CARRYING CAPACITY AND CRACKING RESISTANCE OF THE BRACKET.

By:- Dr. M.W. Chung and Dr. Ing. S.R. Mpinzire

Formerly Reader of University of Hong Kong - HONG KONG
MSc.(Eng.)H.K.; Ph.D. Leeds; C.Eng.; F.I.C.E.; F.I.E. Austr.; F.H.I.E.

Senior Lecture of University of Dar es Salaam- TANZANIA
MSc.(Eng.); Dr. Ing.Warsaw University of Technology- POLAND; C.Eng.; M.I.E.T.

Summary:-
The carrying capacity and cracking resistance of a bracket with tension reinforcement and horizontal stirrups was studied in test on two brackets. The first specimen was vertically loaded while the second one was combined vertically and horizontally loaded. From the results obtained, load-flexural reinforcement strain curves were plotted for comparison purposes. No load-deflection curves for both tested brackets were plotted for additional comparison purposes. It was found that tension reinforcement and horizontal stirrups are equally effective in increasing the strength and limitation of the cracks although the effective amount of reinforcement is limited.

INTRODUCTION.

In pre-cast reinforced concrete construction, the problem of hinged joints in frame girders are among notable construction problems. Even in case of two reinforced beam elements which later have to be monolithically joined together, during assembly hinges have to be constructed for joining them together.

The problem of working out an optimum hinge joint is always a complicated factor which limits the depth of the cantilever. This means that with the application of a hinge joint, there would not rise a need of increasing the depth of the girder in respect to the cantilevered beam. The necessity of limiting the depth of the girder leads in turn for the shear stresses to approach the concrete tensile stress, f_{ct} and sometimes even higher. The above situation causes the formation of the cracks of considerable openings already at working loads (service loads).

Many failures in reinforced concrete brackets were caused by the designers who did not economize the steel reinforcement which with incorrect systems of steel detailing, instead of improving the situation, they worsen it.

The designer must render himself with the situation that a reinforced concrete bracket is an element which is very sensitive to all types of inrealities in the field of reinforcing steel detailing. In order to clear all these in realities, the designer must considerably know deeper the internal forces than what is allowed in statics. Diagnosing correctly the effects of dislocation of the internal forces resulting from cracked reinforced concrete elements, particularly recognizing the possibility of an optimum influence to this phenomenon through a suitable choice of the cross-section and designed reinforcement, needs a creation of a simplified operation model of internal forces which could be adequate to the reality.

The appearance of cracks on the bracket as in the same way as in other reinforced concrete elements, cannot be regarded as dangerous from the earliest stage because cracking should not always cause feeling of threats. On the other hand, the appearance of the cracks which are perceived by necked eyes in non-reinforced concrete elements should not be disregarded because this may be a signal near the exhaustion of the load-carrying capacity.

For decades, in Europe, the design of brackets was based on Rausch's beam analogy [1]. In early 1960as, Niedenhoff based his design on a simple truss analogy consisting of two members: the main reinforcement acting as horizontal tension member and the concrete acting as an inclined compression member [3],[4]. Mehmel/Freitag put up the idea of using a modified truss model based on a statically indeterminate system with the horizontal main reinforcement and the inclined stirrups acting as tension member[5].The three design theories of Rausch, Franz and Niedenhoff, Mehmel and Becker were compared for shear design by comparative tests. When Hegberg studied the above mentioned three models analytically, by using the theorem of minimum deformation energy as a criterion, he found out that the simple statically determinate truss model had the smallest deformation energy, and this model satisfied better the compatibility conditions[6].

Empirical design criteria has been commonly used in U S A. To mention a few design formulas are like those of Kriz and Raths[7]. A semi-empirical approach based on the so called shear-friction hypothesis was proposed by Mast[8]. Hermensen and Cowan [9] gave a modification of Mast's equation by considering the cohesion effect of the concrete, thus yielding better agreement with the test results.

Somerville [10] also proposed a design method in which the depth of the bracket has to be checked with an empirical formula for shear capacity, the bearing stress to be limited to $0.8f_c$ and compression to be checked from beam analysis with the application of Bernoulli theorem. For very short brackets, Somerville was in favor of "shear-friction plus cohesion approach". Mattock and his counterparts [11] have also proposed to use shear friction method.

Certain comments can be drawn on the latter methods. The design formula as proposed by Kriz and Raths do not include the strength of steel, which is against all common practice in predictions of strength in reinforced concrete and contradictory to test results. The specified limits of reinforcement which may be utilized, i.e, $p = A_{s1} + A_{sc} / b.d \le 0.2$ for vertical loads and $p = A_s / b.d \le 0.013$ for both horizontal and vertical loads, have not been justified.

With regard to Somerville's proposal, the empirical formula suggested did not refer to shear failures as such, but was based on results from shear tests which included various failure modes. Further, it is unlikely that Bernoulli's theorem can be applied to short cantilevers in which shear force is significant.

From the above survey, a research was set-up to establish the influence of the configuration and intensity of the reinforcement on the load carrying capacity, shearing of the bracket. In addition to this, the formation andcauses of the crack opening was also investigated.

SPECIMENS, EQUIPMENT AND TEST PROCEDURES.

Test Program and Specimens.

All specimens consisted of a length of 300 x 250mm column with brackets arranged symmetrically, according to Figs.(1-1) and (1-2). The main tension reinforcement of bracket type A (Fig.(1-1)) consisted of straight deformed bars anchored by a bar of equal diameter welded across their ends. Bracket type B to be subjected to combined vertical and horizontal loading was provided with mild steel bars of diameter 18.9mm welded centrally on both bearing plates to supply horizontal tension forces. Both brackets were detailed with horizontal stirrups as shown in Figs.(1-1) and (1-2) and their reinforcement was calculated according to Herberg method [6].

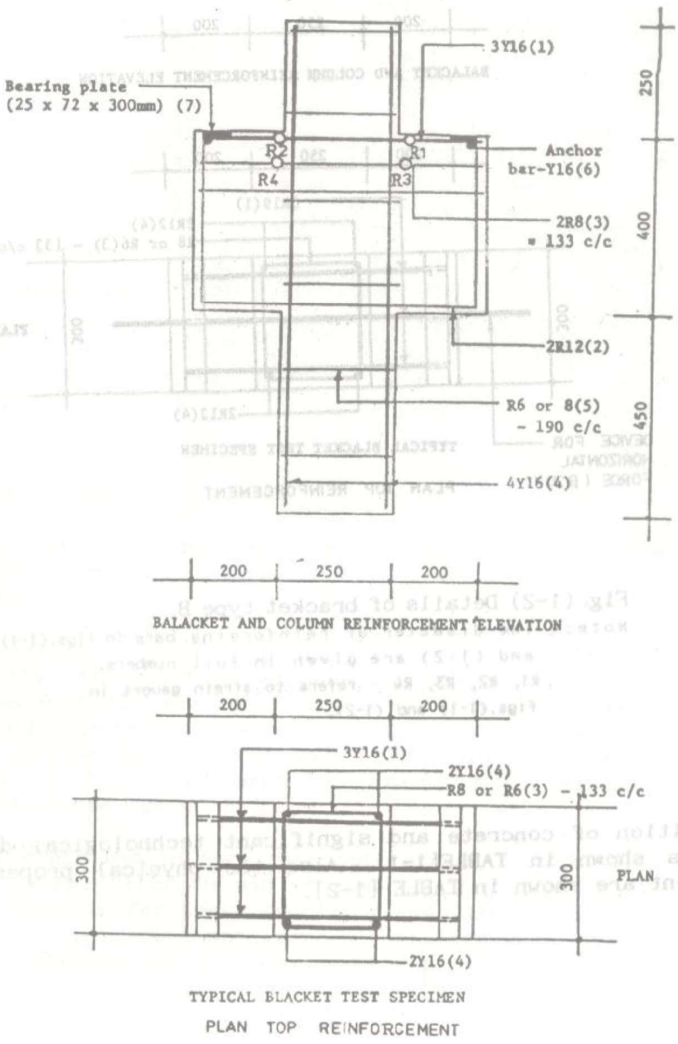

Fig.(1-1) Details of bracket type A.

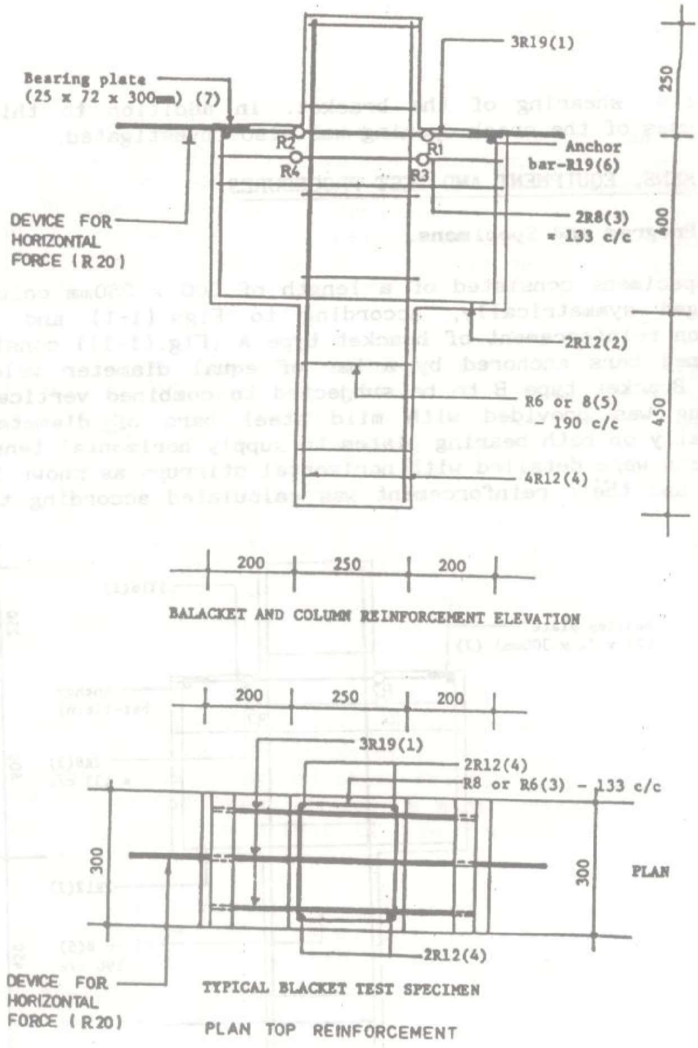

Fig. (1-2) Details of bracket type B.

Note:- The diameter of reinforcing bars in Figs.(1-1) and (1-2) are given in full numbers.
- R1, R2, R3, R4 - refers to strain gauges in Figs.(1-1) and (1-2).

The composition of concrete and significant technological data for both brackets is shown in TABLE[1-1]. Also the physical properties of the reinforcement are shown in TABLE [1-2].

TABLE [1-1] PROPERTIES OF CONCRETE MIXTURES.

SERIES	BRACKET TYPE	MIX RATIO BY WEIGHT	c/w	DEN+ [kg/m^3]	CS+ [N/mm^2]	FS+ [N/mm^2]	SME+ [kN/mm^2]
I	A	1:2.8:3.2	0.74	2310.00	27.56	4.12	23.00
II	B	1:2.8:3.2	0.74	2308.00	29.50	3.43	-

DEN - density of cube,
CS - cube strength,
FS - flexural strength,
SME - static modulus of elasticity.

TABLE [1-2] PHYSICAL PROPERTIES OF REINFORCEMENT FOR BRACKETS A AND B.

BRACKET TYPE	GRADE OF STEEL	BAR SIZE [mm]	No. OF TESTS	SECTION AREA [mm^2]	f_y [N/mm^2]	σ_z [N/mm^2]	E_s [N/mm^2]
A	MS	7.89	3.0	48.13	390.00	405.00	195.00
	HTS	12.18	3.0	116.46	390.00	405.00	210.00
	HTS	16.10	3.0	203.47	445.00	635.00	210.00
B	MS	11.89	3.0	111.03	390.00	405.00	195.00
	MS	15.97	3.0	200.21	396.00	405.00	208.00
	MS	18.90	3.0	280.41	375.00	445.00	195.00

Note:- f_y - average yield strength,
σ_z - average ultimate strength,
E_z - Young Modulus of Elasticity,
MS - mild steel,
HTS - high tensile steel.

MATERIALS AND FABRICATION.

All specimen were cast in wood-forms. The bottom part of the reinforcement caging in the forms were supported from the base of the form on concrete cover blocks.

The concrete was mixed in non-tilting horizontal drum mixer of 0.22m^3 capacity. All batching was done by weight. The concrete was placed in the forms with the aid of internal vibrator. Forms were moved one day after casting. The brackets were subject to wet curing for 4 days followed by storage in the curing room with 100% humidity content at temperature 27^0± 2^{0}C for 16 days and thereafter stored in the air of the laboratory until tested at the age of 28 days.

The cube strength was in each case determined from 12 test cubes of 150mm in size; flexural strength was determined on a 2 prisms measuring 100 x 300mm. The concrete for these specimens was taken from the batches used to prefabricate each bracket specimen. All of them were cured in an identical manner and tested on the same day as the corresponding brackets.

Instrumentation and Test Procedures

The brackets were instrumented with ▪ LA-6 as well as FLK-6 strain gauge mounted on the reinforcement and with FL-60 strain gauges mounted on the concrete.

For convenience both specimens were tested in an inverted position as may be seen in Fig.(1-3).

Fig.(1-3) Testing Arrangements.

The brackets were supported directly on the plain bearing plates, through hinged rollers resting on the tops of the steel frame mounted on the concrete floor.

The brackets were tested in a 1000kN capacity testing machine type "WOLPERT AMSLER" for applying the vertical load; to apply horizontal load, a tension hydraulic jack type "T-PAK NIKE" of 120kN capacity was used.

Vertical Loading Only.

The bracket type A was loaded through steel bearing plates symmetrically welded on top of the tension reinforcement of the bracket as shown in Fig.(1-1). The length of the bearing plates were equal to the width of the brackets. The width and the thickness of the plates were 71mm and 8mm respectively. The load was applied to the bottom of the column stub by the tension testing hydraulic-jack which was able to supply a total safe load of 1000kN. Fig.(1-4) shows the test set-up used for the test involving vertical load only.

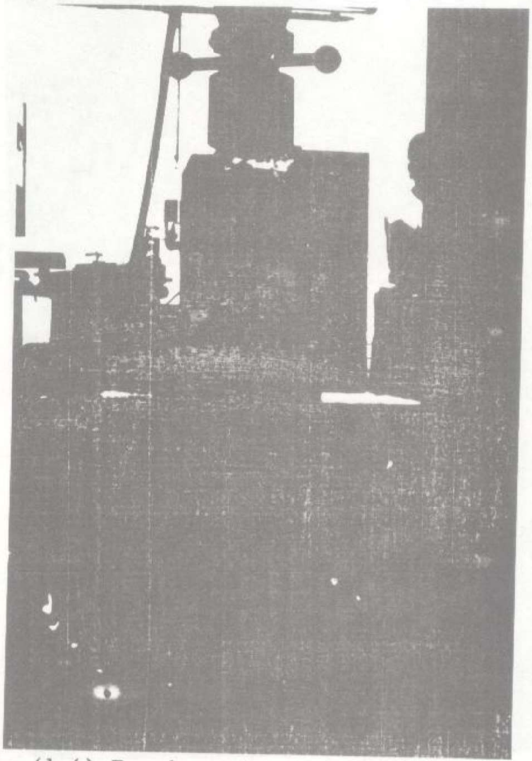

Fig.(1-4) Testing Arrangement for Vertical Load only.

Combined Vertical and Horizontal Loading.

The horizontal force which develop in pre-cast beams as a result of restrained volume changes were simulated by horizontal forces applied at the level of the top of the brackets. To permit a direct transfer of the horizontal forces to the tension reinforcement, the 18.9mm mild steel round bars were also welded on top of the tension reinforcement. The horizontal forces were applied by two tension testing hydraulic jack type "I-PAK NIKE" of 120kN capacity to the bearing plates through 18.9mm diameter steel bars. The tension hydraulic jacks were positioned on each side of the brackets in such a way that the resultant load supplied through the mild steel bars was at least approximately at the level of the top of the brackets.

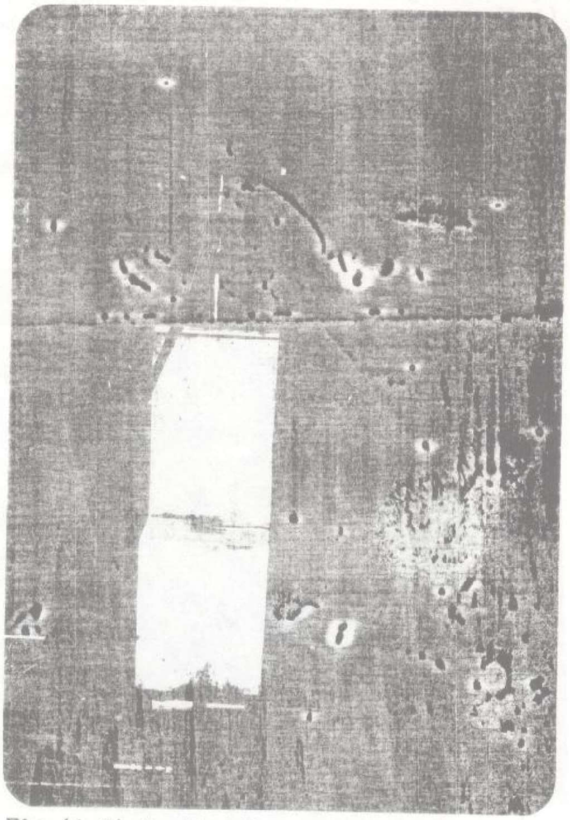

Fig.(1-5) Testing Arrangement for Combined Horizontal and Vertical Loading.

The tension hydraulic jacks used for applying the horizontal forces were calibrated so that the loads could be correlated with the oil pressure. The required constant pressure applied by the hydraulic jacks was continuously controlled on the pressure meter gauge which was connected to the hydraulic jack systems.

The vertical load was applied in the same manner as in the test of the bracket subjected to vertical load only.

The loading system for combined horizontal and vertical loading is shown in Fig.(1-5).

The Results

Behavior of Brackets Subjected to Vertical Loads Only.

Initially the bracket behaved elastically and the stress in the main tension reinforcement was proportionally to the load. The first cracks to appear were the flexural cracks which propagated from the intersection of the column face and the horizontal face of the bracket. These cracks penetrated about half through the depth of the bracket at about half the ultimate load. After formation of these cracks tension reinforcement stress increased much more rapidly.

These cracks were aligned roughly along a line running from the intersection of the horizontal down face of the bracket and column face to a point between the inner edge of the bearing plate and the centre of the bearing plate. The initial length of these cracks was about one-third the effective depth of the bracket. As the load was further increased, the diagonal tension cracks increased in length, at first rapidly, then much more slowly as the ultimate load was approached. The relationship between the applied load and the strains in the horizontal reinforcement is shown in Fig.(1-6). Few representative specimens before and after failure are shown in Figs.(1-7) and (1-8).

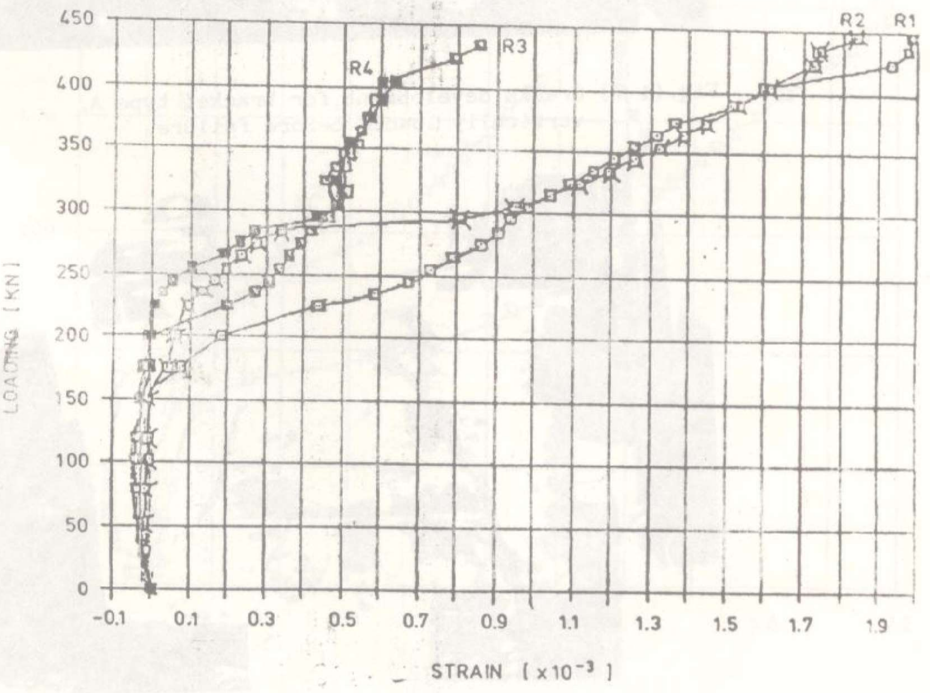

Fig.(1-6) Load-flexural reinforcement strain curves for bracket type A, loaded vertically.

Fig. (1-7) Cracks development for bracket type A, vertically loaded before failure.

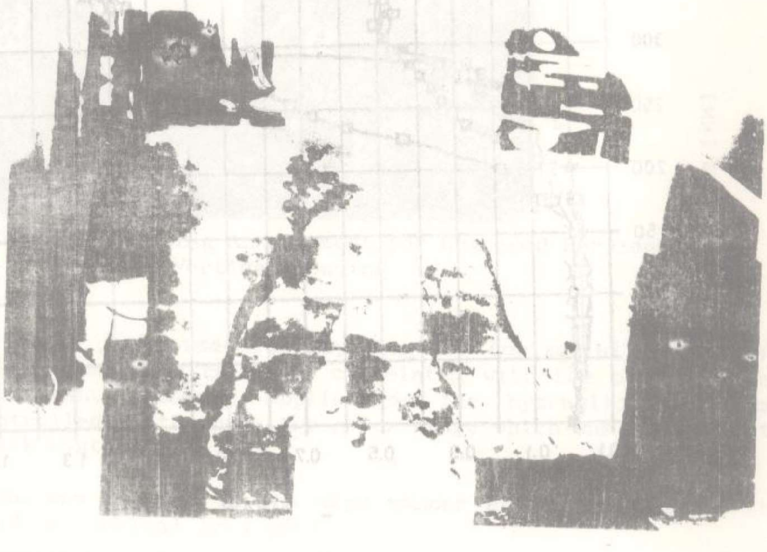

Fig. (1-8) Full cracks development for bracket type A, vertically loaded after failure.

Behavior of Bracket Subject to both Vertical and Horizontal Loads.

As in the case of bracket type A subjected to vertical load, the bracket behaved elastically and the stress in the main tension reinforcement was proportional to the load. The first cracks were later formed than in bracket type A. The cube strength of concrete used in bracket A was lower than that of bracket type B. There were more or less vertical cracks and they were due to direct tension stresses produced in the concrete by the horizontal force. The specimen failed in flexural with opening of flexural cracks adjacent to the column face as the main tension reiforcement yielded; the diagonal tension crack remained fine.

The relationship between the applied load and the strains in horizontal reinforcement is shown in Fig. (1-9). Few representative specimens before and after failure are shown in Figs. (1-10) and (1-11).

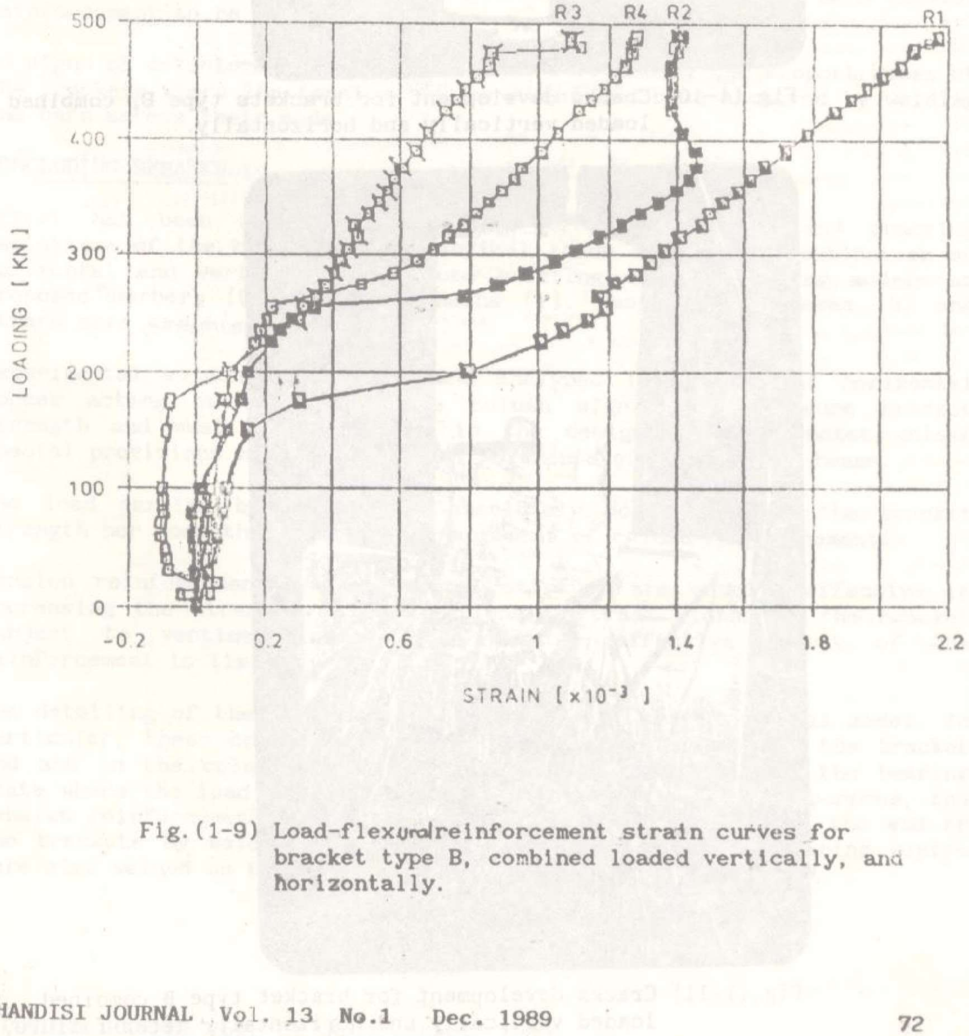

Fig. (1-9) Load-flexural reinforcement strain curves for bracket type B, combined loaded vertically, and horizontally.

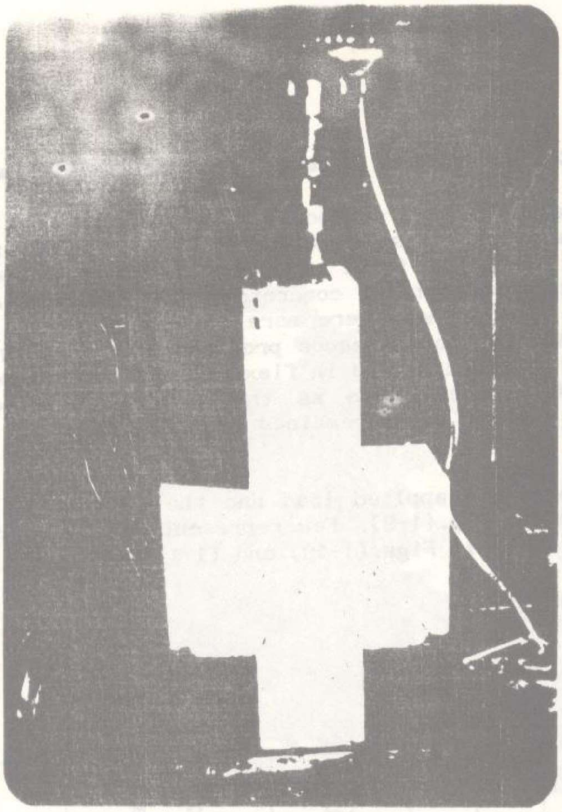

Fig. (1-10) Cracks development for brackets type B, combined loaded vertically and horizontally.

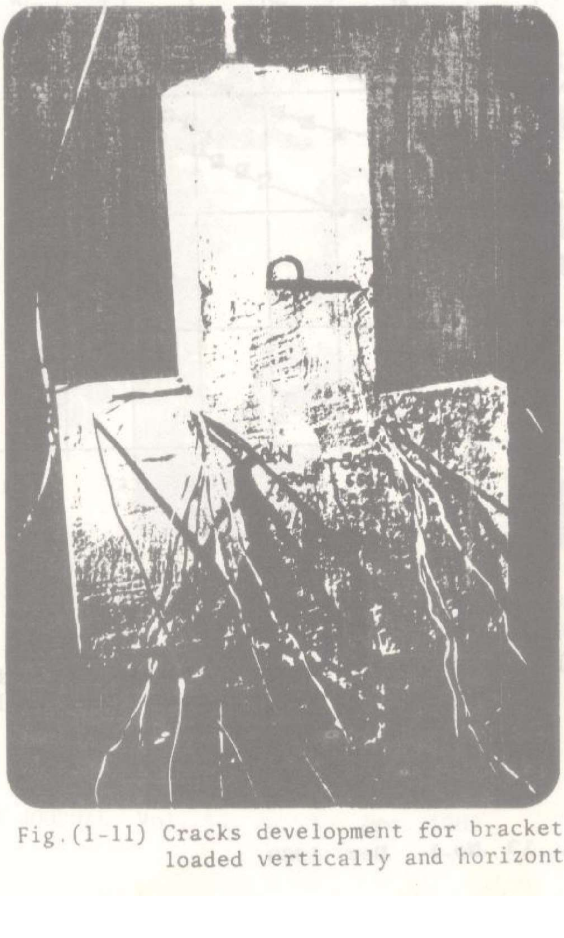

Fig. (1-11) Cracks development for bracket type B combined loaded vertically and horizontally after failure.

The brittle and complete diagonal tension failure of the bracket without stirrups as experienced in the tests carried by Kriz and Raths were not sighted in the tests of brackets type A and B. This provided additional evidence to the argument put ahead by Kriz and Raths that all brackets should be reinforced with horizontal stirrups in addition to the main tension reinforcement in order to eliminate the possibility of this type of failure from occurring.

All brackets had horizontal stirrups, the yield strength of which was approximately half the yield strength of that part of the remaining tension reinforcement not resisting the horizontal force. Both brackets were pre-fabricated from fines and gravel concrete for which reinforcement strains were obtained. It is clearly seen that the stress in the main tension reinforcement at failure was equal to or very close to the yield point of the steel.

Further more, failure of both brackets type A and B were much less abrupt than in the case of brackets without stirrups as reported by Kriz and Raths. This provided that the amount of horizontal stirrups reinforcement provided in the brackets were adequate to eliminate pre-mature diagonal tension failures and to permit the potential strength of the main tension reinforcement to be developed.

No signs of deterioration on the concrete surrounding the support zones of the brackets where the tension reinforcement had been anchored by welding the bars across their ends.

CONCLUDING REMARKS.

Effort has been made to study the carrying capacity and cracking resistance of the brackets with vertical loads and with a combination of horizontal and vertical loads. Some existing and interesting models as proposed by Herberg [6], Kriz and Raths [7], Mast [8], Hermesen [9] and others have examined.

Experimental evidences recorded and analyzed indicates that horizontal forces acting outwards from the column significantly reduce bracket strength and must be considered in the design of the bracket unless special provisions are made for free movements of the support beams.

The load carried by the column absolutely do not affect the bracket strength nor does the amount or arrangement of column reinforcement.

Tension reinforcement and horizontal stirrups are equally effective in increasing the strength and limitations of crack widths of the bracket subject to vertical loads. However, the effective amount of the reinforcement is limited.

The detailing of the bracket has to comply with the structural model. In particular, these concern anchorage of the reinforcement at the bracket end and in the column, and the location and dimensions of the bearing plate where the load is transferred to the bracket. For this purpose, the tension reinforcement in the tested specimen was anchored at the end of the brackets by welding the bars across their ends. The bearing plates were also welded on top at the end of the tension reinforcement.

It has been known for a long time that properly designed and detailed brackets may be constructed without stirrups. Analysis of some existing research reports have also documented this. However, both tests evaluated here were for short term loads, and until further evidence from tests with sustained loads is available, a certain number of stirrups has to be recommended. The stirrups will also be effective in limiting and distributing the cracks and will improve the internal force distribution in the bracket, since the detailed stress condition is uncertain.

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