BEHAVIOUR OF JOINTS IN PREFABRICATED SHEARWALLS FOR SEISMIC ZONES

by

A.R.SANTHAKUMAR [I], R.RADHAKRISHNAN [II], A.SWAMIDURAI [III], J.SIVAKUMAR [IV]

SUMMARY

With the aid of finite difference approximation of the laminar technique, the behaviour of the prefabricated shear wall is followed by an incremental trilinear elasto plastic analysis, which exposes the ductility requirements of the joints and exhibits their sequence of yielding. The behaviour of the three types of joints (ie. keyed welded joint, keyed joint with straight loop and keyed joint with inclined loop) when subjected to cyclic loading are examined in relation to their strength and serviceability characteristics.

INTRODUCTION

In reinforced concrete multistorey buildings shearwalls form one of the most economical means of providing lateral stability against wind or seismic loading. Coupled shearwalls are employed in monolithic multistoreyed buildings. In the report on the 'Response of building to lateral forces', the ACI-Committee 442 favours a cast-in-place construction because of its strength, stiffness and ductility. Comparison of the performance of the shearwall buildings and frame buildings during earthquakes shows that the latter had little damage to the frame but large distruss to the non-structural items, while the former had a few structural cracks but no non-structural damage. The benefits of well designed and detailed cast-in-site structures could be realised from the pre-fabricated shearwalls, if the desirable qualities of strength, stiffness and ductility of these could be ensured. This concept would make the designer utilise the enormous reservoir of strength possessed by the integrated wall panels.

NON-LINEAR ANALYSIS OF PRE-FAB SHEAR WALL

A stepwise plastification of vertical joints between the pre-fabricates of the shearwall was assumed, and incremental non-linear laminar analysis using the finite difference approximation was used. It was further assumed that the horizontal

---

I Assistant Professor in Structural Engineering, College of Engineering, PAUT, Guindy, Madras, India.
II Professor of Civil Engineering, I.I.T., Madras, India.
III Lecturer, PAUT, Guindy, Madras, India.
IV Research Assistant, PAUT, Guindy, Madras, India.

joints are designed with sufficient strength so that they do not reach their ultimate capacity till almost all the vertical joints are strained well beyond their yield limit. Such a sequence of plastification (1) of joints is desirable as the designer would wish to protect the wall against the permanent damage because plastification of horizontal joints would result in irrepairable misalignment of the building. During severe ground motions the attainment of ultimate load and the subsequent elasto-plastic energy absorbing deformations are realities. Therefore it is necessary to quantify the magnitudes of post-elastic deformation in the joints during the various stages of plastification of the structure. This would indicate the joints which are likely to suffer the maximum damage during catastrophic earthquakes. The history of the pre-fabricated shearwall's response is followed through incremental loading till the ultimate load is reached and the required overall ductility is attained. Such a study as indicated in Fig.1 shows the order of ductilities which are desirable if survival during major earthquake is to be assured. The following section discusses the details of an experimental investigation on prefab joints.

JOINTS BETWEEN PRECAST PANELS

When the loads and actions to be resisted can be estimated, joints can be designed on a rational basis, using basic mechanics of structure approaches. A number of formulae (2,3) for computing the strength of vertical joints have been developed based on actual tests conducted on joint assemblies. A study (4) of the equations by various codes and authors shows that a generalised equation can be suggested for calculating the shear strength of joint as shown in Table 1.

EXPERIMENTAL INVESTIGATION

Types of Joint: Three types of joints ie. welded joint, straight looped joint and inclined looped joint were tested. Fig.2 shows the arrangement of reinforcement of welded, straight looped and inclined looped joints. The reinforcement pattern was designed in such a way as to suppress any failure in the panel portion. Since the welded joint may pose constructional difficulties and are likely to prove uneconomical, the other two types have also been studied.

Joining the Panels: In the welded joint the reinforcement in the panels were joined together by butt welding. The width of the joint was 5 cm. The gap between the panels was concreted after placing one 6 mm diameter stirrup around the welds. In straight looped joint the panels were placed side by side and the transverse reinforcements were tied by means of loops. In the diagonal looped joint the transverse reinforcements were connected together by diagonal loops.

Loading System: Fig.3 shows the general arrangement of the loading system adopted. The arrangement was such that the centre line of the point of application of load and joint centre line coincide. Dial gauges were used to measure the deformations. The above set up was to produce pure shear condition in the joint.

Simulation of Reversed Cyclic Loading: Necessary provision was made in the test set up for application of reversed cyclic loading such as would occur during earthquakes. The reversal of shear in such a case is also shown in Fig.3.

Tests on Joints: Tests were carried on quarter scale model prefab wall panels to ascertain their stiffness and load carrying capacities. The load deformation and stiffness characteristics of the joints tested are shown in Fig.4 and Fig.5.

DISCUSSION OF TEST RESULTS

Welded Joint: The joint 1.0 (See Fig.2) was subjected to four cycles of loading. In the first and second cycle, joint was subjected to a maximum load of 0.189 Pu during which no crack was observed. Hence in the third cycle a load of 0.239 Pu and in fourth cycle a load of 0.315 Pu were applied. The joint failed due to crushing of support points. The first two stages (elastic and elasto plastic) are not distinguishable and Pu was found to be 37 tonnes. The premature failure observed during the test was because of stress concentration. Since the support got crushed in specimen 1.0, plywood and hard board were used to avoid stress concentration. The joint 1.1 was taken to 11 cycles of loading. A maximum load of 0.397 Pu was applied during the 7th cycle. Though a line of separation occurred between the joint and wall panel, full yield could not be reached due to inadequate jack capacity. On removal of load the separation cracks closed. For calculating the joint strength of specimens 1.0 and 1.1 a value of $\alpha = 1$ and $\beta = 0.7$ were adopted. (Refer Table 1)

Straight Looped Joint: The specimen 2.0 was subjected to a load of 0.96 Pu in the first cycle, when diagonal cracks were noticed. In the second cycle when the load was reversed, the joint was able to resist only 0.435 Pu. Ultimate load of this joint works out to 11.7 tonnes. Observed ultimate load was 11.2 tonnes which is in agreement with the theoretical formulae. The specimen 2.1 was loaded in increments of 1 tonne. A maximum load of 0.998 Pu was applied when the specimen reached its ultimate capacity. When the joint was subjected to reversed cyclic loading during cycles 2,3 and 4 the maximum load was 0.718 Pu, 0.678 Pu and 0.329 Pu respectively. There was visible sliding of panels. The model resisted a load of 0.998 Pu.

Inclined Loop Joint: The specimen 3.0 was subjected to six cycles of loading. A maximum load of 1.05 Pu was applied in cycle 3. This was the ultimate load. Specimen was able to take only 0.72 Pu, 0.64 Pu and 0.389 Pu in cycles 4,5 and 6 respectively. The joint was not subjected to any further cycle of loading, since the capacity has decreased considerably. For calculating theoretical ultimate load a value $\alpha = 1$ and $\beta = 0$ were used. $\beta = 0$ was used to represent the failure of the loops. Ultimate load of the joint was 8.4 tonnes. The excess load of 0.05 Pu taken by the joint is due to the contribution of inclined loop steel. The inclined loop steel has not contributed substantially for the strength of the joint. However the inclined loops have helped in providing ductility and delaying failure. Specimen 3.1 was able to resist only two cycles of loading. In the first cycle the joint resisted 0.639 Pu. At 0.33 Pu during the next cycle, the two panels started separating off with very large deformations. It was noticed that the loops gave way. The contribution due to loop steel was negligible. This requires attention. The predicted ultimate load and actual observed load during test are compared in Table 2.

CONCLUSION

1. The non-linear analysis shows that the joints may have to be designed to undergo large deformations if survival during earthquakes are to be assured. The ductility demand for the joints may be of the order of 10 to 20.
2. Welded joints perform satisfactorily but may pose constructional difficulties.
3. The general equation suggested in Table 1 corelates well with the observed results of tests.
4. The inclined loop joints provide for considerable post elastic deformation but their strength characteristics needs further research and careful attention.
5. In general tests show that joints can be designed and detailed to perform satisfactorily.

ACKNOWLEDGEMENT

The above work is a part of the research scheme funded by Council of Scientific and Industrial Research, New Delhi, India. The financial assistance received from them is acknowledged. The experimental work formed part of R.Sivaswamy's graduate research and his contribution to this continuing research scheme is acknowledged.

REFERENCES

1. Paulay, T. and Santhakumar, A.R., (1976) Ductile behaviour of Coupled Shear Walls, Proceedings of Structural Division, ASCE, Vol.102, ST 1, pp.93-108.
2. Cholewicki, A., (1971) Load bearing capacity and Deformability of vertical joints in structural walls of large panel building, Building Science, Vol.6, pp.163-184.
3. Hansen, K. and Olsan, S.C., (1973) Design of vertical shear keyed joints in large panel buildings, Report presented at the meeting of CIIB-W2 3.A, Copenhagen, 120p.
4. Santhakumar, A.R., Swamidurai, A. and Sivakumar, J., (1978) Prefabricated Shear Walls, Report II, Council of Scientific and Industrial Research, New Delhi, India, 132p.

TABLE 1 STRENGTH OF JOINT

$$V_u = \alpha f'_c A_k + \beta A_s f_y$$

Where α and β are coefficients which vary for various authors and codes as shown

f'_c Compressive strength of concrete

f_y Yield strength of steel

A_k Total area of keys in joint

A_s Area of reinforcement in the joint

Code or Author	α	β
Cholewicki (unreinforced)	0.7	0
Cholewicki (reinforced)	1.80	1
Hansen and Olsan	0.08	1.1
Polish Code (unreinforced)	0.65	0
Polish Code (reinforced)	2.00	0
Structural Engineering Research Centre, India.	1.20	1.0

TABLE 2 COMPARISON OF ULTIMATE LOADS

S.No.	Type of joint	Theoretical (tonnes)	Actual (tonnes)	Remarks
1.	Welded	37.0 (α = 1, β = 0.7)	14.7	Full value of Pu could not be obtained for lack of jack capacity
2.	Straight looped	11.7 (α = 1, β = 0.7)	11.6	Good
3.	Inclined looped	8.4 (α = 1, β = 0)	8.87	Higher actual value is due to contribution of steel

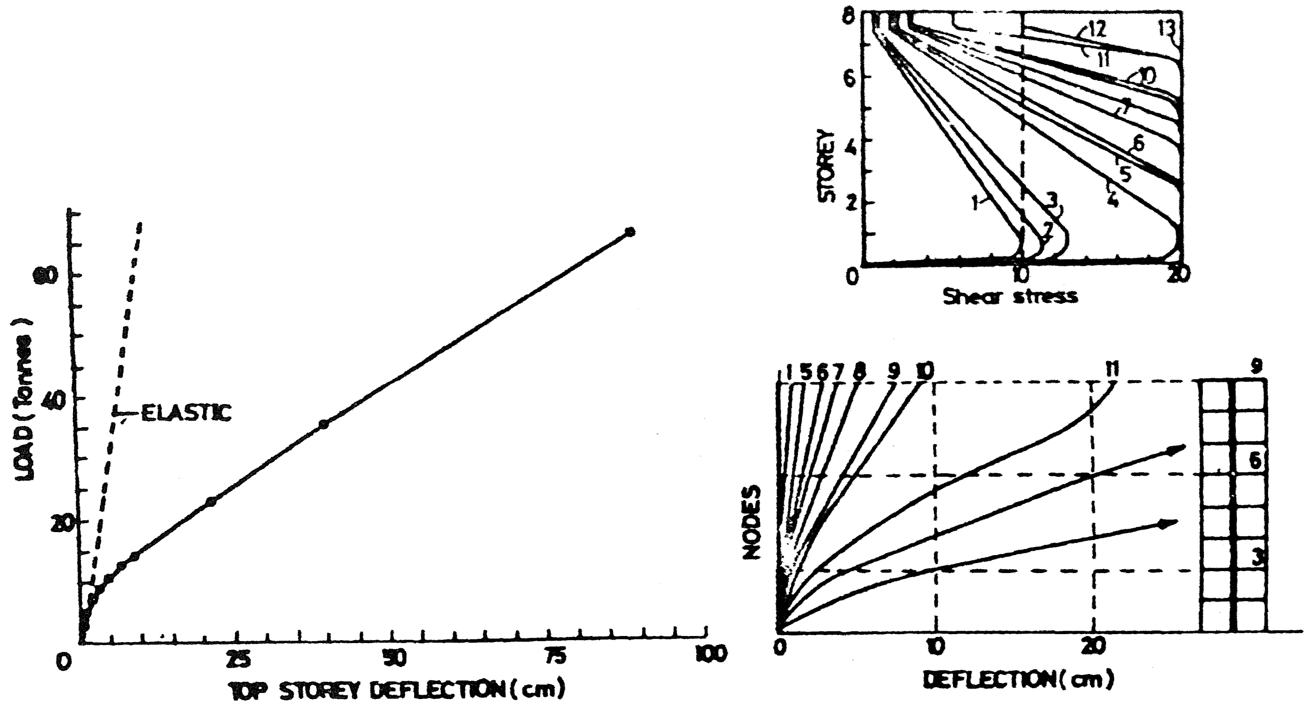
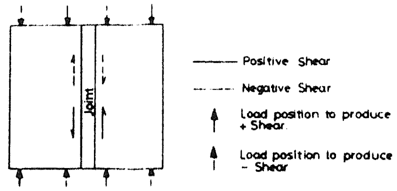


FIG.1 RESPONSE OF PRE-FAB. SHEAR WALL



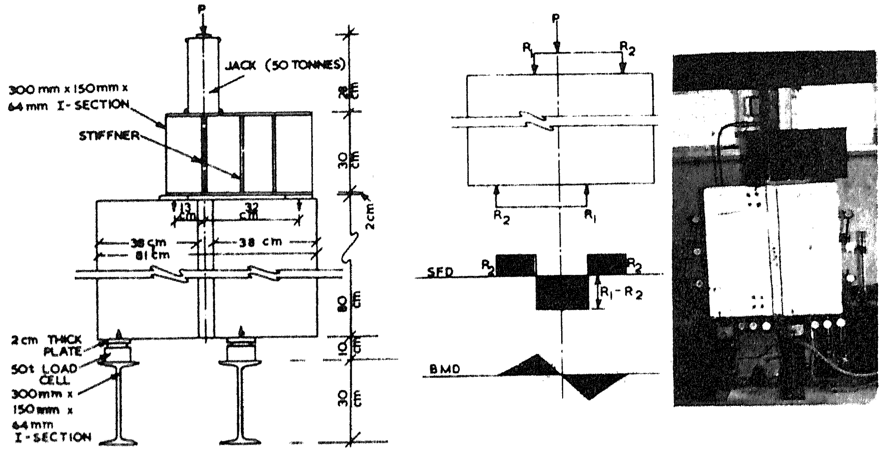


FIG. 3 LOADING SYSTEM

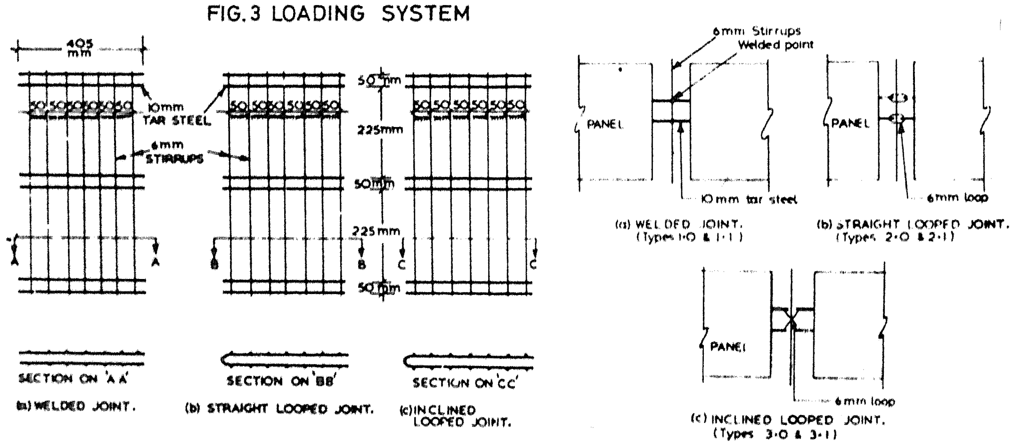


FIG. 2 REINFORCEMENTS IN JOINTS

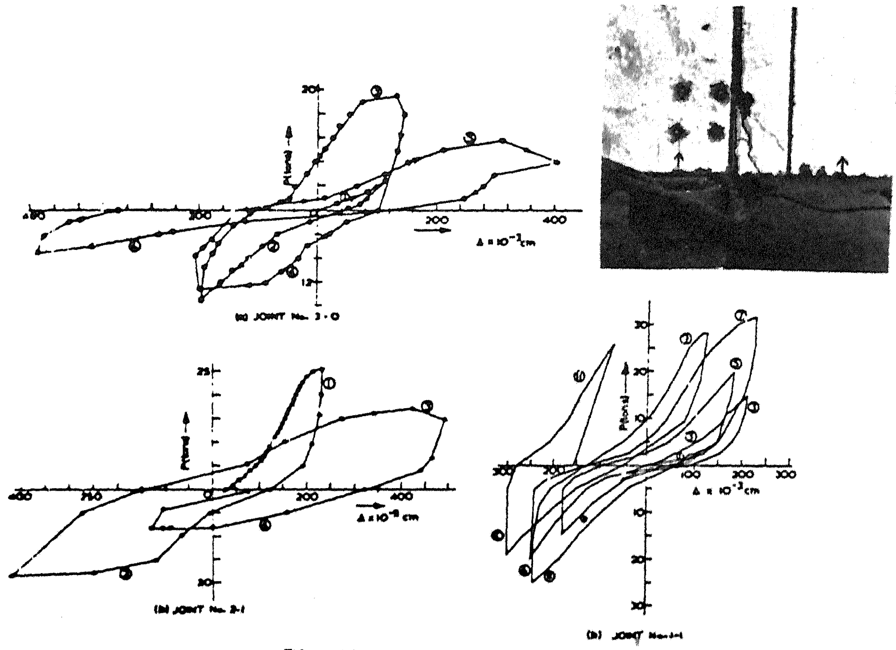


FIG. 4 LOAD-DEFLECTION BEHAVIOUR

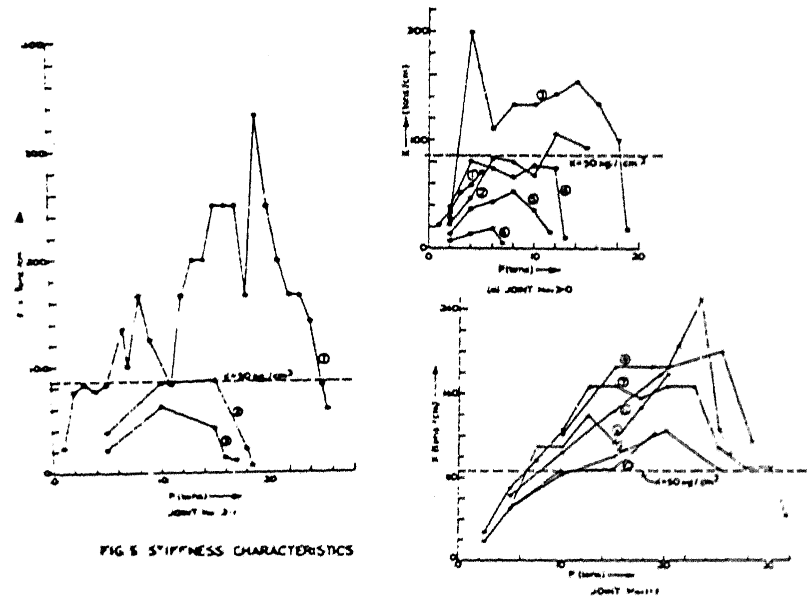


FIG 5 STIFFNESS CHARACTERISTICS