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PUNCHING STRENGTH DECAY OF SLAB-COLUMN CONNECTIONS UNDER SEISMIC LOADING

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ABSTRACT

An experimental investigation of the punching shear behavior of internal slab-column joints under seismic conditions is presented. Three large scale slab-column subassemblies were tested. The primary variable was the amount and the type of punching shear reinforcement. The enhanced ductility and energy dissipation capacity and the improved failure type was a result of the shear reinforcement.

KEYWORDS

Columns (supports); connections; cyclic loads; ductility; flat concrete plates; flat concrete slabs; punching shear; reinforced concrete; stiffness; strength.

INTRODUCTION

When concentrated axial loads act in small slab's regions it is possible to be observed early punching shear failure before slab develop its maximum flexural strength. Punching shear failure is a brittle sudden and catastrophic type of failure and is more dangerous than the other modes such as the flexural and the torsional type. This type of failure which is usually the source of collapse of flat plate and flat slab buildings, exhibits two main features, the vertical deflection and the truncated conic failure surface.

Flat slab buildings have many advantages such as : (1)simple and cheap form work, (2)easy placement of flexural reinforcement, (3)minimum obstruction to utility and duct placement, (4)minimal unusable space is created between stories, (5)capability of developing economical procedures of prestress with which the greater spans are faced. The last results in decreasing of the slab's depth and in aesthetically pleasing lines.

The only disadvantage of flat slab buildings is their vulnerability in earthquake loadings. Thus, during an earthquake and after a topic punching shear failure there is danger of taking place progressive collapse. The performance of flat slab structures subjected to seismic loading has attracted increasing attention recently (Morrison et al., 1983; Moehle and Diebold, 1985; Robertson and Durrani, 1991, 1992). Considerable research has focused on the slab-column connections, culminating in the recently published recommendations by ACI-ASCE Committee 352 (1988). In this paper an effort is presented to improve the punching shear behavior of slab-column subassemblages during earthquake type loading.

TESTING PROGRAM

Specimens and variables

Three specimens were tested. The plate portion of each specimen was 120 mm thick and 1600 mm square; the column was 650 mm height and 200 mm square. The top and bottom reinforcement mats are shown in Fig. 1. Specimen L_1 had no shear reinforcement in the vicinity of the column, Specimen S_1 had 6 inclined 45 deg. bars with diameter 8 mm as slab shear reinforcement in the vicinity of the column Fig. 1(d) while in specimen F_1 steel fibres 50 mm in length were added to the concrete mix ($30 kg/m^3$), in this region.

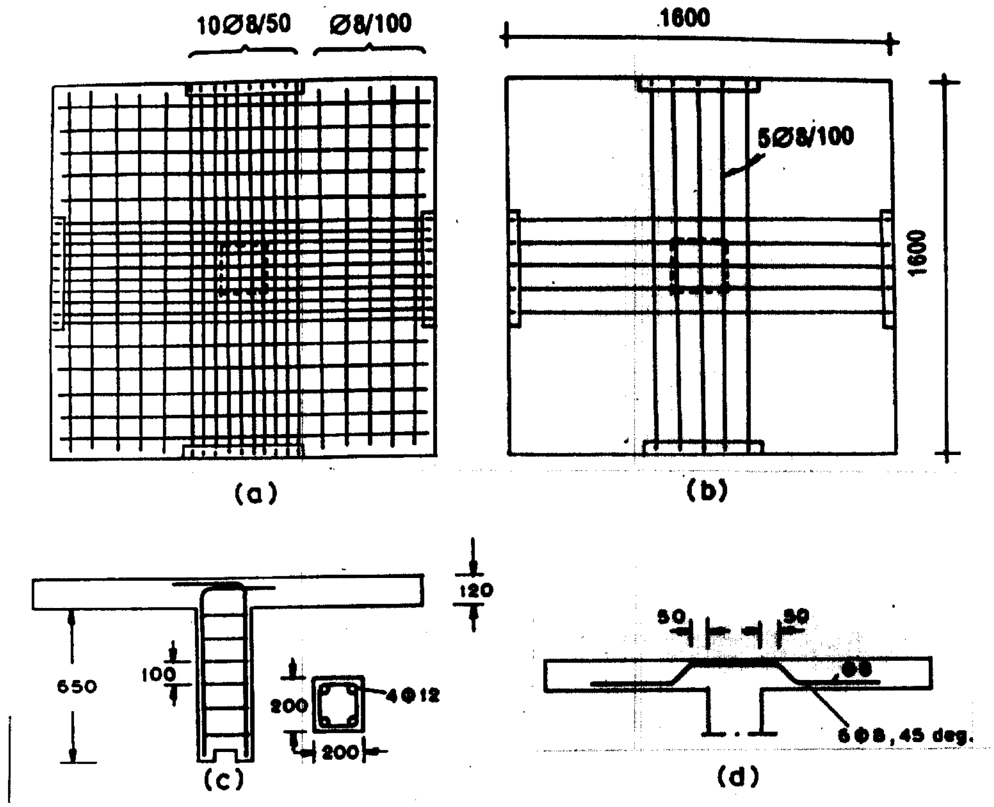


Fig. 1. (a)Top reinforcement mats of slab, (b)Bottom reinforcement mat of slab, (c)Reinforcing details of column and (d)Punching shear reinforcement of specimen S_1 (dimensions in mm)

Summary of specimens' steel yield stress, in MPa, bar size: Ø8=495, Ø10=465, Ø12=530. The concrete mix had 28-day compressive strength of 25 MPa, 25 MPa, 27 MPa for specimens L_1 , S_1 and F_1 respectively.

The ultimate load of specimen L_1 is computed according to Greek Code (1995) as follows:

$d = (d_x + d_y)/2 = 10$ cm

$u = 4c + \pi d = 4 \cdot 20 + 3.14 \cdot 10 = 111$ cm $= 1.11$ m

$\rho = 0.5/10 \cdot 5 = 0.01 =$ Ø8/5

$v_{Rd1} = 1.6 \cdot \tau_{Rd} \cdot k \cdot (1+50\rho) \cdot d$

$v_{Rd1} = 1.6 \cdot 300 \cdot (1.6 - 0.1) \cdot (1+50 \cdot 0.01) \cdot 0.10 = 108$ KN/m

$V_{Rd1} = u \cdot v_{Rd1} = 1.11 \cdot 108 = 120$ KN

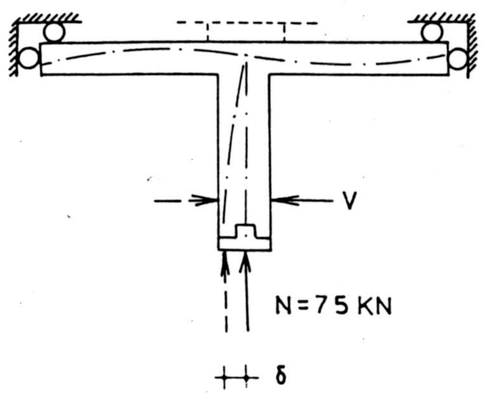
Suggesting that $q \cong 0.25g$ (usual slabs' load), the column axial load in the case of seismic action is

$$N_{(earthquake)} \cong 120 \frac{g + 0.39}{1.35g + 1.5q} \cong 75 \text{KN}$$

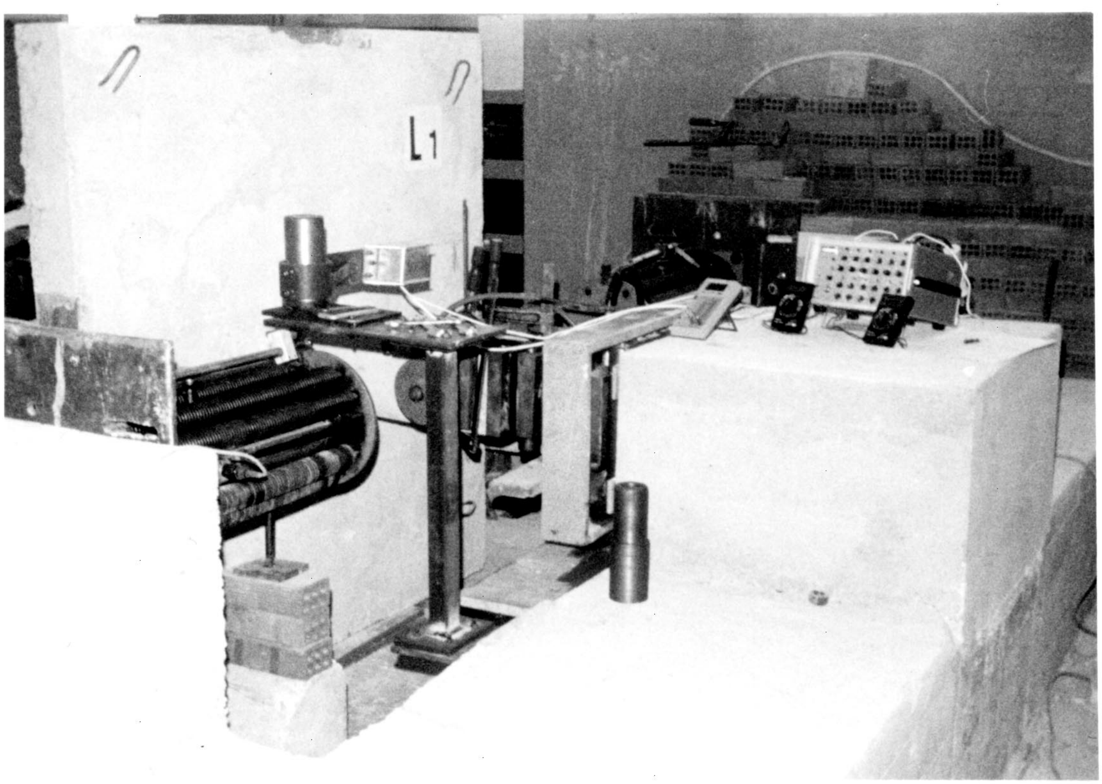
Specimens were loaded at first with an axial load of 75 KN and then were loaded by applying repeated lateral forces at the free end of the column.

Test setup

The structural frame assembly of experimental arrangement is shown in Fig. 2(a). The tests of the slab-column specimens were carried out in the testing frame shown in Fig. 2(b). Lateral loading was applied to the column's end by two one way actuators of 200 KN capacity. The load applied by the actuators was measured with two load cells attached to the specimens. The load point displacement was measured by a potentiometer. The axial load was imposed by a hydraulic jack (300 KN capacity). The axial load was kept constant and equal to 75 KN during the test.



(a)

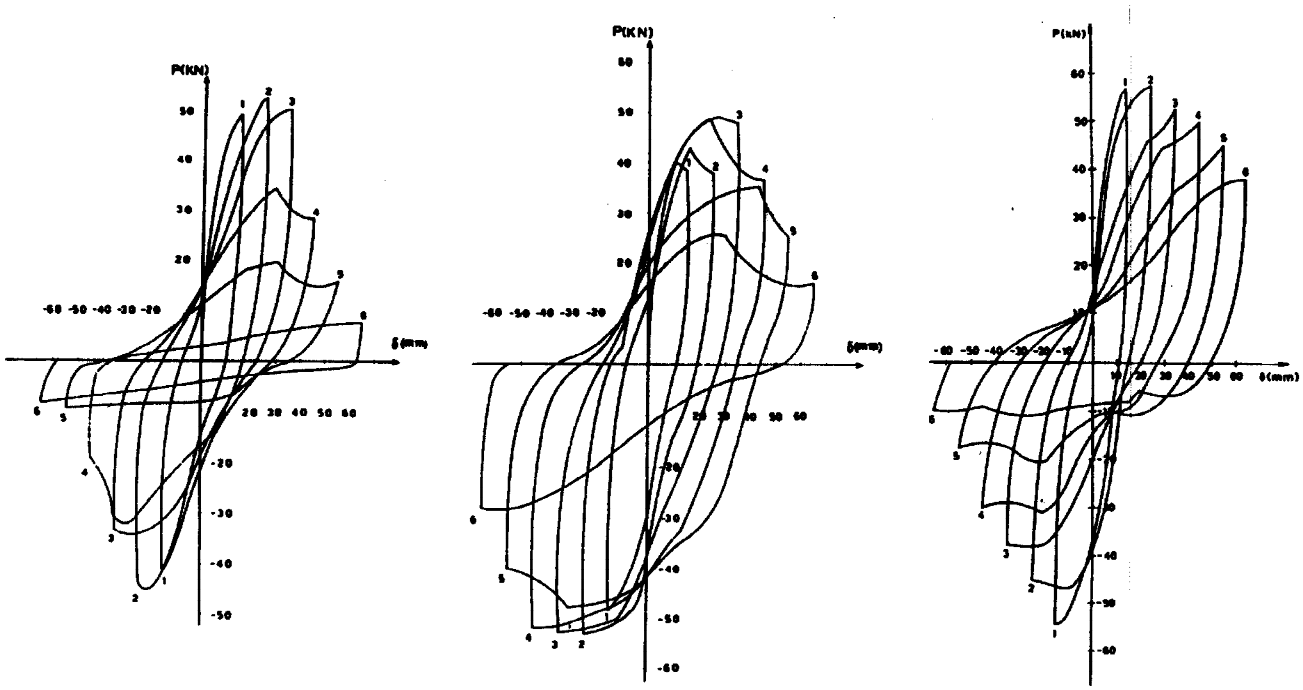


(b)

Fig. 2. (a)Structural frame assembly of experimental arrangement,
(b)General view of experimental arrangement

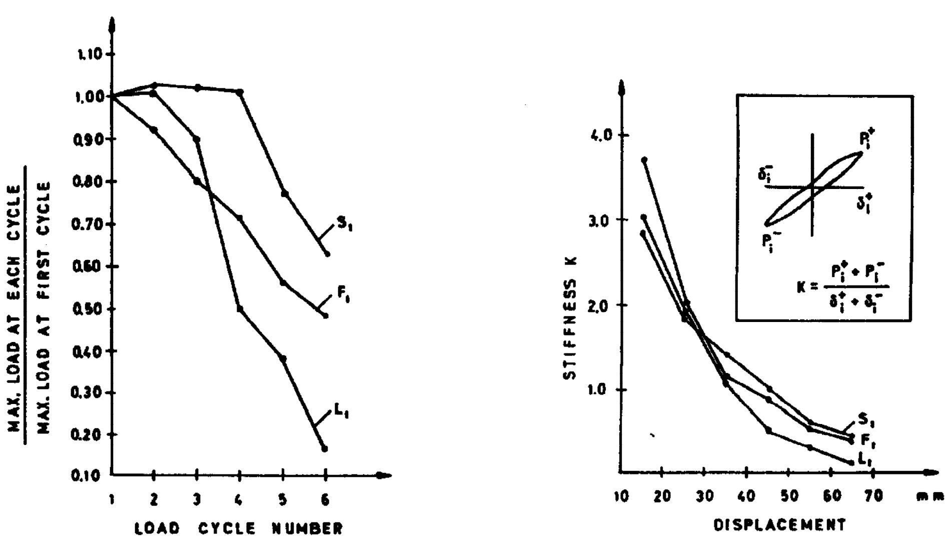
TEST RESULTS

Plots of the applied load versus the displacement of the load point for all specimens are shown in Fig. 3. The ratio of the maximum load carried by the specimens during each cycle of loading to that of the first cycle is shown in Fig. 4(a). The peak-to-peak stiffness for all specimens is shown in Fig. 4(b). The load carrying capacity for specimen L_1 was sharply reduced after the first cycle of loading while the reduction in load carrying capacity for specimen F_1 was lower and for specimen S_1 was not as severe Fig. 4(a). The stiffness of all the subassemblies decreased rapidly as the specimens was subjected to successive cycles of increasing displacement. Nevertheless, the stiffness degradation of specimen S_1 was lower than those of the other specimens L_1 and F_1 (Fig. 4(b)).



Specimen L_1 Specimen S_1 Specimen F_1

Fig. 3. Load versus deflection response for specimens L_1 , S_1 and F_1



(a) (b)

Fig. 4. (a)Cyclic load carrying capacity of specimens, (b)Connection stiffness-versus-displacement

Specimen L_1 failed in punching shear. This failure was sudden and catastrophic (Fig. 3, 4). Specimen's F_1 failure occurred due to punching of the slab by the column (Fig. 5). The presence of steel fibres in slab in the vicinity of the column resulted in a gradual failure. Thus, the steel fibres changed the type of failure from sudden and catastrophic to gradual and less brittle (Fig. 3, 4 and 5). During the first three cycles of loading specimen S_1 developed flexural cracks at the end of the column near the slab and in the slab. Subsequent two cycles (4th and 5th) resulted in an increase in the width of the flexural cracks in the column and in significant cracking and spalling of the slab concrete cover (in the bottom side) because of flexure. During the last cycle of loading at the 65 mm deflection non symmetrically punching shear cracks were observed in the top side of slab. Thus this type was classed as almost flexural failure (Fig. 3, 4 and 5).

The perimeter of truncated punch cone base is the critical perimeter according to Eurocode 2 (1991). It was observed for specimen L_1 that the distance between this critical perimeter and the loading surface was approximately equal to 3d. Thus, it was two times larger than the computed distance according to Eurocode 2 (1991) and three times higher than the computed distance according to Greek Code (1995).

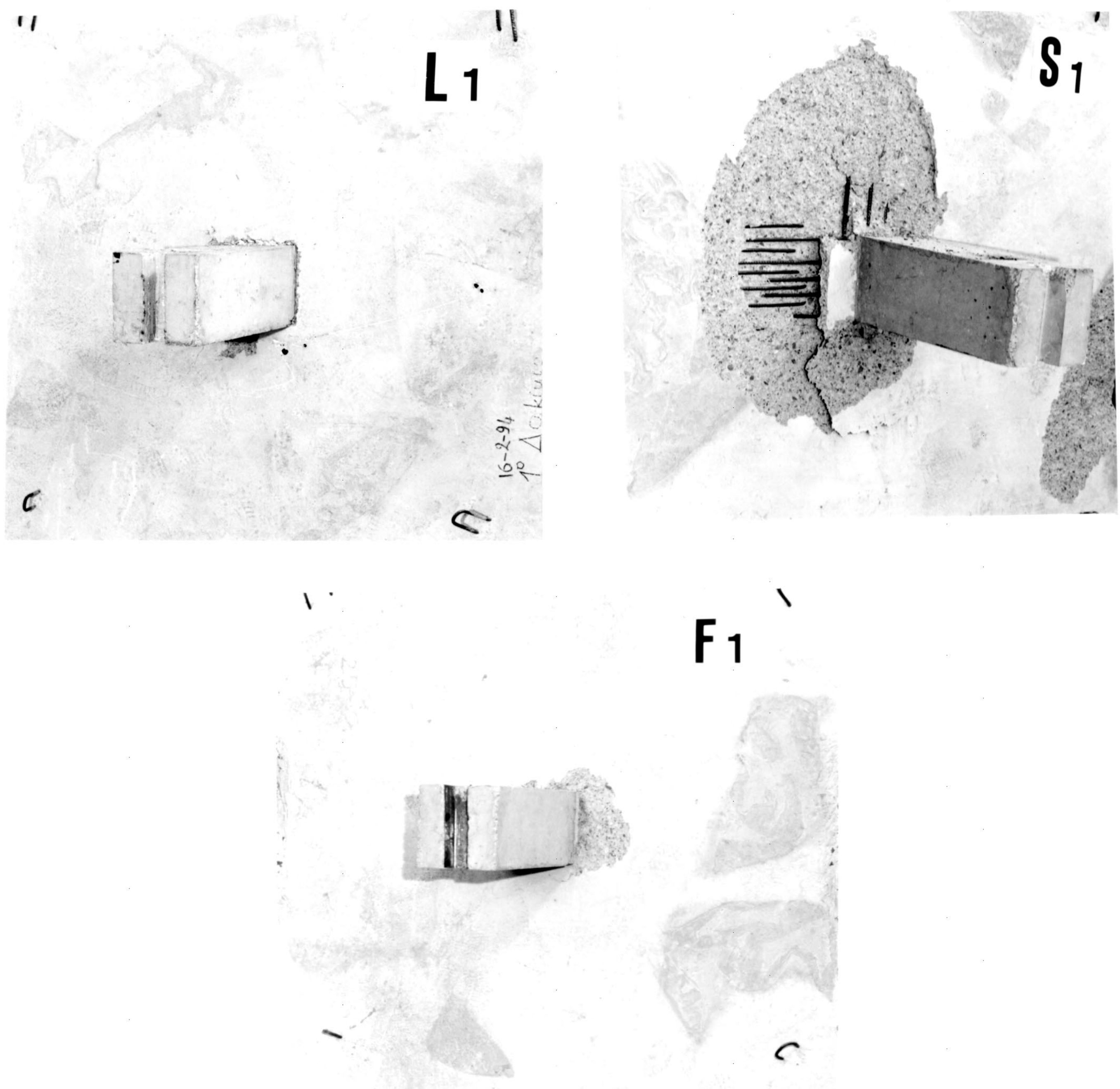


Fig. 5. Photographs of distress for all the specimens

CONCLUSIONS

Based on the test results described in this paper, the following conclusions were drawn.

1. The presence of punching shear reinforcement improves the seismic performance of slab-column subassemblages and reduces considerably the deterioration of the specimens after reaching their maximum capacity. This reinforcement resulted in changing of type of failure from punching shear failure, to almost flexural mode.

2. Slab-column connection with inclined 45 deg. bars as punching shear reinforcement performed considerably better than connection with steel fibres for increasing the shear capacity.

3. The inclined bars and the steel fibres increased the ductility and the energy dissipation capacity of the slab-column subassemblies.

4. A minimum shear reinforcement for the slab-column joints must be specified by the Codes for the design of earthquake resistant structures in order to avoid the brittle failure. The study reported here, is part of a larger experimental and analytical research program sponsored by the Laboratory of Reinforced Concrete Structures of Aristotle University of Thessaloniki. One of the purpose of this research is to explicit propose this minimum reinforcement percentage.

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