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Improvement of the earthquake resistance of R/C beam-column joints under the influence of *P-Δ* effect and axial force variations using inclined bars

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Abstract: In this study, theoretical and experimental results are presented which were obtained during an investigation of the influence of the *P*-Δ effect that was caused by the simultaneous changing of the axial load *P* of the column and the lateral displacement Δ in the external beam-column joints. The increase or decrease of Δ was simultaneous with the increase or decrease of the axial compression load *P* and caused an additional influence on the aseismic mechanical properties of the joint. A total of 12 reinforced concrete exterior beam-column subassemblies were examined. A new model, which predicts the beam-column joint ultimate shear strength, was used in order to predict the seismic behaviour of beam-column joints subjected to earthquake-type loading plus variable axial load and *P*-Δ effect. Test data and analytical research demonstrated that axial load changes and *P*-Δ effect during an earthquake cause significant deterioration in the earthquake-resistance of these structural elements. It was demonstrated that inclined bars in the joint region were effective for reducing the unfavourable impact of the *P*-Δ effect and axial load changes in these structural elements.

Key words: 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.

1. Introduction

During an earthquake lateral inertial forces produce overturning moments, which increase the column compressive stresses on one side of the structure and reduce them on the other. In addition, the vertical component of earthquake motion exists, which is often ignored by engineers. The forces caused by overturning moments can be important for beam-column joints, especially the exterior ones, in some multistorey structures. The axial forces in exterior beam-column connections change continuously during an earthquake and may differ significantly from those derived by an elastic analysis for factored lateral loads (Townsend and Hanson 1977, Penelis and Kappos 1997).

The first seismic behaviour 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 column axial load on the seismic

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response of these elements has emerged (Paulay *et al*. 1980, Tsonos *et al*. 1995, Agbabian *et al.* 1994) but still, little is known about the influence that axial force variations may have on the hysteretic behaviour of these elements.

The response of reinforced concrete ductile beam-column subassemblies to inelastic cyclic lateral loading, including the influence of *P*-Δ effect has been studied since the late 1970s (Bertero and Popov 1977, Bertero 1979, Soleimani *et al.* 1979). 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 load was maintained. The conclusion of these studies is that the *P*-Δ effect can be extremely pronounced on the lateral load deformation response of the lower stories of tall buildings, and must be considered in the aseismic design of such buildings. Uzumeri (1977) showed that "secondary moments due to *P*-Δ effect combined with stiffness deterioration have a significant importance in the design of the structural system".

Park and Paulay (1984) first introduced the idea of using crossed inclined bars in reinforced concrete beam-column joints. Tsonos *et al*. (1992), applied the idea of Park and Paulay in reinforced concrete exterior beam-column joints and carried out some experiments. The results showed that the joints with inclined bars performed considerably better than those with conventional reinforcement. In this paper, an experimental and analytical investigation was conducted to study the response of reinforced concrete ductile exterior beam-column connections to seismic-type loads with variable axial load and *P*-Δ effect, which was produced by a simultaneous change in axial load *P* and lateral displacement Δ. The use of inclined bars in reinforced concrete exterior beam-column joints was also examined with regard to the decrease of the unfavourable effects of the *P*-Δ effect and axial load change.

2. Background and previous research into the seismic performance of R/C beam-column subassemblages under axial force variations and *P*-Δ effect

2.1 Varying column axial load

Twenty-two reinforced concrete exterior beam-column connections were tested by Townsend and Hanson (1977). Test loading conditions were chosen to represent earthquake loading. One of the three parameters examined in detail was the magnitude of column axial load, including axial tension. The magnitude of axial forces applied to these specimens is shown in Table 1. Test results showed that hysteresis loops for these connections were significantly thinner for no axial column load or for tension axial column load than for axial compression loads.

Uzumeri (1977) investigated the behavior of cast-in-place reinforced concrete exterior beam-column joints subjected to seismic loading. Eight exterior beam-column subassemblages were tested. Throughout the tests, the columns of these specimens were under high constant axial compressive force. The magnitudes of axial forces applied to these specimens are shown in Table 2.

Uzumeri concluded that this large axial compressive force applied to the concrete struts of the joints of the above specimens was detrimental.

Field observations as well as the results of the dynamic analysis of high-rise structures indicate that the combined effect of high over-turning moment and excitation of the vertical vibration modes may result in a very significant reduction in the compressive forces obtained from static structural analysis (Agbabian *et al*. 1993).

Table 1 Specimens tested by Townsend and Hanson (1977)

Specimen	P/P_b
1	0
2	+0.60
3	–0.40
4	+0.40
5	–0.40
6	–0.40
7	+0.40
8	0
9	0
10	0
11	+0.60
12	+0.60
13	+0.60
14	+0.40
15	–0.40
16	+0.60
17	+0.40
18	0
19	0
20	+0.60
21	+0.40
22	–0.40

Positive loading represents tension loads
P is the axial column load
P_b is the balanced column load

Table 2 Specimens tested by Uzumeri (1977)

Specimen	1	2	3	4	5	6	7	8
P/P_b	1.07	1.07	1.05	1.08	1.12	1.10	1.35	1.66

P is the axial column load
P_b is the balanced column load

Agbabian *et al*. (1993) tested three interior beam-column subassemblages under variable amplitude cyclic displacement-controlled loading. Axial load levels of 10%, 5% and 0% of the squash capacity (% P_u) were applied. The results of this study indicated that the behavior of the beam-column joint region was clearly and significantly affected by the axial column load. Thus, the overall displacement response of the subassemblage decreased by 22% for a decrease in the axial load from 10 to 5% of the squash load.

Higazy *et al*. (1996) investigated the seismic performance of ordinary-strength concrete and marginal high-strength concrete interior beam-column subassemblages subject to reduced axial column compression of 5% of the squash capacity (% P_u) or axial tension of 5% of the squash capacity % P_u (P_u is the ultimate column capacity in compression).

Six interior beam-column subassemblages were tested. Through the results of this study, it was demonstrated that the seismic behavior of a beam-column connection is sensitive to reductions in axial column compression more so for tension application. The vulnerability of subassemblage performance was manifested in losses in ductility, joint-shear strength, and energy dissipation capacities due to the introduction of a tensile axial force of 5% of the column squash capacity or the absence of column compression.

It has to be pointed out that in the experimental studies presented previously (Townsend and Hanson 1977, Uzumeri 1977, Agbabian *et al*. 1993, Higazy *et al*. 1996), axial loading in columns was kept constant during cyclic loading. However, it is understood that in an actual R/C building subjected to a seismic motion, the axial load in a column may change (within a fraction of a second) from high compression to zero column axial load or to net tension. Therefore, its behavior is more complex than previously described (Penelis and Kappos 1997). Penelis and Kappos (1997) state that only a limited number of experimental studies (Jirsa *et al*. 1980, Abrams 1987, Tsonos *et al*. 1995) have addressed the problem of changing axial loading in columns and in beam-column joints subjected to cyclic shear, wherein the typical loading history was that high shear in the column or in the beam-column joint was accompanied by high compression, while low shear was accompanied by low compression or tension. The axial load history of the study Tsonos *et al.* (1995) was also used in the present study, Fig. 3(b).

2.2 *P*-Δ *effect*

In the exterior beam-column subassemblages tested by Uzumeri (1977), the columns were subjected to a high axial compressive force. Uzumeri theoretically showed that if the magnitude of the column axial load present in these subassemblages is high, a rapid deterioration in their strength, stiffness and energy dissipation capacity will take place due to the *P*-Δ effect.

Bertero, Popov and Soleimani (Bertero and Popov 1977, Bertero 1979, Soleimani *et al*. 1979) investigated the seismic behavior of four similar interior beam-column subassemblages under the influence of the *P*-Δ effect. They concluded that for a lateral displacement ductility of 4, the *P*-Δ effect causes a decrease in the storey shear capacity of greater than 40%. This effect cannot be neglected in either analysis or design as it may lead to premature lateral instability.

3. Beam-column joint specimen details

Two reinforced concrete beam-column subassemblies, MX_2 and MX_4, were constructed with crossed, inclined bars in the joint region. The results were compared with the results of ten similar specimens previously reported by Tsonos *et al.* (1995). Thus, twelve specimens in four series are detailed in Table 3. The axial load for the series A specimens remained constant throughout the test. The axial load for the other specimen series M, MS and MX varied during the tests. The specimens in series A, M, MS and MX had dimensions as 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

Table 3 Description of beam-column connection specimens

Series	Speci-men	Column reinforcement	Beam reinforcement	Joint transverse reinforce-ment	Inclined rein-forcing bars	Concrete compres-sive strength (MPa)	$\frac{P_{max}}{P_b}$	Flexural strength ratio M_R	$\frac{\tau_{cal}}{\sqrt{f_c}}$	$\frac{\tau_{exp}}{\tau_{ult}}$	$\frac{\tau_{cal}}{\tau_{ult}}$
A	A_1	8Ø14	4Ø14	3Ø8	0	26	0.51	2.65	0.80	0.65	0.79
	A_2	4Ø14 4Ø10	4Ø10 4Ø12	4Ø8	0	31	0.45	1.30	0.87	0.62	0.79
	A_3	8Ø14	6Ø14	3Ø8	0	34	0.42	1.88	0.96	0.61	0.83
	A_4	8Ø10	8Ø14	3Ø8	0	34	0.42	0.72	0.70	0.66	0.61
M	M_1	8Ø14	4Ø14	3Ø8	0	25	0.94	3.20	0.70	0.77	0.71
	M_2	4Ø14 4Ø10	4Ø10 4Ø12	4Ø8	0	34	0.72	1.54	0.78	0.85	0.68
	M_3	8Ø14	6Ø14	3Ø8	0	27	0.95	2.00	1.00	1.10	1.01
	M_4	8Ø10	8Ø14	3Ø8	0	33.50	0.72	1.00	1.30	1.05	1.13
MS	MS_3	8Ø14	6Ø14	5Ø8	0	26	1.01	1.95	1.03	1.08	1.02
	MS_4	8Ø10	8Ø14	5Ø8	0	33.50	0.73	1.00	1.30	0.90	1.13
MX	MX_2	4Ø10	4Ø10 4Ø12	4Ø8	4Ø14	33	0.75	1.50	0.80	0.91	0.70
	MX_4	4Ø10	8Ø14	3Ø8	4Ø10	33	0.74	1.00	1.30	0.94	1.14

An overstrength factor a = 1.25 for the beam steel is included in the computations of joint shear stress $\tau_{cal} = \gamma_{cal}\sqrt{f_c'}$ MPa
Ø8, Ø10, Ø12, Ø14 = bars with diameters of 8 mm, 10 mm, 12 mm, 14 mm
Summary of specimens' steel yield stress in MPa,
Bar size: Ø8 = 495, Ø10 = 465, Ø12 = 530, Ø14 = 485.

as the second series (M) specimens M_i. The MS series specimens, however, had 70% more 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. The seismic performance of all these specimens was explained by the use of a new joint shear strength formulation (Tsonos 1999).

The principal variables of the testing program were: a) Column axial load, b) P-Δ effect, c) Normalized horizontal joint shear stress, γ, as defined by the ACI-ASCE Committee 352 (ACI 352R-1985), d) Transverse joint reinforcement, and e) Inclined reinforcing bars in the joint region. Two test set-ups were used. The tests involving the series A specimens were carried out in the

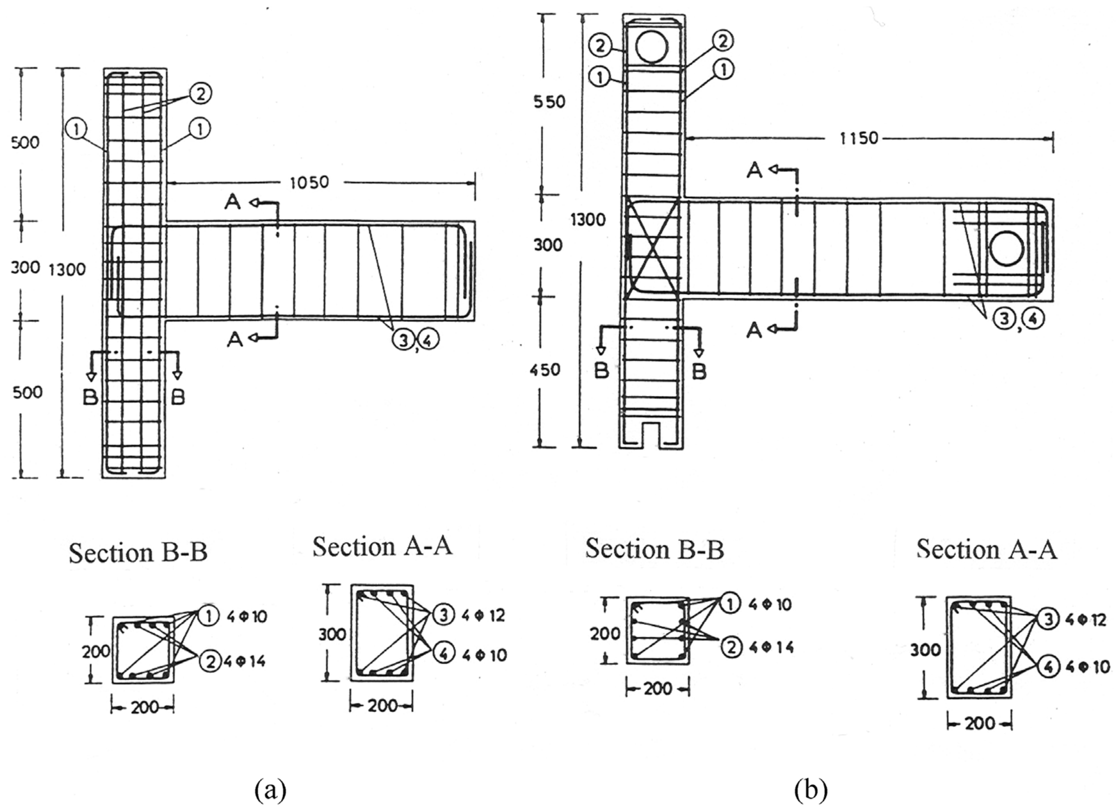
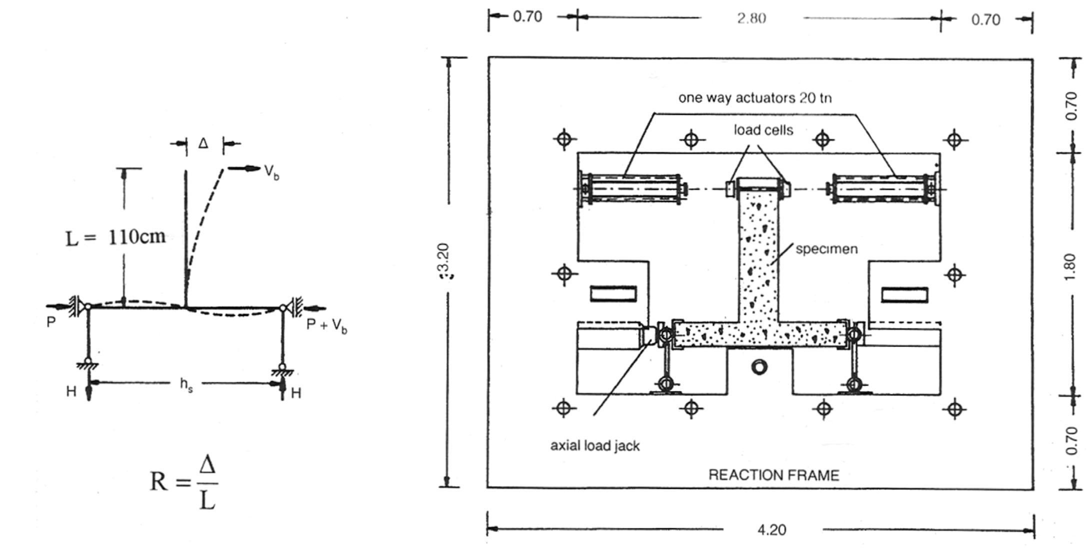


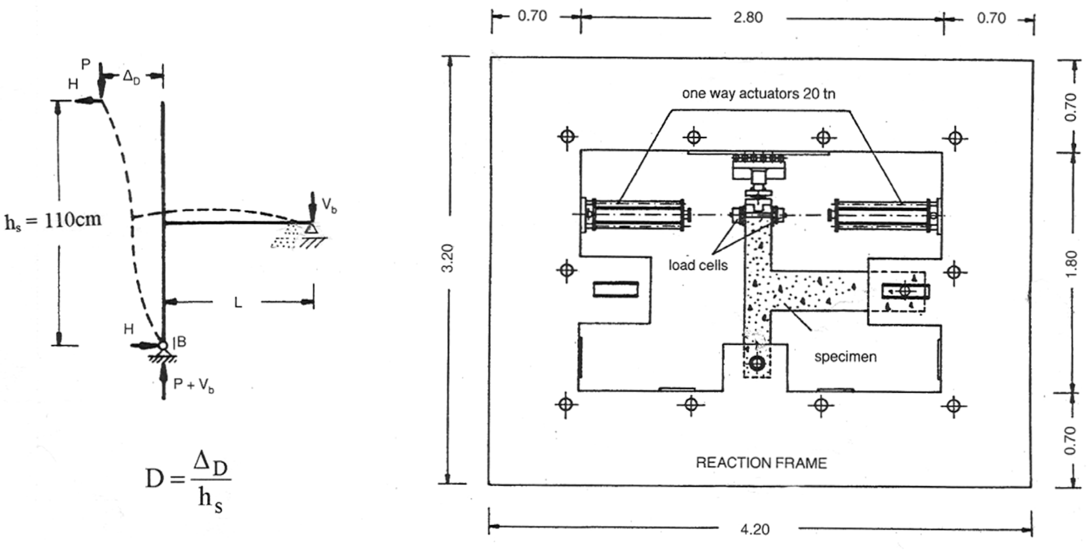
Fig. 1 (a) Typical specimen of type A, (b) Typical specimen of type MX (dimensions in mm)

testing frame shown in Fig. 2(a). Tests involving the series M, MS and MX specimens were carried out in the testing frame shown in Fig. 2(b).

All specimens were loaded transversely according to the displacement history shown in Fig. 3(a). Specimens in the A series versus the M, MS and MX series were loaded with different tests setups. Displacements were imposed at the beam tip in the A series and displacements were imposed at the top of the column in the M, MS and MX series. However, the drift angle R(%) of specimens of type A ($R = \Delta/L$) and the lateral drift D(%) of specimens of type M, MS and MX ($D = \Delta_D/h_s$) were equal because Δ (displacements imposed at the beam tip in the A series) = Δ_s (displacements imposed at the top of the column in the M, MS and MX series), and $L = h_s$ as is clearly shown in Fig. 2. The axial load for the series A specimens was kept constant at approximately $0.45P_b$ during the tests. It is essential to point out that in the experimental studies investigating the seismic performance of R/C beam-column subassemblages under axial force variations (Townsend and Hanson 1977, Uzumeri 1977, Agbabian *et al.* 1993, Higazy *et al.* 1996), axial loading in columns was kept constant during cyclic loading. However, it is understood that during an earthquake, the ground moves in two horizontal directions as well as vertically in a complex combination of frequencies and displacements. The lateral inertial forces produce overturning moments, which increase the column stresses on one side of the structure and reduce them on the other side (Penelis and Kappos 1997, Agbabian *et al.* 1994, Higazy *et al.* 1996).



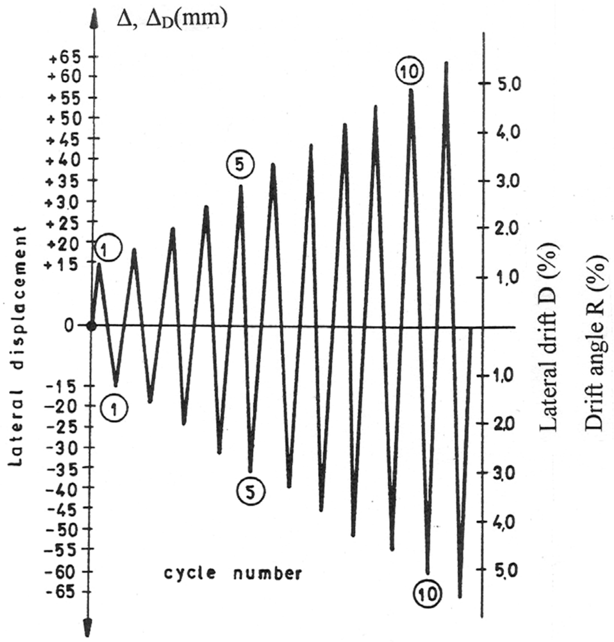
(a)



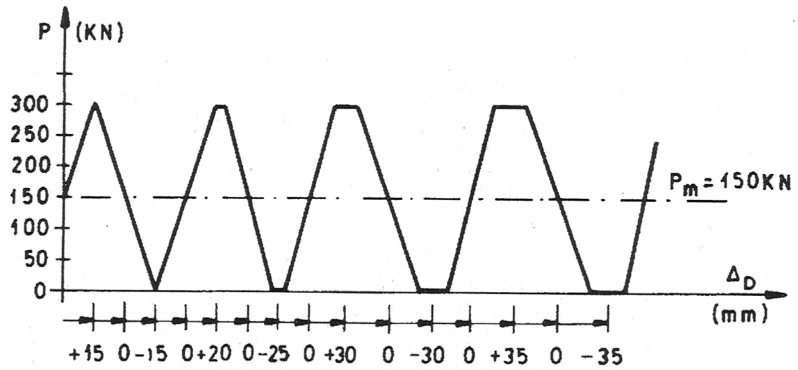
(b)

Fig. 2 (a) Test set-up for specimens of type A, (b) Test set-up for specimens of types M, MS and MX (dimensions in m)

Penelis and Kappos (1997) state that only a limited number of experimental studies (Jirsa *et al*. 1980, Abrams 1987, Tsonos *et al*. 1995) have addressed the problem of changing axial loading in columns and in beam-column joints subjected to cyclic shear, wherein the typical loading history was that high shear in the column or in the beam-column joint was accompanied by high compression while low shear was accompanied by low compression or tension. The axial load



(a)



(b)

Fig. 3 (a) Loading sequence, (b) Axial force history for specimens of types M, MS and MX

history of the study by Tsonos *et al*. (1995) was also used in the present study, Fig. 3(b).

Specimens of type M, MS and MX were subjected to variations in axial load during their earthquake-type loading. Their axial load history is shown in Fig. 3(b). The axial force is considered to vary linearly with lateral deflection up to a maximum or minimum value. Thus, the axial load of M, MS and MX series specimens was raised from 150 kN to 300 kN during Δ_D from 0 to +15 mm; then was reduced from 300 kN to 0 during Δ_D from +15 to −15 mm and was raised again from 0 to 150 kN during Δ_D from −15 mm to 0. This axial load variation was continued throughout the subsequent cycles of loading (Tsonos 1999).

4. Theoretical considerations

4.1 Theoretical model for predicting the beam-column joint ultimate shear strength

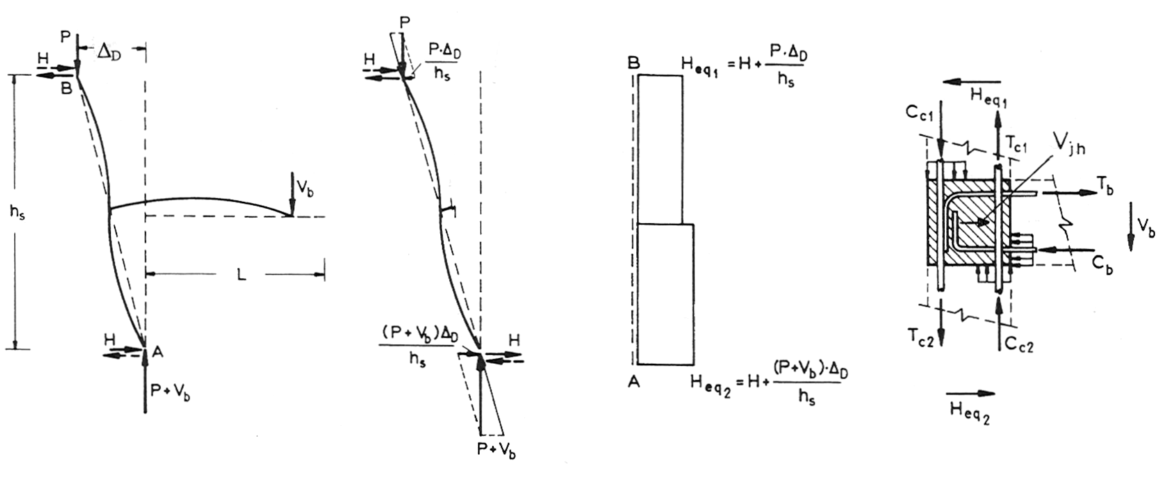
A formulation was developed at the Aristotle University in Thessaloniki, which gives the beam-column joint ultimate shear strength. This shear strength formulation can be used to predict the failure mode of the subassemblages and thus the actual values of connection shear stress. Therefore, when the computed joint shear stress is greater than or equal to the joint capacity, $\tau_{cal} = \gamma_{cal}\sqrt{f_c'}$ MPa $\geq \tau_{ult} = \gamma_{ult}\sqrt{f_c'}$ MPa, since the connection fails earlier than the beam(s), the expected and thus the predicted actual value of connection shear stress will be near $\tau_{ult} = \gamma_{ult}\sqrt{f_c'}$ MPa.

When the calculated joint shear stress is lower than the connection strength, $\tau_{cal} = \gamma_{cal}\sqrt{f_c'}$ MPa < $\tau_{ult} = \gamma_{ult}\sqrt{f_c'}$ MPa, since the connection permits its adjacent beam(s) to yield, the expected and thus the predicted actual value of connection shear stress will be near $\tau_{cal} = \gamma_{cal}\sqrt{f_c'}$ MPa.

More details about the above formulation can be found in references (Tsonos 1996a, 1997, 1999), where the validity of the formulation was checked using test data for 38 exterior and interior beam column subassemblages that were tested in the Laboratory of Reinforced Concrete at Aristotle University in Thessaloniki, as well as data from similar experiments carried out in the United States. This formulation was used to explain the experimental results of the 12 specimens in the present study.

4.2 Analysis for P-Δ effect

An exterior beam-column subassemblage of a sway reinforced concrete frame subjected to cyclic loading and the applied forces is shown in Fig. 4(a). The axial load P (in the upper column) and



(a) (b) (c) (d)

Fig. 4 (a) Exterior beam-column subassemblage of sway reinforced frame subjected to seismic loading showing the imposed forces, (b) Inclined and horizontal components of the columns' axial load, (c) Diagram of columns' shear forces, (d) Free body diagram for exterior beam-column joint of sway reinforced concrete frame

$P + V_b$ (in the lower column) can be replaced by inclined and horizontal force components as shown in Fig. 4(b). The first of these components acts parallel to the line joining the ends of the column and, assuming small deformations, is equal to P and $P + V_b$ in the upper and lower columns respectively. The horizontal component is equal to $(P \cdot \Delta_D)/h_s$ and $(P + V_b) \cdot \Delta_D/h_s$, respectively, at the upper and lower columns. When the column is to the right of vertical, the values of column axial force change from P and $P + V_b$ to $P + V_b$ and P, respectively.

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

$$V_{jh} = T_b \pm \frac{P \cdot \Delta_D}{h_s} - H \tag{1}$$

When the column is to the left of vertical (Fig. 4d) where Δ_D is the lateral displacement and h_s is the height of the column and,

$$V_{jh} = T_b \pm \frac{(P + V_b) \cdot \Delta_D}{h_s} - H \tag{2}$$

when the column is to the right of vertical.

The increase of column axial load increases also the joint shear stress (Park and Paulay 1984). Thus, beam-column connections designed to develop shear stress significantly lower than the joint ultimate strength (as defined by $\gamma_{ult}\sqrt{f_c'}$ psi) for axial loads derived by an elastic analysis for factored lateral loads, possibly subjected to variations of axial load during an earthquake, may develop shear stress very close to the joint ultimate strength, when values of column axial load are high and include the influence of the P-Δ effect. In this case shear failure of joints may occur.

4.3 Inclined bars

An experimental investigation of the behavior of external beam-column joints with inclined reinforcing bars under seismic conditions and under constant axial loading (approximately equal to 0.45 P_b) was conducted by Tsonos *et al.* (1992).

A simple technique to prevent these elements from failing in premature shear failure was implemented. The experimental program included fourteen specimens in seven series, as detailed in Table 4. All specimens had the same dimensions which were the same as those of the A series specimens in the present study (Fig. 1a).

Each of the first six series consisted of two exterior reinforced concrete beam-column subassemblages, one conventionally reinforced in the joint region (Type S specimen) and an identical specimen reinforced with inclined bars and hoop reinforcement in this region (Type X specimen). The reinforcing bars in the beams and columns were the same in both types of specimens. The specimens in each series differed only in reinforcement details in the joint region. The second specimen in each series was reinforced with four crossed, inclined bars bent diagonally across the joint core (as shown in Fig. 1b), instead of the four intermediate longitudinal bars in the column of the first conventionally reinforced specimen. These four intermediate bars are the vertical joint shear reinforcement of these specimens, as proposed by codes, such as Eurocode 8 (1993) and NZS 3101 (1982). All these series were used to determine the efficiency of the inclined bars when they replace an equal amount of vertical joint shear reinforcement.

The seventh series consisted of two specimens reinforced in the joint region with a different percentage of inclined bars compared with that of specimen X_6 (Table 4). The influence of the

Table 4 Summary of experimental program of the study by Tsonos *et al.* (1992)

Series	Specimen	Column reinforcement	Beam reinforcement	Joint transverse reinforcement	Vertical joint shear reinforcment	Inclined reinforcing bars
I	S_1	3∅14 3∅14	2∅14 2∅14	3∅8	4∅14	0
	X_1	"	"	3∅8	0	4∅14
II	S_2	2∅10 2∅10	2∅12 + 1∅10 2∅12 + 1∅10	3∅8	4∅14	0
	X_2	"	"	3∅8	0	4∅14
III	S_3	2∅10 2∅10	2∅12 + 2∅10 2∅12 + 2∅10	3∅8	4∅14	0
	X_3	"	"	3∅8	0	4∅14
IV	S_4	3∅10 3∅10	2∅14 + 2∅12 2∅14 + 2∅12	3∅8	4∅14	0
	X_4	"	"	3∅8	0	4∅14
V	S_5	2∅14 + 1∅10 2∅14 + 1∅10	4∅14 4∅14	3∅8	4∅14	0
	X_5	"	"	3∅8	0	4∅14
VI	S_6	2∅14 2∅14	4∅14 4∅14	3∅8	4∅14	0
	X_6	"	"	3∅8	0	4∅14
VII	X_7	2∅14 2∅14	"	3∅8	0	6∅10
	X_8	"	"	3∅8	0	4∅10

increase in the amount of inclined reinforcing bars in the seismic behavior of exterior beam-column joints is examined with the aid of this series of specimens.

The test results showed that the use of crossed inclined bars in the joint region is one of the most effective ways to improve the seismic resistance of exterior reinforced concrete beam-column joints. The improvement in the seismic resistance of this structural element was more noticeable as the inclined bar reinforcement increased.

The test results also showed that the presence of inclined bars in the joint introduces a new shear transfer mechanism, in addition to the two known mechanisms for conventionally reinforced joints, which remain active during the tests.

Plots of applied load versus displacement at the load point, and cracking patterns for representative specimens in the study by Tsonos *et al.* (1992) are shown in Fig. 5.

Two similar specimens to those of type M in the present study were subjected to a series of cyclic lateral loads plus variable axial load and P-Δ effect to simulate severe earthquake damage. The loading sequence and the axial force history for these two specimens was the same as that of specimens M in the present study (Fig. 3). The connections of both subassemblages exhibited premature shear failure during the early stages of seismic loading. The specimens were then repaired and strengthened by jacketing. Inclined bars in the form of collar stirrups were used in the

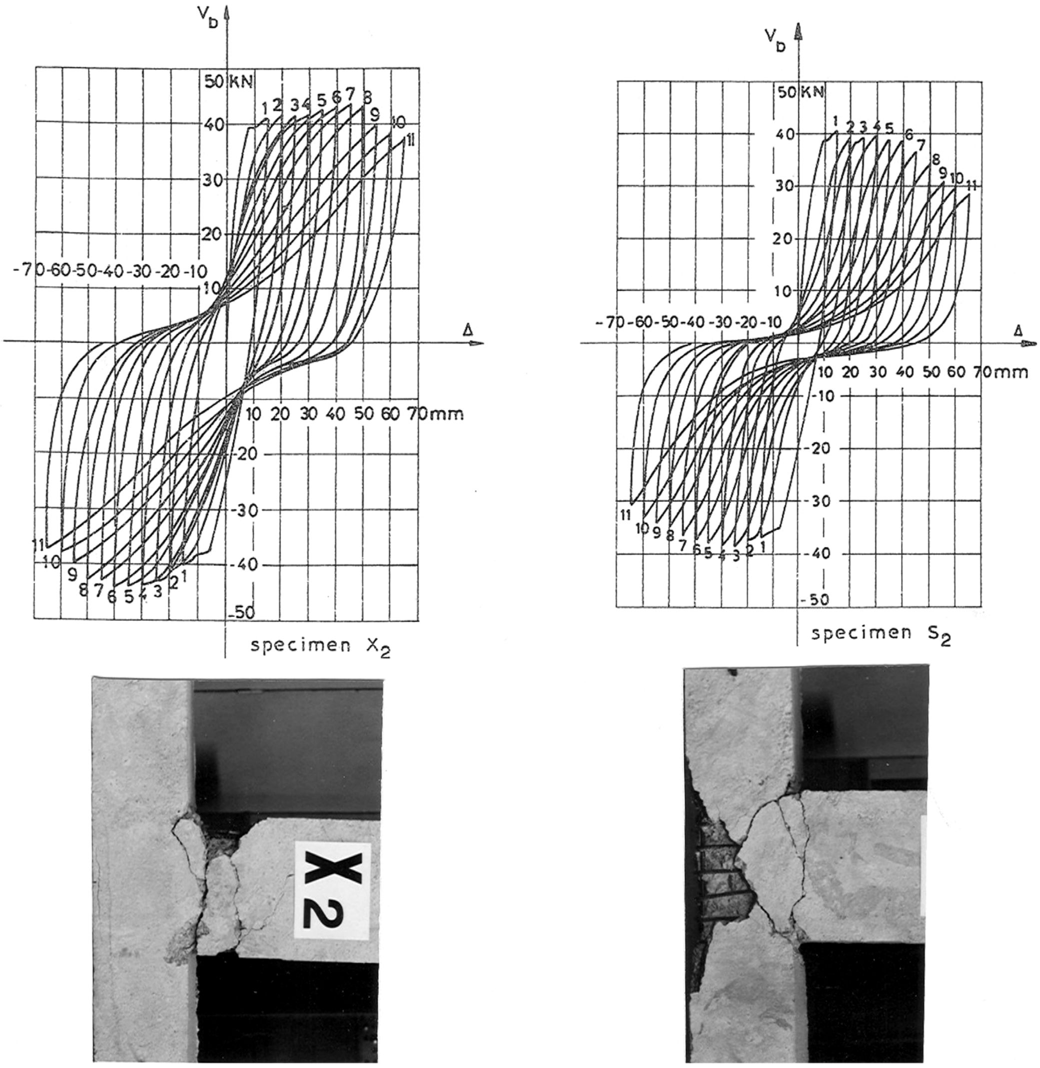


Fig. 5 Hysteresis loops and cracking pattern for specimens S_2 and X_2 (Tsonos *et al.* 1992)

joint region of the strengthened specimens instead of additional hoops. The strengthened specimens were then subjected to the same load history as that imposed on the original test specimens. The strengthened specimens exhibited higher strength, greater stiffness and better energy dissipation capacity than the original specimens. The failure mode of the strengthened specimens involved the formation of a plastic hinge in the beam near the column juncture and damage concentration in this region only (Tsonos 1999).

Yousef and Yakimov (1995), Karayannis *et al.* (1998) and Bakir (2003) also showed that inclined bars are a feasible solution for increasing the shear strength of the cyclically loaded beam-column joints. The use of inclined bars in reinforced concrete exterior beam-column joints are examined in the present work with regard to the decrease in the unfavourable effects of the P-Δ effect and axial load variations.

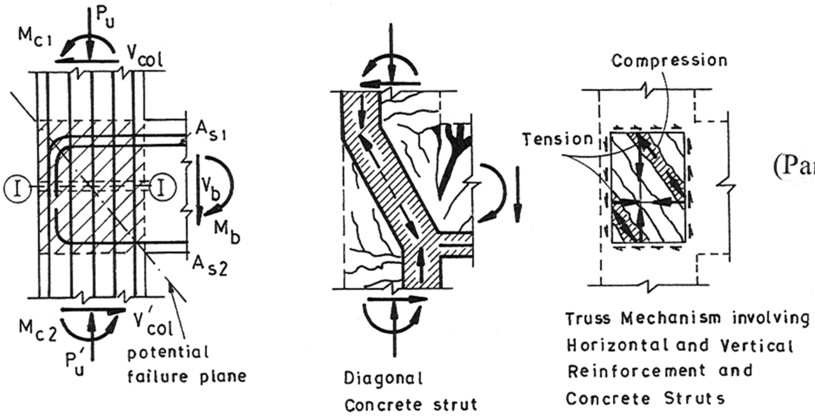


Fig. 6 External beam-column connection and the two mechanisms of shear transfer

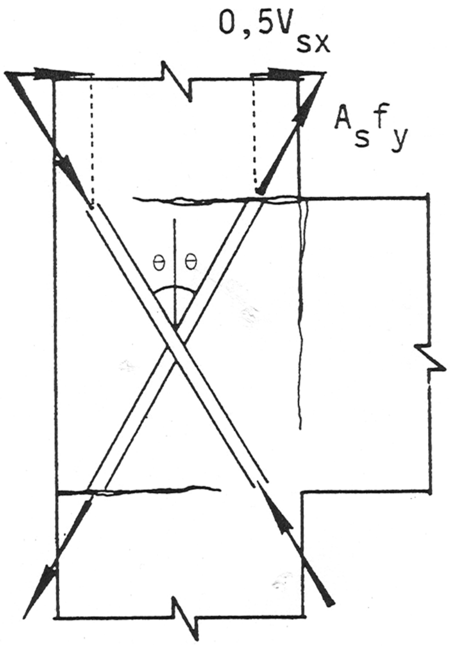


Fig. 7 Truss mechanism utilizing inclined bars

The main shear transfer mechanisms for Specimens MX_2 and MX_4 of the present study are the following:

a. Truss mechanism utilizing transverse reinforcement (Fig. 6);

b. Concrete diagonal compression strut (Fig. 6);

c. Truss mechanism utilizing inclined bars (Fig. 7).

The determination of the portion of shear carried by the third mechanism of the preceding list can be based on the statically determinate model whose equilibrium is illustrated by the force polygon (Fig. 7). After first yielding, the shear carried by the inclined bars is

$$V_{sx} = 2A_s \cdot f_y \cdot \sin\theta \tag{3}$$

where
A_s = the area of inclined bars
f_y = the yield stress of these bars and
θ = the inclination of these reinforcing bars to the column axis.

5. Discussion of test results

5.1 Failure modes

5.1.1 Specimens A and MX

In this mode there was formation of a plastic hinge in the beam near the column and more damage concentration in this region, but there was also little damage in the joint, with partial loss of joint concrete cover (Figs. 8, 10).

5.1.2 Specimens M and MS

In this mode, despite the appearance of hairline cracks during the first two or three cycles of loading, both in the joint and at the end of the beam, subsequent cycles resulted in an increase in the width of the cracks only in the joint region. The specimens' beams were almost intact at the conclusion of the tests (Figs. 8, 10).

Plots of applied load versus displacement at the load point, and cracking patterns for representative specimens studied in the program are shown in Figs. 8, 10.

A comparison of the loss of strength for the specimens of types A and M is shown in Fig. 9. This figure plots the percentage yield strength versus the displacement ductility. The yield load and displacement for each specimen were determined from the strain-gage data based on the yielding of the beam's longitudinal reinforcement at the face of the column. The average of the maximum positive and negative loads during each cycle was divided by the yield strength to obtain the percent yield strength. The displacement ductility, plotted as the abscissa in Fig. 9, is defined as the displacement at the end of each cycle divided by the yield displacement of the specimen. The yield displacement was calculated from the strain-gage data and corresponds to the displacement at which the yielding of the beam longitudinal reinforcement at the face of the column was recorded. It is clear that specimens of type A performed very well by maintaining their yield strength for displacement ductilities of almost 4.0. For specimens of type M, however, the load-carrying capacities were sharply reduced to values below its yield strength after a displacement ductility of 2.0 (Fig. 9). The performance of specimens of type M was theoretically predicted by Uzumeri (1977).

The highest values of axial load of approximately $0.85P_b$, in the upper half cycles of loading of specimens M were detrimental for the concrete struts of the two mechanisms in the joint of specimens M as was demonstrated by Uzumeri (1977). These large axial compressive forces of specimens M plus the P-Δ effect, also produced 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, which were very close to 1.00 (Table 3). As a consequence of the above, premature shear failure of the joints of these specimens was observed (Fig. 8). The load carrying capacity of specimens of type M was sharply reduced after the first two cycles of loading, especially in the

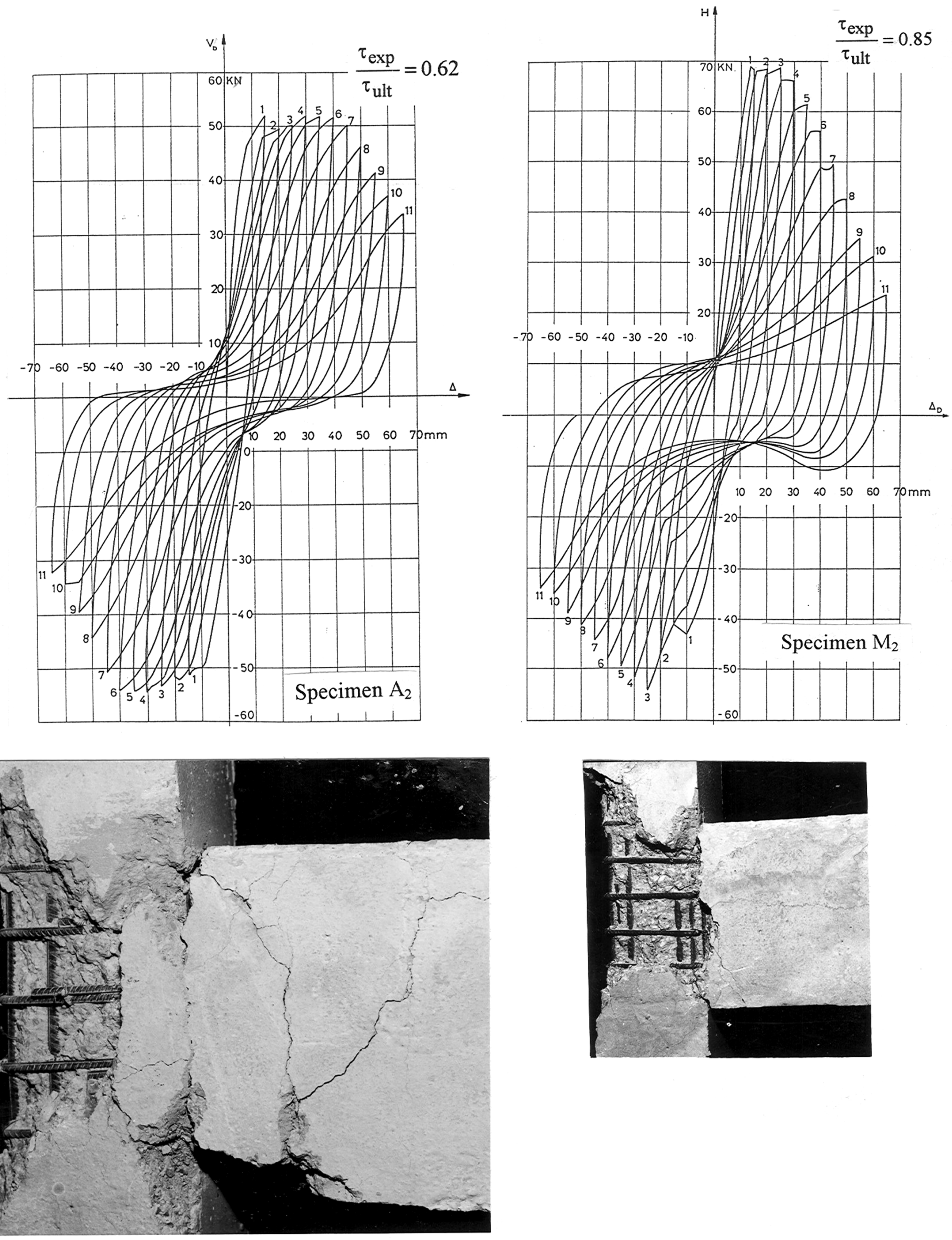


Fig. 8 Hysteresis loops and cracking pattern for specimens A_2 and M_2

upper half cycles. In the lower half cycles of specimens of type M, due to the low values of axial load, the load carrying capacity was reduced compared to that of their counterparts of type A (Fig. 8). This was demonstrated by Townsend and Hanson (1977), Agbabian *et al*. (1993) and Higazy *et al*. (1996). However, in the lower half cycles of specimens M, due to the low values of axial load, the

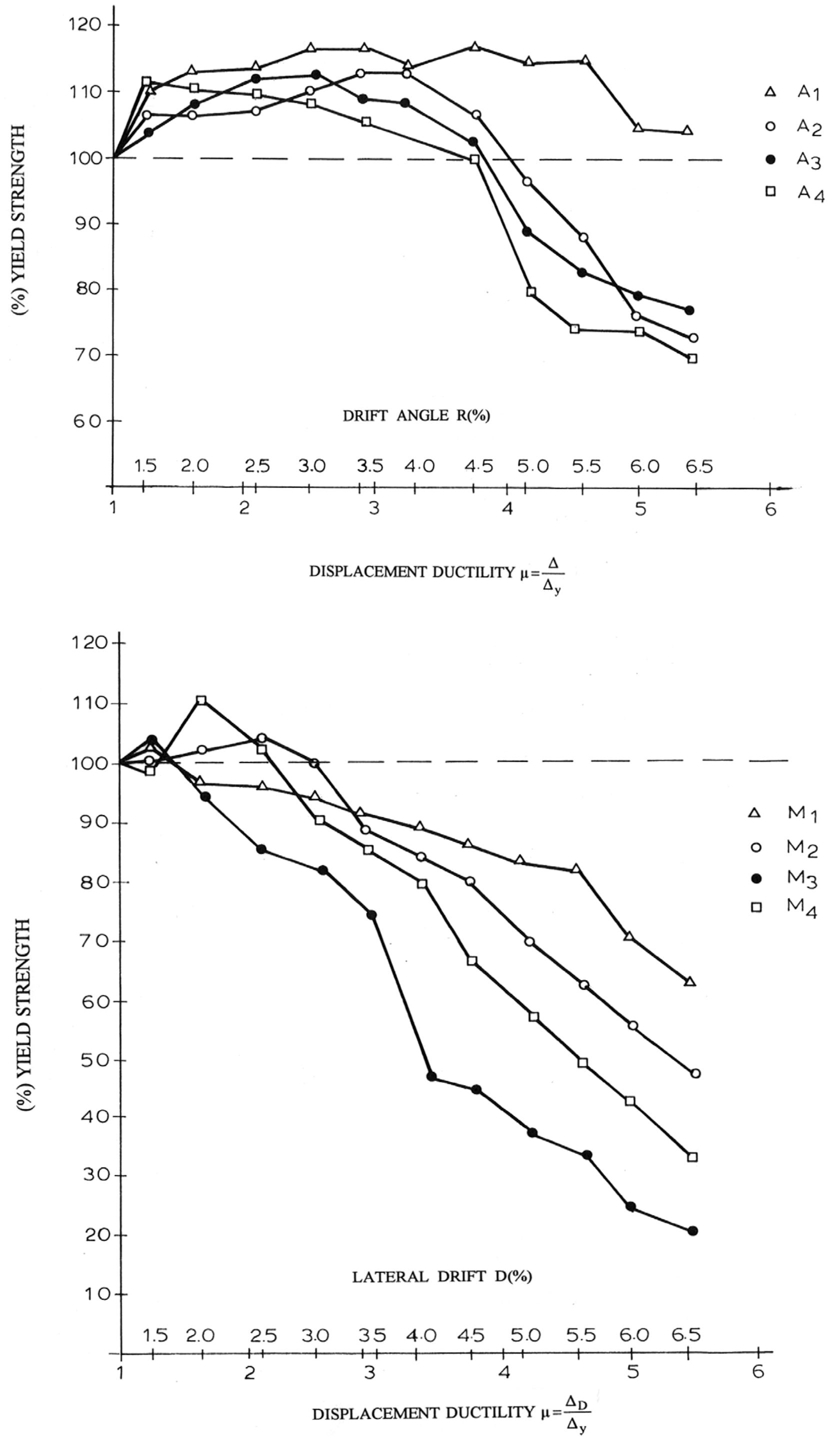


Fig. 9 Maximum load carried by each specimen of type A and type M at various displacement levels

influence of the P-Δ effect is significantly decreased and the values of joint shear stress were lower than the ultimate shear strength. Thus, a more stable behaviour was observed in these half cycles compared to that in the upper half cycles (Fig. 8). Specimens A which had constant axial load, approximately equal to $0.45P_b$, during the test, exhibited increased load carrying capacity both in the upper and lower half cycles 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 ($\gamma_{exp}/\gamma_{ult} \approx 0.65$, Table 3, Figs. 8, 9). Specimens M_3, M_4, MS_3 and MS_4 exhibited high joint shear stress: $\tau_{cal}/\tau_{ult} \geq 1.0$ or $\tau_{exp}/\gamma_{ult} \approx 1.0$ (Table 3). Specimens MS_3 and MS_4 had 70% more joint transverse reinforcement than specimens M_3 and M_4 respectively. The higher amount of joint transverse reinforcement of MS_3 and MS_4 resisted more joint shear stress than that resisted by the joint transverse reinforcement of M_3 and M_4. A comparison of the cyclic load carrying capacity between specimen pairs MS_3, MS_4 and M_3, M_4 indicated that an increase in the joint transverse reinforcement improves the overall behaviour of the specimens under high joint shear stress (Fig. 10).

The improved retention of strength in the beam-column subassemblages, as the values of the ratio τ_{cal}/τ_{ult} decrease, was demonstrated (Tsonos 1996a, 1999). It is worth noting that for $\tau_{cal}/\tau_{ult} \leq 0.50$ the beam-column joints of the subassemblages performed excellently during the tests and remained

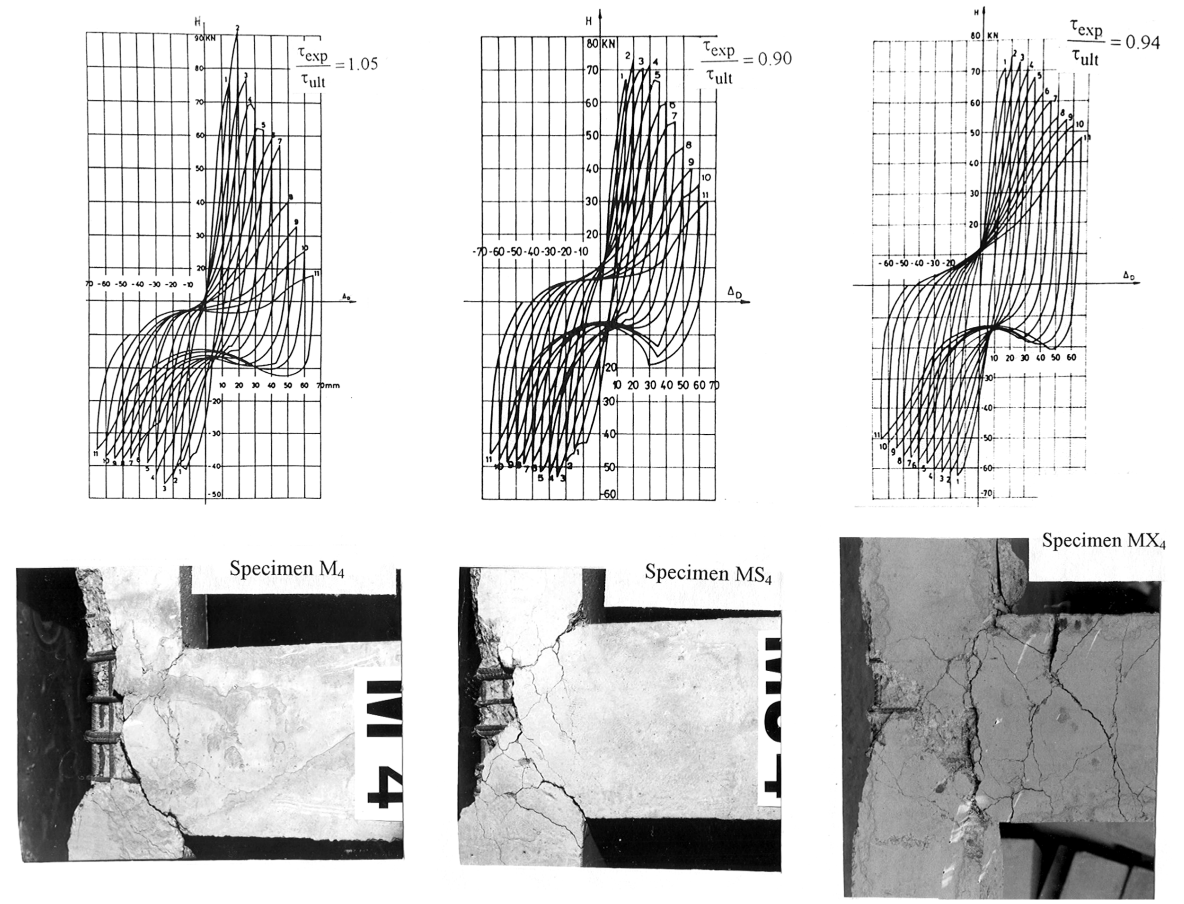


Fig. 10 Hysteresis loops and cracking pattern for specimens M_4, MS_4 and MX_4

intact at the conclusion of the tests. The joints of these subassemblages possessed the amount of transverse reinforcement required by the relevant codes (Tsonos 2001a, 2001b, 2002a, 2002b). In this case (for $\tau_{cal}/\tau_{ult} \leq 0.50$), additional joint transverse reinforcement will remain inactive and will produce a useless steel congestion. Specimens MX_2 and MX_4 were reinforced with four-crossed inclined bars bent diagonally across the joint core, (Fig. 1b). The hysteresis loops of specimens with inclined bars demonstrated increased strength, stiffness and energy dissipation capacity and less joint damage compared to those of their corresponding conventionally reinforced specimens M_2 and M_4 (Fig. 10). 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 (see Fig. 7). It was demonstrated that this mechanism could remain active throughout the test (Tsonos *et al.* 1992). 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 and by the comparison of the load-displacement envelopes of these specimens (Fig. 11). As shown in Fig. 10, the failure mode of

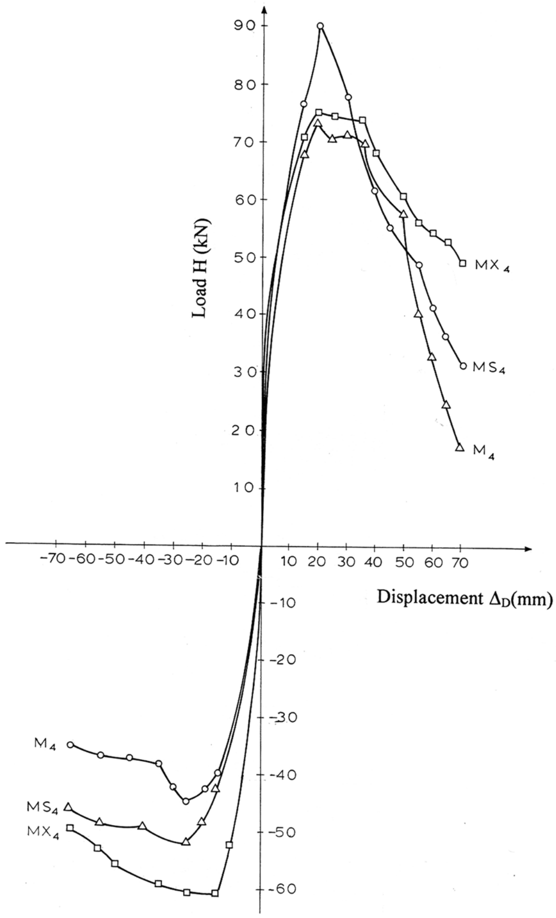


Fig. 11 Load-displacement envelopes of specimens M_4, MS_4, and MX_4

specimens MX_4 involved formation of a plastic hinge in the beam near the column juncture and more damage concentration in this region. On the contrary, in the specimen M_4, damage was concentrated only in the joint region. The beam of this specimen remained intact at the conclusion of the test. Specimen MS_4 exhibited the same failure mode as that of specimen M_4 but its joint damage was of a lesser degree.

5.1.3 Bar slippage

The hooked beam bars of all specimens in the present study had sufficient development lengths according to the requirements of the ACI-ASCE Committee 352, as is clearly shown in Table 5.

A major cause of the loss of stiffness in beam-column subassemblies is the pullout of the beam's longitudinal reinforcement from the joint.

Strain-gage measurements were used to determine beam bar pullout. Because the maximum displacement the specimens were subjected to increased with each cycle of loading, the strains in the beam's longitudinal reinforcement were expected to increase during each cycle of loading. If the maximum strains during each two consecutive cycles of loading remained the same or decreased, it was concluded that a pullout of the bar had taken place. Thus, the beam's longitudinal reinforcement in specimens of type A maintained adequate anchorage up to the 6^{th} cycle of drift angle R ratio of 4.0 percent. Slippage of these bars in specimens of type A was recorded after the 6^{th} cycle of drift angle R of 4.0 percent (see Fig. 12 for specimen A_2).

The beam's longitudinal reinforcement in Specimens of type M maintained adequate anchorage up to the 3^{rd} cycle of lateral drift D of 2.0 percent. Significant slippage of these bars was recorded after the 3^{rd} cycle of lateral drift D of 2.0 percent (see Fig. 12 for specimen M_2).

The beam's longitudinal reinforcement in specimens MS_3 and MS_4 maintained adequate anchorage up to the 4^{th} cycle of lateral drift D of 3.0 percent. Significant slippage of these bars was recorded after the 4^{th} cycle of lateral drift D of 3.0 percent.

Table 5 Development length l_{dh} of hooked beam bars terminating in the joints

Specimen	l_{dh} (mm) (1)	l_{dh} (mm) (2)
A_1	340	267
A_2	340	228
A_3	340	232
A_4	340	234
M_1	340	272
M_2	340	217
M_3	340	262
M_4	340	235
MS_3	340	266
MS_4	340	234
MX_2	340	235
MX_4	340	235

(1) provided values of l_{dh}
(2) required values by ACI-ASCE Committee 352 of l_{dh}
$l_{dh} = \alpha \cdot f_y \cdot$ (psi) $d_b/75\sqrt{f'_c}$ (psi)

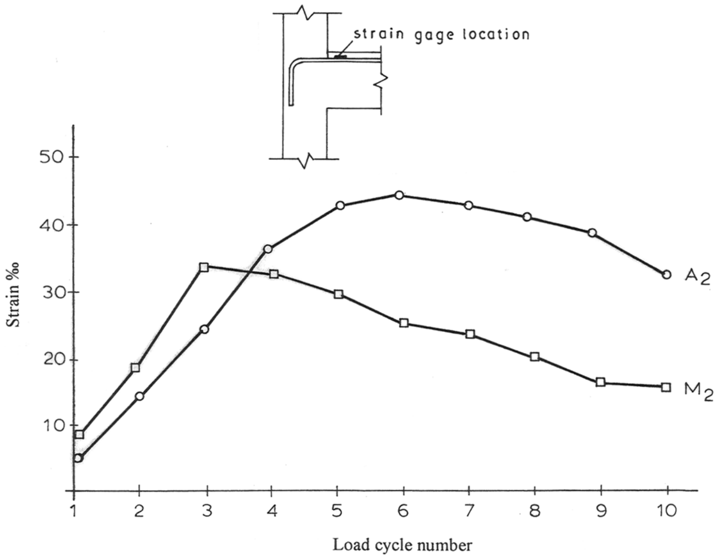


Fig. 12 Maximum strain during each cycle of loading in beam longitudinal reinforcement of specimens A_2 and M_2

The beam's longitudinal reinforcement in specimens MX_2 and MX_4 maintained adequate anchorage up to the 7th cycle of lateral drift D of 4.5 percent. Slippage of these bars of specimens MX_2 and MX_4 was recorded after the 7th cycle of lateral drift D of 4.5 percent.

6. Conclusions

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

1. A comparison of the seismic performance of specimens M with the performance of specimens A indicates that axial load changes and *P*-Δ effect during a seismic-type loading increase significantly the joint shear stress which results in more rapid deterioration in the beam-column joint earthquake resistance.
2. For high values of joint shear stress ($\tau_{cal}/\tau_{ult} \geq 1.0$), additional joint transverse reinforcement significantly improves the beam-column joint strength and ductility and enhances the overall behaviour of the subassemblage. It is worth noting that for low values of joint shear stress, and especially for $\tau_{cal}/\tau_{ult} \leq 0.50$, the beam-column joints reinforced with the transverse reinforcement required by the codes performed excellently during the tests and remained intact at the conclusion of the tests. In this case, the additional joint transverse reinforcement will remain inactive.
3. External beam-column joints with inclined bars performed considerably better than those with conventional reinforcement under seismic type loading with variations of axial load and *P*-Δ effect. 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.

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