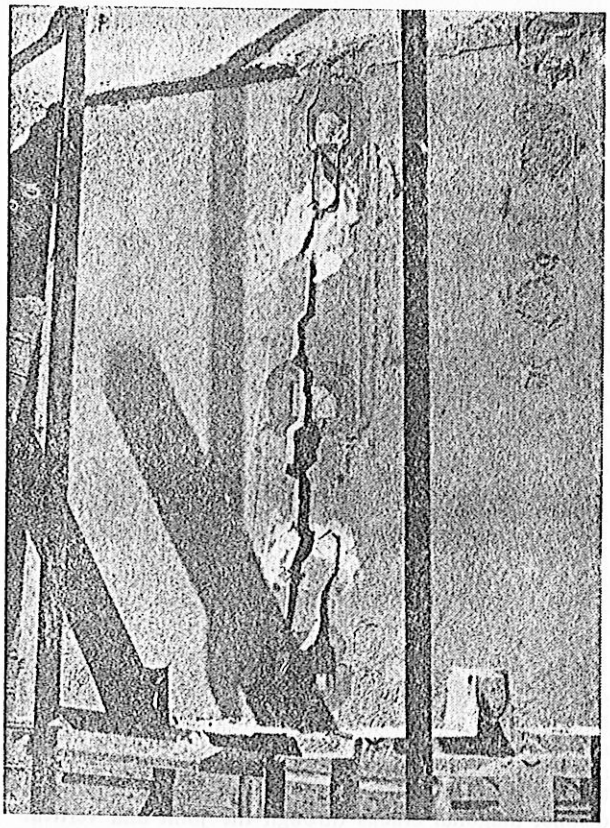

Fig 4 Typical crack pattern after failure of joint

stresses in the joints could be obtained since there were too many apparent contradictions in the results. However, in general, the plane of principal stress was found to be approximately at 45° to the vertical, which is to be expected in case of a pure shear problem. On analysis of the strain measurements of the rossettes, the general trend regarding distribution of shear stress along the height of the joint was that shear stress was minimum near the mid-height, and maximum near the ends; although in many cases this was contradicted. The strain readings might have the anomalies due to non-homogeneity of the surface, spalling of grains below the arms of the strain rossettes and due to the fact that some error in the reading of any of the three arms will affect considerably the magnitude and direction of principal and shear stresses.

From the readings of the strain gauges fixed to the reinforcement in the joint, it was found that maximum stress level in the reinforcement was of the order of 800 to 1200kg/cm². Hence, it is clear that the failure of the joint was not due to yielding of reinforcement. This was further verified from inspection of the reinforcement after failure of the joints. No sign of yielding of reinforcement could be noticed. It may be inferred that failure of reinforcement was due to loss of anchorage only. It was found that the stresses in the reinforcement were very small at the initial stages. However, with further loading, stresses in the reinforcement increased.

Participation of transverse reinforcement in transferring shear force may be by direct transfer of a part of the shear force or by transfer of the horizontal component of shear force as given by equation (1). Based on the above argument various investigators have suggested formulae for the load bearing capacity of vertical joint. All these formulae take the general form

$$P_f = k_c f_{sh} n a_k + k_s A_s f_s \tan\phi \qquad \ldots(2)$$

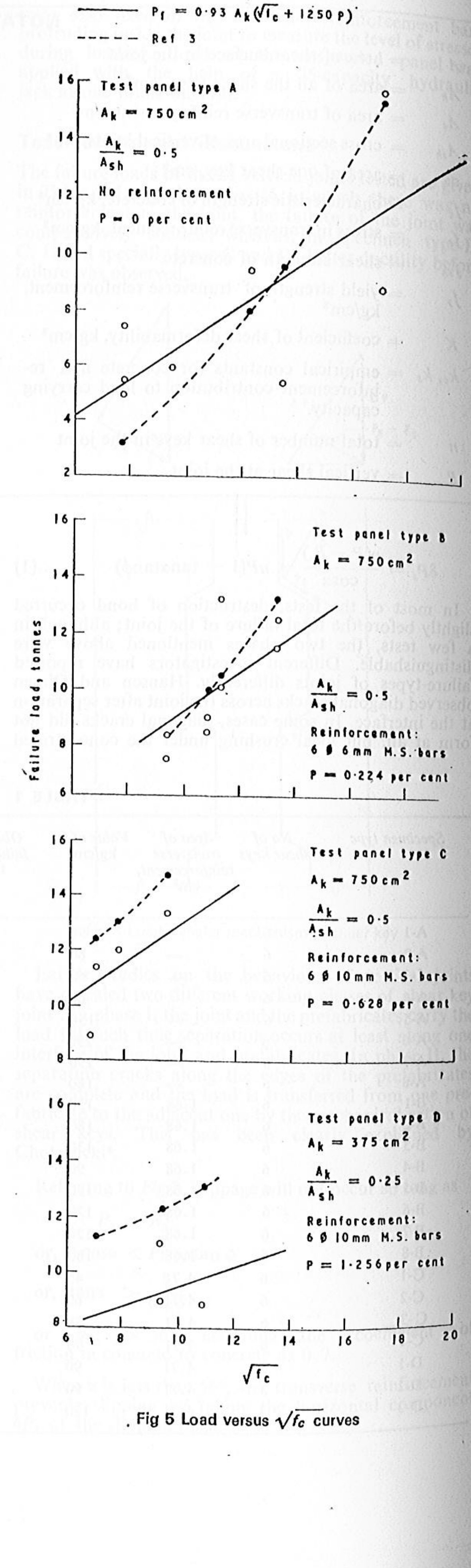

Fig 5 Load versus $\sqrt{f_c}$ curves

The first product of equation (2) denotes the contribution made by the area of shear key due to its shearing strength. The second product term corresponds to the load bearing capacity of reinforcement in shear, determined on the basis of the well known 'shear friction hypothesis' in which the effect of concrete is neglected and the same is compensated by overestimating the coefficient of friction, tan ϕ as 1.7[7].

The load bearing capacities obtained from the test results of joint types A and B have been plotted against $\sqrt{f_c}$ in *Fig* 5. A best fit curve by the least square method, which has been taken as a straight line, has been obtained from the experimental results and given by the following equation

$$P_f = 0.93A_k(\sqrt{f_c} + 1250p) \quad \ldots(3)$$

In *Fig* 5 are shown the plots of experimentally obtained load bearing capacity versus $\sqrt{f_c}$ for joint types C and D as well as equation (3). It can be seen that the experimental results confirmed satisfactorily with equation (3).

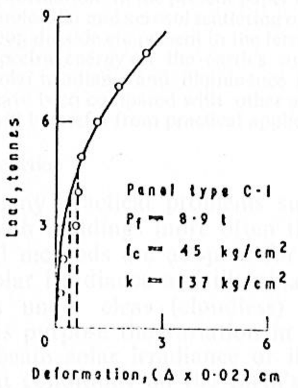

Fig 6 Load deformation curve

Cholewicki and Pommerette have also given some empirical formula based on test results and related the load carrying capacity with the area of transverse reinforcement and shear key and with the shearing strength of infill concrete[4]. It has been found the formulae given by them overestimates the experimental results obtained in the present investigation *Table* 1. Hansen and other workers have related the load carrying capacity with the characteristic srength of infill concrete and the area of transverse reinforcement and shear key[3]. The empirical formula suggested by them is as follows

$$P_f = 0.09A_k\sigma_c + A_sf_y \quad \ldots \quad (4)$$

subject to the condition that

$$0.01 < \frac{A_sf_y}{\sigma_c A} < 0.08; \ \sigma_y \not> 6000\text{kg/cm}^2; \text{ and } 0.2$$

$\frac{A_k}{A} < 0.5$ and height to depth of shear key $\not> 8$.

Equation (4) has been plotted in *Fig* 5, inspite of some of the conditions mentioned above not being complied with in a few test specimens of the present investigation. It can be seen that equation (4) gives a fair approximation of the load carrying capacity for joint types A, B, C, *Table* 1. However, for joint type D, equation (4) given by Hansen overestimates the experimental results, which is quite obvious when equations (3) and (4) are compared. It is noted that relatively heavier weightage has been assigned to the share of load by transverse reinforcement in equation (4).

The vertical deflection of the suspended panels under load relative to that of the fixed panel measured near the top end and near to the joint with the help of two dial gauges gives a measure of the coefficient of shear deformability of the joint which is defined as

$$K = \frac{\Delta\tau}{\delta\Delta}$$

A typical load versus shear deformation curve for joint C-1 has been plotted in *Fig* 9. The shear deformability of joint C-1 at about half the failure load is found to be 137kg/cm³. However, from the test programme, the value of K has been found to vary from about 50 to 150kg/cm³, *Table* 1. Although no definite correlation could be established between the coefficient of shear deformability and the characteristic strength of infill concrete, it was generally observed that transverse reinforcement in the joint increased the value of K. One would logically say that the coefficient of shear deformability should increase with higher strength of concrete, but the same was not established in the tests since in many cases of relatively higher strength concrete, shear deformability coefficient was low This may be due to the technological factors like finish of the edges of the panels, improper bonding at the interface of the joints, water cement ratio, shrinkage, etc, and also due to the limitation of the testing set-up. Thus technological factors play an important role in determining the shear deformability characteristics of veritical joint. Such technological imperfection may also lead to early initiation of cracks and loss of mechanical bond at the interface. However, it is felt that it may not influence adversely the load carrying capacity to a noticeable extent since failure load of joint is due to the shearing resistance of the infill concrete of the shear keys and the participation of the transverse reinforcement in resisting shear.

Conclusions

The observations made in the paper are based on testing of 23 specimens only and as such requires verification from further testing. The following conclusions can be drawn:

(*i*) Load carrying capacity of shear key joint depends mainly on the area of shear keys and transverse reinforcement and on the characteristic strength of infill concrete. Failure load of such joints can be predicted approximately by the formula given by equation (3), which is based on 23 tests.

(*ii*) The failure of the joints was due to loss of anchorage of the transverse reinforcement in the joint associated with diagonal cracking and vertical shearing of the keys of infilled concrete.

(*iii*) Joints with transverse reinforcement showed better ductility as compared to unreinforced joint.

(*iv*) Coefficient of shear deformability varied from 50 to 150kg/cm³ and was greatly influenced by technological factors like shrinkage and bond at the interface, water-cement ratio, surface finish at the edges of panels and no approximate correlation could be drawn between the value of k and the various parameters considered in this study.

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