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INFLUENCE OF P-Δ EFFECT AND AXIAL FORCE VARIATIONS ON SEISMIC PERFORMANCE OF R/C BEAM-COLUMN JOINTS

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ABSTRACT

This paper compares the experimental response of beam-column subassemblies subjected to seismic type loading plus variable axial load and P-Δ effect with the response of similar specimens subjected to cyclic loading but under constant axial load. Test data demonstrated that axial load changes and P-Δ effect during an earthquake cause significant deterioration in the earthquake resistance of these structural elements.

KEYWORDS

Beam-column frame; beams (supports); columns (supports); connections; cyclic loads; earthquake resistant structures; hinges (structural); joints (junctions); reinforced concrete; reinforcing steel; shear strength; structural analysis.

INTRODUCTION

The first seismic behavior study of beam-to-column connections was conducted by Hanson and Connor (1967). One of the primary variables of the study was the column axial load. Since then, significant experimental evidence relevant to the influence of the axial column load on the seismic response of these elements has emerged. With the exception of the specimens described by Paulay et al., (1980), and Tsonos et al. (1996), axial forces were considered to be constant. The conclusion of these studies is that earthquake resistance of reinforced concrete beam-column connections is influenced by the level of axial force. Still little is known about the influence that axial force variations may have on the hysteretic behavior of these elements.

The response of reinforced concrete ductile beam-column subassemblies to inelastic cyclic lateral loading including the influence of P-Δ effects has been studied since the late 1970s (Bertero and Popov, 1977; Bertero, 1979; Soleimani et al., 1979). During an earthquake, P-Δ effects are produced only through changing axial loads. Again the few experimental studies (Bertero and Popov, 1977; Bertero, 1979; Soleimani et al., 1979) conducted to date have been performed on beam-column joints in which the P-Δ effect was produced while constant axial loads were maintained. The conclusion of these studies is that the P -Δ effect causes significant slippage (pull-out) of the beams' main longitudinal reinforcement along the beam-column joint, which subsequently leads to the formation of an unstable kinematic mechanism in the subassemblage.

In this paper an extensive experimental investigation was conducted to study the response of reinforced concrete ductile exterior beam-column connections to seismic type loading with varying axial load and P-Δ effect, which was produced by a simultaneous change in the axial load P and the lateral displacement Δ .

BEAM-COLUMN JOINT SPECIMEN DETAILS

The experimental program included 12 specimens in four series as detailed in Table 1. The axial load for the series A specimens remained constant throughout the test. The axial load of the other series specimens M, MS and MX was made variable during the tests. The specimens in series A, M, MS and MX had the dimensions shown in Fig. 1. The specimens in a given series (e.g. A_1) had the same beam, column and beam-column connection reinforcement as the corresponding specimens of the second series specimens (e.g. M_1). The third series MS consisted of two specimens MS_i (where i=3.4) each of them having the same beam and column reinforcement as the second series (M) specimens M_i . The MS series specimens, however, had 70% more joint transverse reinforcement than their counterparts in series M. The fourth series MX consisted of two specimens MX_j (where j=2.4) which were reinforced with four crossed inclined bars bent diagonally across the joint core, as shown in Fig. 1(b), instead of the four intermediate longitudinal bars in the column of the conventionally reinforced specimens A_j and M_j of series A and M.

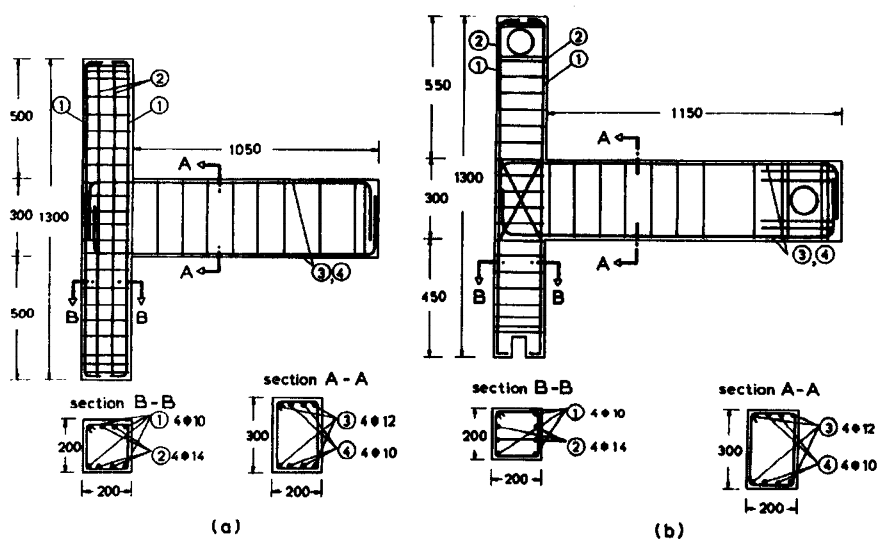


Fig. 1. (a)Typical specimen of type A, (b)Typical specimen of type MX (dimensions in cm, 1 cm=0.394 in)

The principal variables of the testing program were: a)The axial load in the columns, b)The P-Δ effect, c)The horizontal joint shear stress which is determined by parameter γ, as defined by the ACI-ASCE Committee 352 (ACI 352R-1985), d)The transverse joint reinforcement, and e)The inclined reinforcing bars in the joint region. All specimens were loaded transversely according to the load history shown in Fig. 2(a). The axial load for the series A specimens was kept constant approximately $0.45P_b$ during the test. The axial load of the other series (M, MS and MX) specimens varied during the test according to the history shown in Fig.2(b).

Table 1. Description of beam-column connection specimens

Series	Specimen	Column Reinforcement	Beam Reinforcement	Joint Transverse Reinforcement	Inclined Reinforcing Bars	Concrete Compressive Strength (psi)	P_{max} / P_b Maximum Column Axial Load
A	A_1	8Ø14	4Ø14	3Ø8	0	3770	0.51
	A_2	4Ø14 4Ø10	4Ø10 4Ø12	4Ø8	0	4490	0.45
	A_3	8Ø14	6Ø14	3Ø8	0	4960	0.42
	A_4	8Ø10	8Ø14	3Ø8	0	4900	0.42
M	M_1	8Ø14	4Ø14	3Ø8	0	3620	0.94
	M_2	4Ø14 4Ø10	4Ø10 4Ø12	4Ø8	0	4930	0.72
	M_3	8Ø14	6Ø14	3Ø8	0	3910	0.95
	M_4	8Ø10	8Ø14	3Ø8	0	4860	0.72
MS	MS_3	8Ø14	6Ø14	5Ø8	0	3770	1.01
	MS_4	8Ø10	8Ø14	5Ø8	0	4880	0.73
MX	MX_2	4Ø10	4Ø10 4Ø12	4Ø8	4Ø14	4800	0.75
	MX_4	4Ø10	8Ø14	3Ø8	4Ø10	4780	0.74

Ø8, Ø10, Ø12, Ø14 = bars with diameters of 8mm, 10mm, 12mm, 14mm
Summary of specimens' steel yield stress in ksi,
Bar size: Ø8=71.74, Ø10=67.65, Ø12=76.67, Ø14=70.30

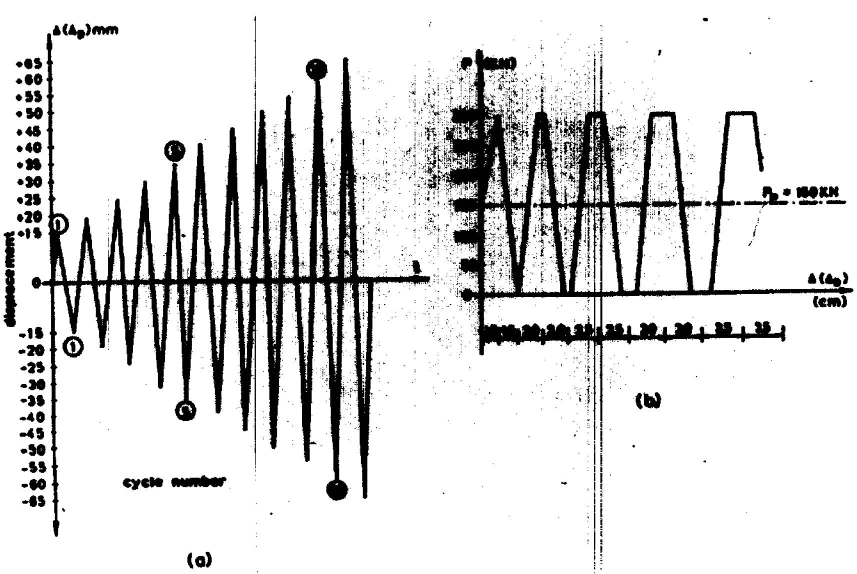


Fig. 2. (a)Loading sequence, (b)Axial force history for specimens of types M, MS and MX

THEORETICAL CONSIDERATIONS

Figure 3(a) shows a reinforced concrete exterior beam-column joint for a moment resisting frame. The shear forces acting in the joint core are resisted (i)partly by a diagonal compression strut that acts between diagonally opposite corners of the joint core, and (ii)partly by a truss mechanism formed by horizontal and vertical reinforcement required in the joint core and concrete compression struts. Both mechanisms depend on the core concrete strength. Thus, the ultimate concrete strength of joint core under compression/tension gives also the ultimate strength of the connection. Consider the section I-I in the middle of the joint height (Fig. 3(a)). The forces acting in the concrete in this section from each mechanism is shown in Fig. 3(b). Each force acting in the core is analysed into two components along x and y axis

$$V_{jv} = D_{cy} + (T_1 + \dots + T_4 + D_{1y} + \dots + D_{vy}) = D_{cy} + D_{sy}$$

and

$$V_{jh} = D_{cx} + (D_{1x} + \dots + D_{vx})$$

where V_{jv} and V_{jh} are the vertical and horizontal joint shear stress forces respectively.

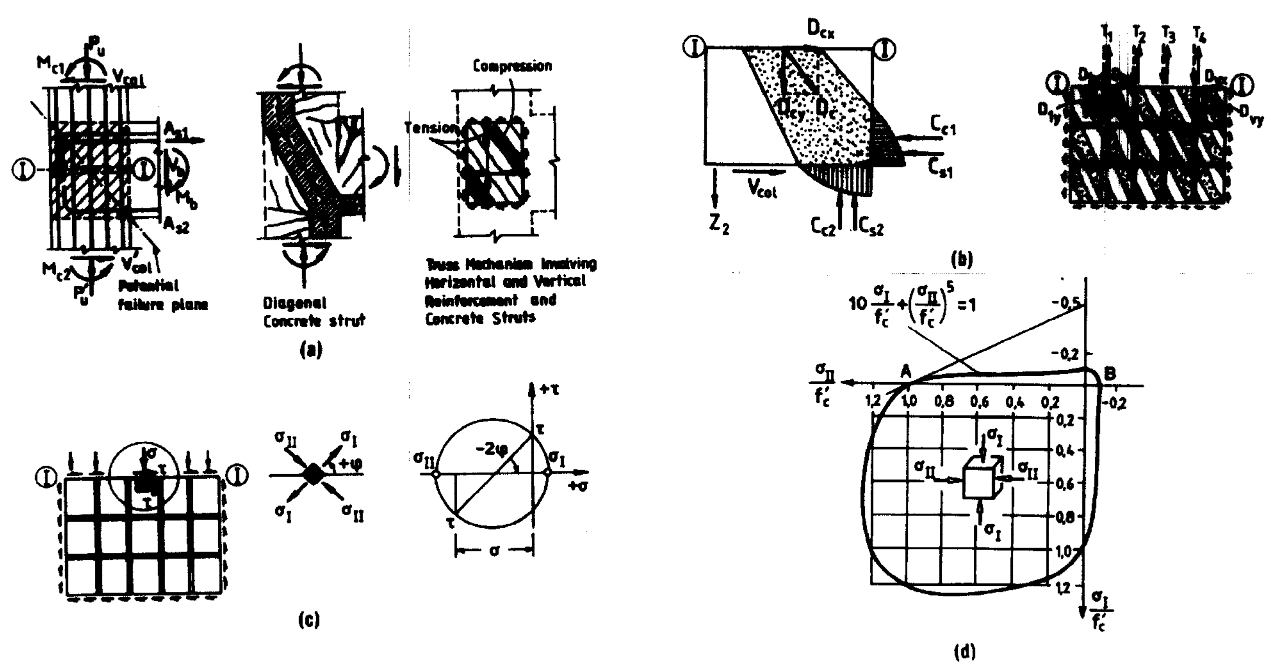


Fig. 3. (a) External beam-column connection and the two mechanisms of shear transfer, (b) Forces acting in the joint core concrete through section I-I from the two mechanisms, (c) Stress state of element of this studied region, (d) Representation of concrete biaxial strength curve by a parabola of 5th degree

The vertical normal compressive stress σ and the shear stress τ uniformly distributed over the section I-I are given by the following equations

$$\sigma = \frac{V_{jv}}{h_c' \cdot b_c'} \tag{1}$$

$$\tau = \frac{V_{jh}}{h_c' \cdot b_c'} \tag{2}$$

From Eq. (1) and (2) :

$$\sigma = \frac{V_{jv}}{V_{jh}} \cdot \tau \tag{3}$$

It is well known that

$$\frac{V_{jv}}{V_{jh}} = \frac{h_b}{h_c} = \alpha \tag{4}$$

Thus

$$\sigma = \alpha \cdot \tau \tag{5}$$

The maximum principle stresses are given by Mohr's circle (Fig.3(c)) and the following expression result

$$\sigma_{I,II} = \pm\frac{\sigma}{2} + \frac{\sigma}{2}\sqrt{1+\frac{4\tau^2}{\sigma^2}} \tag{6}$$

From the diagram of Bevavior of Concrete under Biaxial Stresses (Kupfer et al., 1969), it was concluded that the branch AB can be represented by a 5th degree parabola (Fig. 3(d)). Thus, for branch AB:

$$10\frac{\sigma_I}{f'_c} + \left(\frac{\sigma_{II}}{f'_c}\right)^5 = 1 \tag{7}$$

f'_c is the increased joint concrete compressive strength due to confining, which is given by the model of Sheikh and Uzumeri (1982), according to the expression

$$f'_c = 0.85 K_s \cdot f_c \tag{8}$$

f_c is the joint concrete compressive strength and K_s is the parameter of the model.

By substituting Eq. (5) and (6) into Eq. (7) and using $\tau = \gamma\sqrt{f'_c}$, the following expression is derived

$$(x+\psi)^5 + 10\psi - 10x = 1 \tag{9}$$

where

$$x = \frac{\alpha\,\gamma}{2\sqrt{f'_c}} \tag{10}$$

and

$$\psi = \frac{\alpha\,\gamma}{2\sqrt{f'_c}}\sqrt{1+\frac{4}{\alpha^2}} \tag{11}$$

The solution of the system of equations (9)-(11) gives the beam-column joint ultimate strength.

The validity of the above formulation needs to be checked and will be established using test data for 38 exterior and interior beam-column subassemblages tested in the Laboratory of Reinforced Concrete at Aristotle University of Thessaloniki (23, included the specimens of the present program) and in the United States (15). The comparison between experimental and predicted results per system of Eq. (9)-(11) is shown in Fig. 4. An excellent correlation is observed.

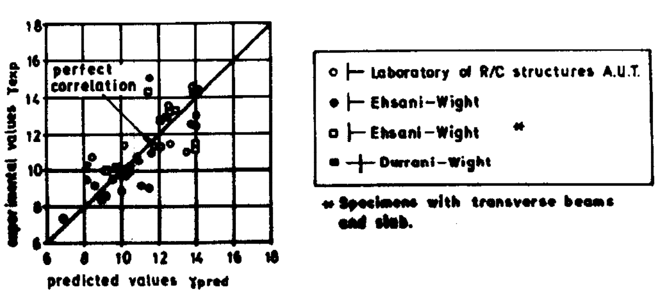


Fig. 4. Correlation of experimental and predicted by the formulation (Eq.(9)-(11)) values of strength of beam-column subassemblages

The horizontal shear force acting in the joint and including P-Δ effect is:

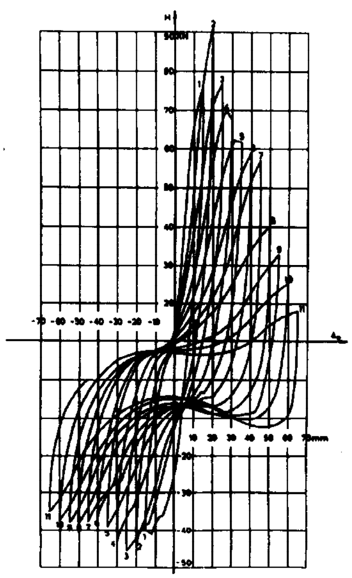
$$V_{jh} = T_b \pm \frac{P \cdot \Delta}{h_s} - H \tag{12}$$

where Δ is the lateral displacement and h_s is the height of the column.

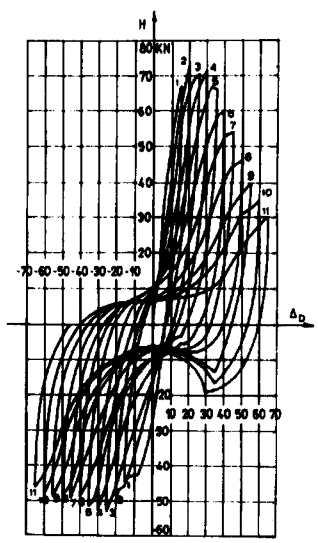
The increase of axial column load increase also the joint shear stress (Park and Paulay, 1984; Pantazopoulou and Bonacci, 1992). Thus, beam-column connections designed to develop shear stress significantly lower than the joint ultimate strength (as defined by the presented formulation $\gamma_{ult}\sqrt{f'_c}$ psi) for axial loads derived by an elastic analysis for factored lateral loads, possibly under variations of the axial load during earthquakes develop shear stress very close to the joint ultimate strength, when the values of column axial load are high and including the influence of P-Δ effect. In this case explosive shear failure of joints is inevitable.

DISCUSSION OF TEST RESULTS

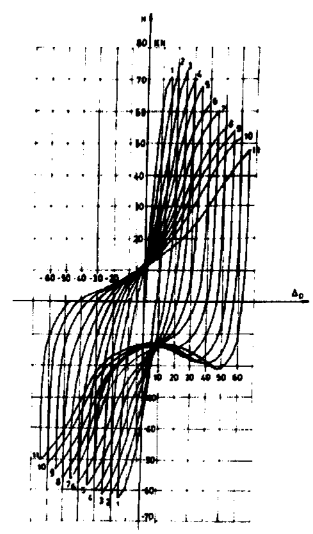
Plots of applied load versus displacement at the load point for representative specimens used in the program are shown in Fig. 5. In order to facilitate the observation of strength degradation the ratio of maximum load carried by the specimens during each upper half cycle of loading to load in the first upper half cycle, and the ratio of maximum load carried by the specimens during each lower half cycle to load in the first lower half cycle are shown in Fig. 6.



Specimen M_4



Specimen MS_4



Specimen MX_4

Fig. 5. Load-versus-deflection response for specimens M_4 , MS_4 and MX_4

The high values of axial load, approximately equal to 0.85 P_b, in the upper half cycles of loading of specimens M plus the P-Δ effect, produced also high values of joint shear stress which is very close to the ultimate joint shear strength, as is clearly demonstrated by the values of ratio $\gamma_{exp}/\gamma_{ult}$ of these specimens, that were very close to 1.00 (Fig. 6). Explosive shear failure of the joints of these specimens was observed. The load carrying capacity of specimens M was sharply reduced after the first two cycles of loading especially in the upper half cycles. In the lower half cycles of specimens M due to the low values of axial load (Fig. 2(b)), the influence of P-Δ effect is significantly decreased and the values of joint shear stress were lower than the ultimate shear strength (Fig. 6). Thus, a more stable behavior was observed in these half cycles compared to that in the upper half cycles. Specimens A which had constant axial load, approximately equal to 0.45 P_b, during the test, exhibited increased load carrying capacity compared to that of their counterparts of type M, because their joint shear stress were significantly lower than the ultimate strength of the joint thus avoiding premature joint shear failure ($\gamma_{exp}/\gamma_{ult} \approx 0.65$, Fig. 6). Specimens MS_3 and MS_4 had 70% more joint transverse reinforcement than specimens M_3 and M_4 respectively. A comparison of the cyclic load carrying capacity between specimen pairs MS_3, MS_4 and M_3, M_4 (Fig. 6) indicated that the increase in the joint transverse reinforcement significantly improves the overall behavior of the specimens. Specimens MX_2 and MX_4 were reinforced with four crossed inclined bars bent diagonally across the joint core, (Fig. 1(b)). The hysteresis loops of specimens with inclined bars demonstrated increased strength, ductility and energy dissipation capacity and less joint damage compared with those of their corresponding conventionally reinforced specimens M_2 and M_4 (Fig. 5, 6). The significant improvement in beam-column joint earthquake-resistance of specimens MX_2 and MX_4 was due to the presence of inclined bars, which introduces a new mechanism of shear transfer in addition to the two well known mechanisms of conventionally reinforced joints, the truss mechanism of inclined bars. It was demonstrated that this mechanism can remain active throughout the test (Tsonos, et al., 1992).After the first yielding, the shear carried by the inclined bars is

$$V_{sx} = 2A_s \cdot f_y \cdot \sin\theta \tag{13}$$

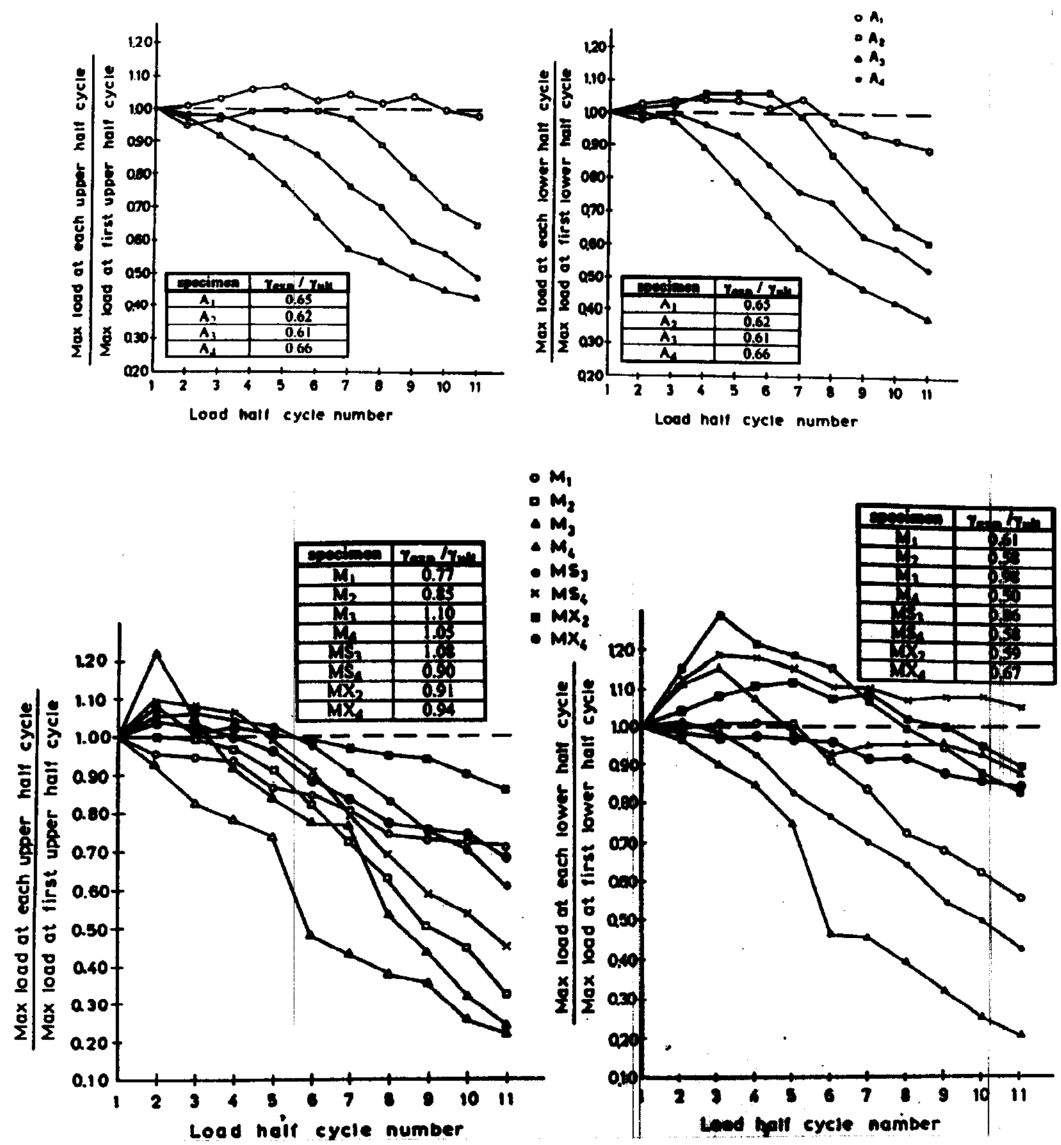


Fig. 6. Cyclic load-carrying capacity for all the specimens

where A_s is the area of inclined bars, f_y is the yield stress of these bars and θ is the inclination of these reinforcing bars to the column axis. The improvement in beam column joint performance due to the presence of inclined bars was much higher than that obtained by the presence of 70% greater amount of transverse reinforcement as is clearly demonstrated by the comparison of the hysteresis loops of specimens M_4, MS_4 and MX_4 (Fig. 5).

CONCLUSIONS

This experimental and analytical study focused on the influence of axial load variations and P-Δ effect on the behavior of beam-column joints and led to the following conclusions:

1. A comparison of the seismic performance of specimens M with hte performance of specimens A indicates that axial load changes and P-Δ effect during a seismic type of loading increase significantly the joint shear stress which results in remarkable deterioration in the beam-column joint earthquake resistance.
2. External beam-column joints with inclined bars performed considerably better than those with conventional reinforcements under seismic type loading with variations of axial load and P-Δ effect.
3. Additional joint transverse reinforcement improves significantly the beam-column joint strength and ductility and enhances the overall behavior of the subassemblage, even for high values of joint shear stress.

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