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Load carrying capacity of top slab reinforcement in punched flat-plate floors

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ABSTRACT

Flat-plate buildings can experience punching failure at connections during strong earthquakes or when subjected to gravity overload. Such failures occur when the slab-column connections do not have adequate shear resistance. Once the punching failure has occurred, the integrity of the floor system depends entirely on the load carrying capacity of the slab reinforcement continuous through the column. As a defense against progressive collapse, the present seismic building codes require slab bottom reinforcement to be continuous through the columns, thus acting as hanger bars in the event of failure. Many existing flat-slab buildings do not have such reinforcement detail and thus rely entirely on the capacity of top slab reinforcement to carry the gravity load should a punching failure occur. This paper presents the results of an experimental investigation to determine the load carrying capacity of the top slab reinforcement in punched flat-plate floors. Results indicate that with a 3 in. spacing the top slab reinforcement was able to carry the load corresponding to one third of the total theoretical punching capacity of the slab-column connection while the top rebars spaced at 6 in. were able to support one half of the punching load. Increasing the length of embedment and the size of the bars increased the load carrying capacity by a small margin.

INTRODUCTION

The current design procedures for flat-plate buildings ensure sufficient margin of safety against punching of slab-column connections under normal service load conditions. Therefore, very little research has been done to evaluate the load carrying capacity of the top slab reinforcement for punched conditions. It is believed that following punching failure, the top slab reinforcement immediately rips through the slab cover and becomes ineffective in carrying the load (Mitchell and Cook 1984). Recent concerns over the safety of older, non-ductile flat-plate buildings under seismic loadings have motivated the need for evaluating the capacity of connections to sustain the gravity loads after punching has occurred.

For gravity load design the ACI 318 Building Code (1989) requires minimum extensions into adjacent spans for top slab reinforcement in slab-column frame systems. For a column strip without drop panels using straight

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bars with a minimum of 50% of the bottom reinforcement continuous into the column, the suggested minimum bar length from the face of the column for the top reinforcement is 0.30 of the clear span. This minimum length is required to provide adequate development under design loads in order to carry the negative bending moment produced at the slab-column connection. For seismic loading the ACI Committee 352 (1989) recommends certain minimum continuous bottom reinforcement through the column in each principal direction in order to maintain the integrity of the floor system.

TESTING PROGRAM

The primary objective of this research was to investigate the load carrying capacity of the top slab reinforcement at slab-column connections following a punching failure. To achieve this objective, three variables were chosen on the basis of common design details found in flat-plate floors. Because the thickness of the slab and the amount of cover used are standard in most systems, the bar size, the spacing, and the length of the bar were chosen as variables. Eight half-scale slab-column concrete specimens were tested during this investigation. Table 1 lists the specimen designations and the respective reinforcement details for each specimen.

Table 1. Specimen details

Specimen	Bar Size	Bar Length (Fraction of Clear Span)	Center-to-Center Spacing*
B3L3S3	No. 3	0.30	3 in.
B3L3S6	No. 3	0.30	6 in.
B3L4S3	No. 3	0.40	3 in.
B3L4S6	No. 3	0.40	6 in.
B4L3S3	No. 4	0.30	3 in.
B4L3S6	No. 4	0.30	6 in.
B4L4S3	No. 4	0.40	3 in.
B4L4S6	No. 4	0.40	6 in.

* 1 in. = 25.4 mm

The cover and spacing were kept constant by tying each steel bar to four 0.75 in. plastic chairs. Two well greased metal plates 6.5 in. deep, placed

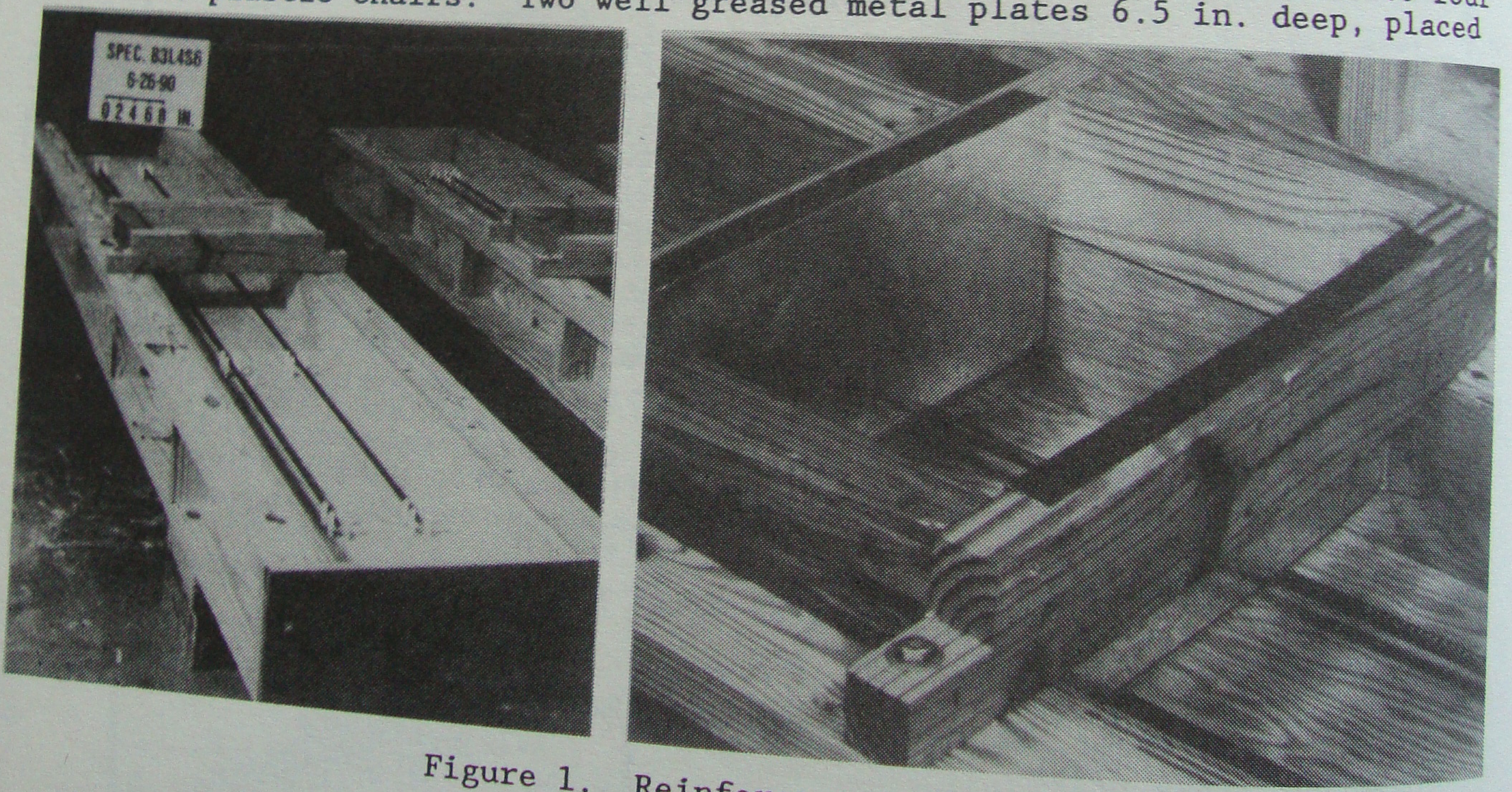

Figure 1. Reinforcement detail

back-to-back, were inserted vertically on each side of the column faces to simulate partially pre-cracked punching failure. Approximately 1.5 in. of concrete was left intact at each column face (see Fig. 1). Two hooks were embedded in each specimen about 2.5 in. in the concrete and about 2 ft. from each end to allow easy maneuverability in and out of the test machine.

All specimens were cast simultaneously using ready mixed concrete of 3000 psi (20.7 MPa) specified compressive strength at 28 days. Furthermore, high early strength cement was used to be able to test the specimens over the short duration of the project. Control cylinders (6 in. x 12 in.) were tested on each day a specimen was tested to determine the concrete compressive strength. In addition, two beams were tested to determine the modulus of rupture of the concrete.

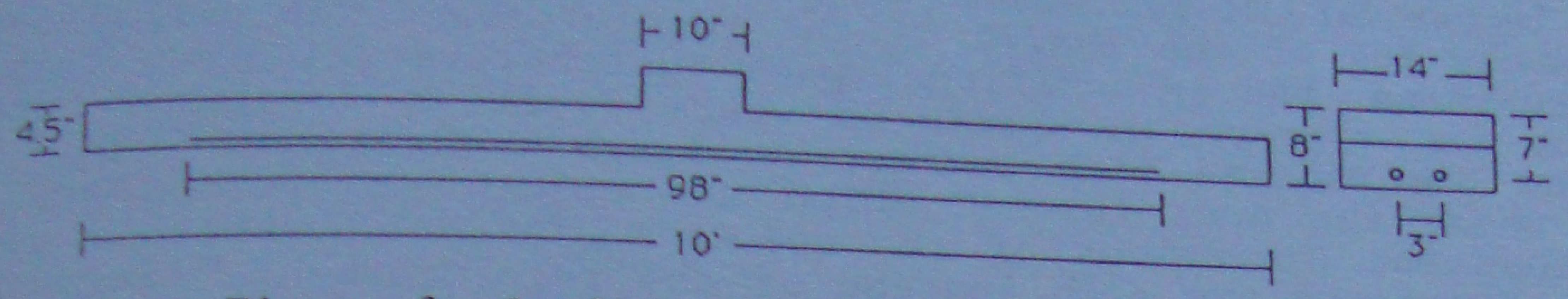

Figure 2. Configuration of Specimen B4L4S3

Instead of testing the specimens with full slab width, it was considered appropriate to model only the strip of slab in the column region since only the reinforcement passing through the column affected the load carrying capacity of the connection (Fig.2). Each specimen was tested in an inverted position to facilitate load application. A general test set up is shown in Fig. 3. The specimens were tested under displacement control to monitor the

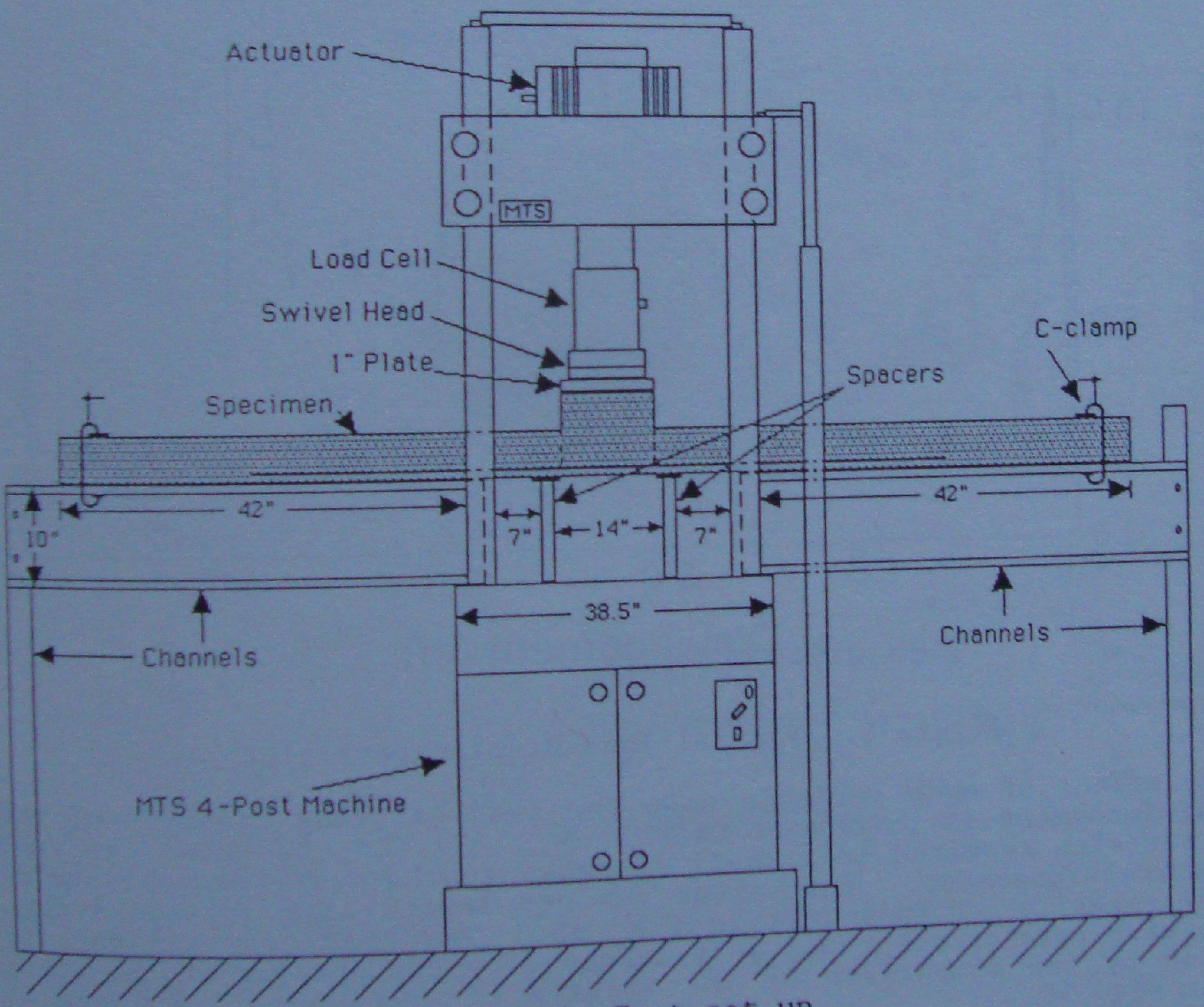

Figure 3. Test set up

progressive pull-out of reinforcement through the slab cover. The displacement was applied at a constant rate while a computer automatically recorded both the load and the deflection. Visual observations at appropriate stages of the ripping of the slab reinforcement were made during each test while the displacement was held steady.

TEST RESULTS

A typical load-deflection plot as recorded during the test is presented in Fig. 4. All of the load-deflection plots show a common trend: an initial peak during which the column punched through the remaining intact concrete followed by a drop and a gradual increase in load, peaking after several inches of displacement with a sudden loss of resistance after approximately 75% of the length of the reinforcement was pulled out. Some of the specimens, had an additional peak just before failure. This peak is extraneous and appears in specimens where friction developed between the column and the punched slab surface at large deflections. This friction occurred where the area through which the column was pushed was slightly smaller than the dimensions of the column. The load was thus carried by both the column and the slab instead of the reinforcement. Because of the great irregularities of its data due to excessive friction, the results for test B4L4S3 are not included in computations and comparisons.

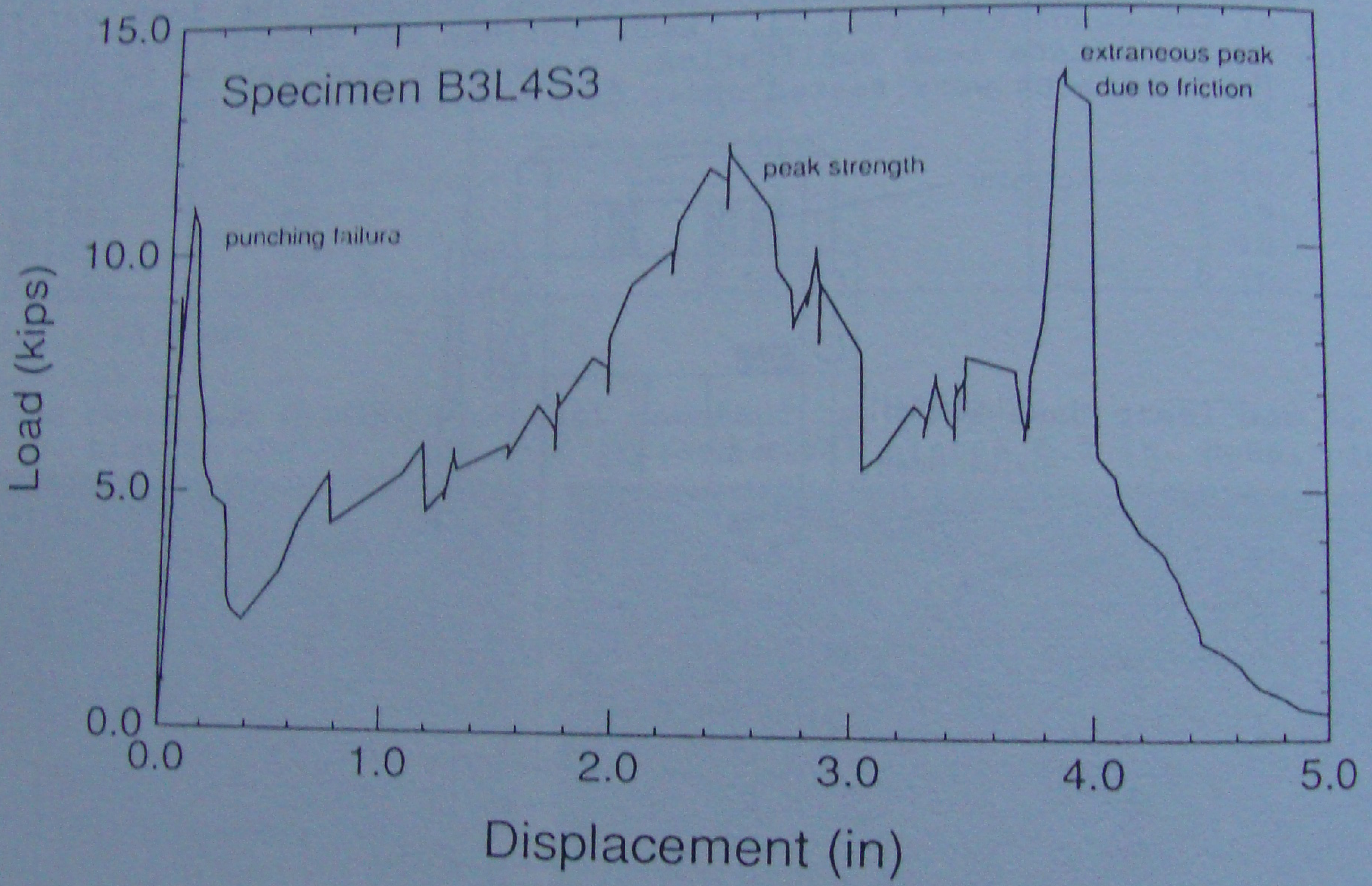

Figure 4. Typical load-displacement plot

The effect of each variable upon the load carrying capacity of the top slab reinforcement is presented in Fig. 5 . As Fig. 5a shows, an increase in the bar size resulted in a small increase in the carrying capacity of the top slab reinforcement. Increasing the bar length also increased the load carrying capacity as indicated in Fig. 5b. The load carrying capacity of the top slab reinforcement was most affected by the spacing of the rebars as indicated by the comparison of the results of specimens B4L3S6 and B4L3S3 shown in Fig. 5c.

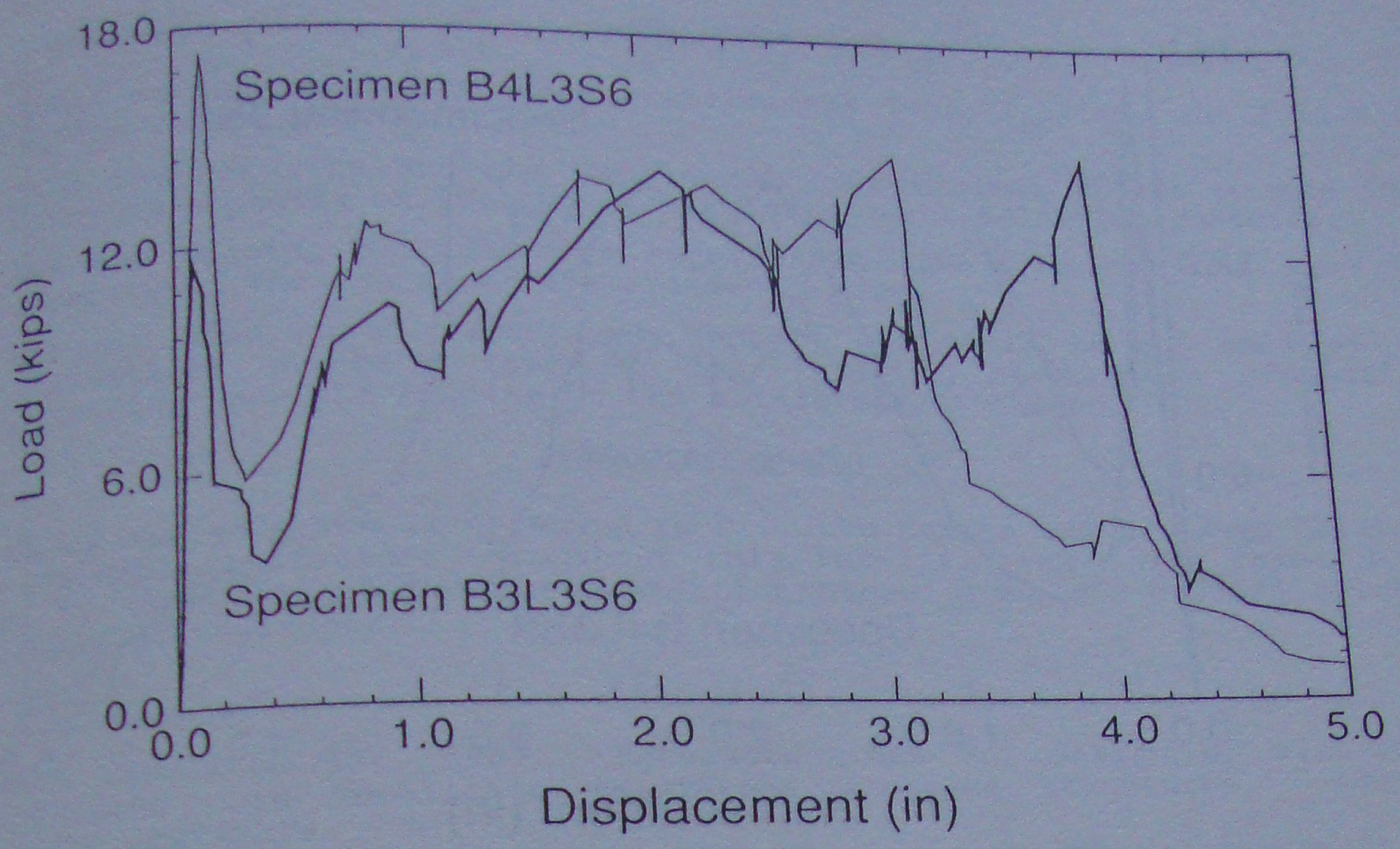

Figure 5a. The effect of bar size

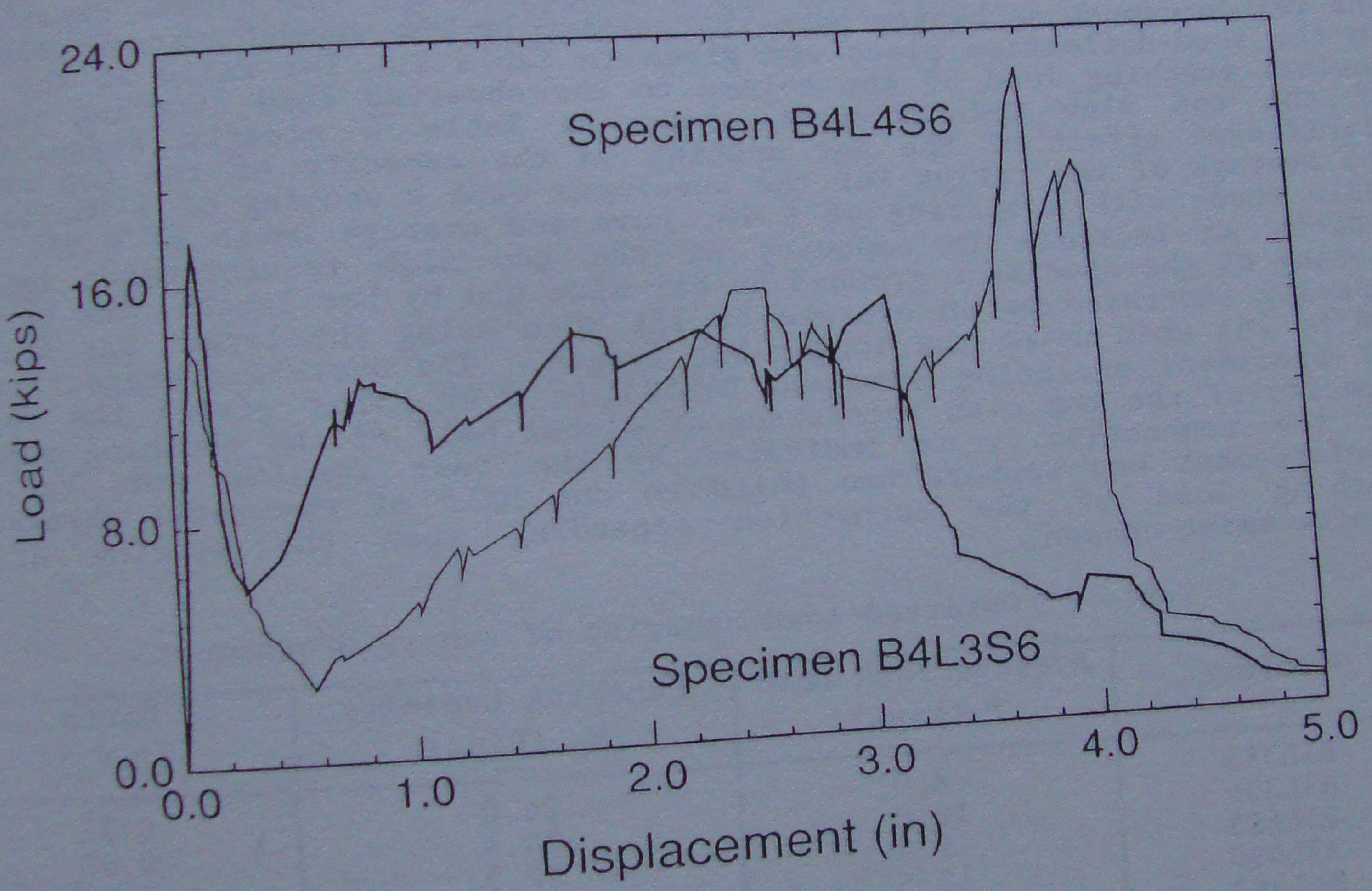

Figure 5b. The effect of bar length

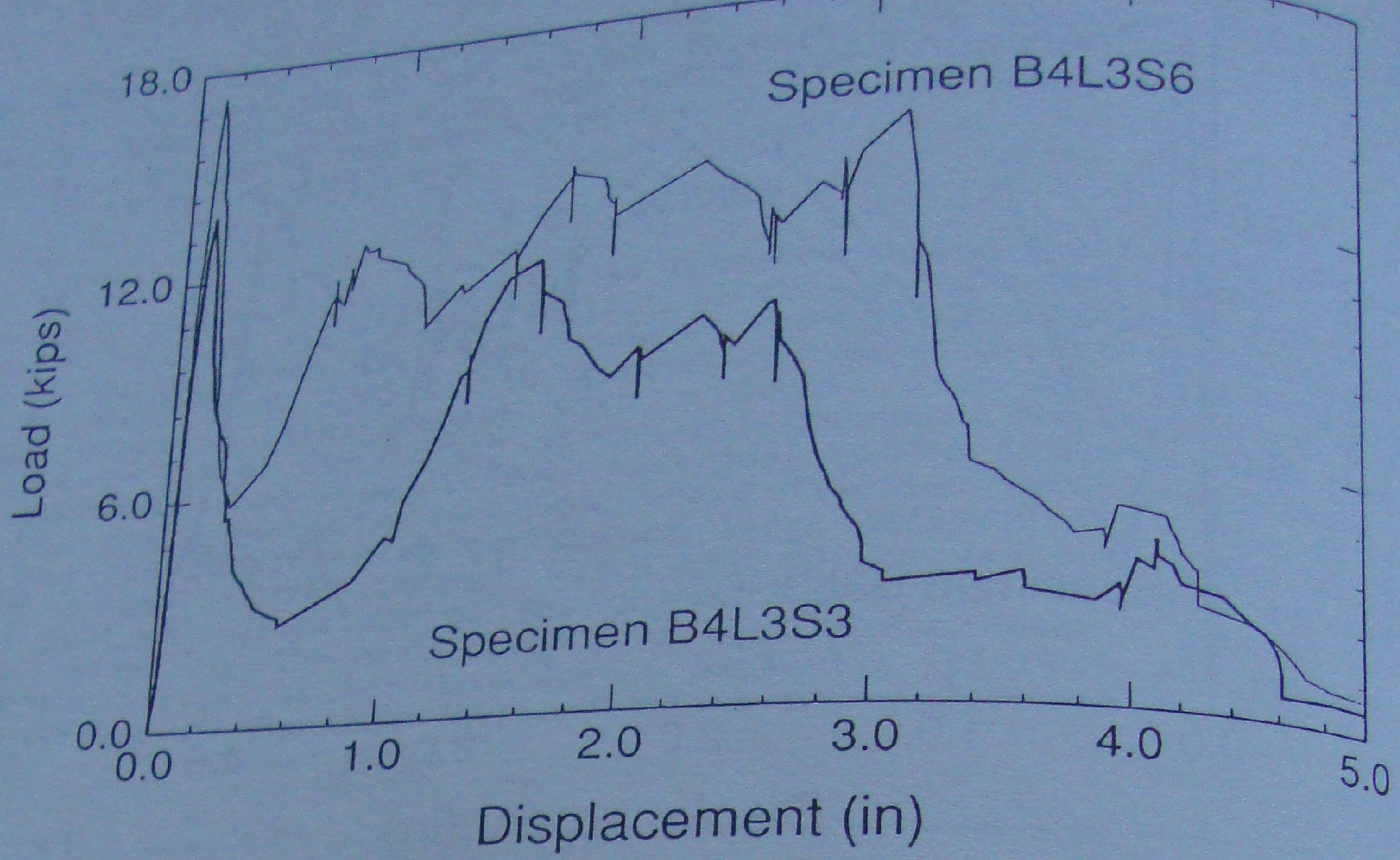

Figure 5c. The effect of bar spacing

The theoretical punching strength of each slab-column connection calculated using the ACI Code (1989) procedure and the actual capacity of the reinforcement as hanger bars represented by the second average peak load in the load-deflection plots are given in Table 2. The ratio of the theoretical punching load of the column to the observed load carrying capacity of the top slab reinforcement shown in Table 2 clearly suggests the significant effect of the bar spacing on the capacity of the top rebars. The average of the ratios for the specimens with a spacing of 3 in. is 0.38 while those with a spacing of 6 in. have and average ratio of 0.55. This depicts an increase in capacity of the top slab reinforcement by 43%. Looking at the specimens grouped by bar size and by bar length, the carrying capacity increased by an average of 13% when using the larger bar diameter and by 16% when using the longer bar length. The overall average ratio of all specimens excluding specimen B4L5S3 is 0.48, thus giving the average capacity of the top slab reinforcement almost half of the punching capacity for the connection. As indicated by the test results, the top slab reinforcement can support one third to one half of the total theoretical punching load of the connection depending upon the spacing of the reinforcement chosen.

Table 2. Observed load capacity of top reinforcement

Specimen	Average Peak Load P (kips)	Punching Capacity V_c (kips)	Ratio (P/V_c)
B3L3S3	8.0	24.0	0.33
B3L3S6	12.0	24.3	0.50
B3L4S3	9.0	22.2	0.41
B3L4S6	12.5	22.1	0.57
B4L3S3	10.0	24.2	0.41
B4L3S6	13.0	24.2	0.54
B4L4S3	20.0	24.4	0.82
B4L4S6	14.0	24.4	0.57

CONCLUSIONS

Based on the results of this experimental investigation, the following conclusions can be drawn:

1) both the bar size and the bar length have minimal effect on the load carrying capacity of the top slab reinforcement following punching;
2) the bar spacing has the most significant effect on the load carrying capacity of the top slab reinforcement; and
3) the top slab reinforcement can support one third to one half of the theoretical punching load of the interior slab-column connection, depending upon the spacing of the reinforcement chosen.

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