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SOME RECENT RESEARCH IN NEW ZEALAND INTO ASPECTS OF THE SEISMIC RESISTANCE OF PRESTRESSED CONCRETE FRAMES

R. Park* and K.J. Thompson**

1. INTRODUCTION

Prestressed concrete has been widely used for structures carrying gravity loads but has not had the same acceptance for structural systems which resist seismic loads. Part of this caution in the use of prestressed concrete for earthquake resistant structures has been due to the paucity of experimental and theoretical studies of prestressed concrete structures subjected to seismic type loading. A survey of the design applications and research conducted on the seismic resistance of prestressed concrete structures was published in 1970(1). More recent design and research information was presented at the FIP Symposium on Seismic Structures which was held in Tbilisi, USSR, in 1972(2). A summary of the papers of the Tbilisi symposium is contained in a report of the FIP Commission on Seismic Structures which was presented to the 7th Congress of the FIP in New York in 1974(3).

In the past there has been a lack of detailed building code provisions for the seismic design of prestressed concrete. For example, the building code of the American Concrete Institute, ACI 318-71(4), contains special provisions for the seismic design of reinforced concrete structures but does not have corresponding provisions for prestressed concrete. However, the Seismic Committee of the N.Z. Prestressed Concrete Institute has recently prepared a set of design recommendations(5), and the FIP Commission on Seismic Structures is currently preparing design recommendations(6).

The seismic design recommendations of the N.Z.P.C.I.(5) are based mainly on the special provisions of ACI 318-71(4) and the research conducted at the University of Canterbury by R. W. G. Blakeley(7) and the authors(8,9). This paper will briefly summarize the recent research work completed by the authors.

2. TESTS ON BEAM-INTERIOR COLUMN JOINTS

Ten beam-interior column plane frame assemblies were tested under static cyclic loading which simulated seismic loading (8,9). The beams had an 18 in x 9 in (457 mm x 229 mm) cross section and the columns had a 16 in x 12 in (406 mm x 305 mm) cross section. Each test unit represented the joint region of a plane frame extending approximately between the points of contraflexure of the members. Fig. 1 shows a unit under test. The columns were subjected to an axial load and the beam and column ends were loaded to simulate the shears and bending moments induced by seismic loading in the elastic and inelastic ranges. The columns were designed to have a greater flexural strength than the beams and hence plastic hinging was expected to occur in the beams.

As an example of the test results, Figs. 2 and 3 show the measured load-deflection characteristics and the observed damage of Unit 4 which had a prestressed beam containing three tendons (one at the top, one at mid-depth and one at the bottom of the section, giving a uniform prestress of 1123 psi (7.74 MPa)) and of Unit 5 which had a prestressed beam containing two tendons (one at the top and one at the bottom of the section giving a uniform prestress of 1198 psi (8.25 MPa)). The inelastic deformations of Unit 4 were due to plastic hinge rotation in the beam, whereas in Unit 5 a joint core shear failure commenced after the first inelastic load cycle and the inelastic deformation concentrated in the joint core during the subsequent load cycles. Fig. 4 shows the measured load-deflection characteristics of Unit 7 which had a partially prestressed beam containing nonprestressed steel (at the top and bottom of the section) and three prestressed tendons (one in the top, one at mid-depth and one at the bottom of the section, giving a uniform prestress of 675 psi (4.65 MPa)), and with approximately the same flexural strength as Units 4 and 5. The inelastic deformations of Unit 7 were due to plastic hinge rotation in the beams and the improved performance of this unit compared with Units 4 and 5 was due to the nonprestressed steel acting as compression reinforcement in the plastic hinge regions.

The tests illustrated the need for closely spaced stirrups placed with minimum cover in the plastic hinge regions of beams to avoid excessive loss of flexural capacity caused by reduction of the concrete section when crushing of unconfined concrete commences. It is recommended that stirrup spacing should not exceed 4 in (102 mm) or ¼ of the effective depth, as recommended for reinforced concrete beams in Appendix A of ACI 318-71(4). Units 4, 5 and 7 had a stirrup spacing of 3½ in (89 mm). The tests also indicated the value of nonprestressed compression steel in maintaining flexural capacity after concrete crushing, and in increasing the energy dissipation, providing bond failure does not occur through the joint core.

A critical aspect of the test results was the joint core behaviour. Shear reinforcement in the joint core had been designed using the method recommended for reinforced concrete in Appendix A of ACI

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* Professor of Civil Engineering, University of Canterbury, Christchurch.

** Assistant Engineer, Ministry of Works and Development, Wellington.

318-71(4). In this method the horizontal shear induced in the joint core by the beam internal forces and column shear is assumed to be carried by a mechanism involving the concrete (from aggregate interlock, etc.) plus a mechanism involving the hoops assuming 45° diagonal tension cracking and concrete struts, as in structural members. In all the units tested the beams reached at least 95% of their theoretical flexural strength in the first inelastic loading cycle accompanied by yielding of the joint core hoops in some units. For those units in which the joint core hoops yielded, further inelastic loading cycles resulted in a degradation of the joint core shear strength due to repeated opening and closing of diagonal tension cracks in alternating directions. In these units the shear strength of the joint core governed the strength of the unit and the inelastic deformation of the unit occurred mainly in the joint core. Thus although the ACI 318-71 Appendix A approach allowed the attainment of the required joint core shear strength satisfactorily in the first inelastic load cycle, in some units degradation of joint core shear capacity occurred in subsequent inelastic loading cycles. Thus the ACI 318-71 Appendix A approach for joint core shear strength cannot be regarded as being adequate for plane frames subjected to intense cycles of seismic loading. In those units without a prestressing tendon at mid-depth joint core shear failure always occurred illustrating the benefit to joint core behaviour to be gained from the presence of a central tendon.

In the tests the critical diagonal tension crack was observed to run from corner to corner of the joint core (see Figs. 2, 3 and 4). It is recommended that horizontal shear reinforcement for the joint core should be designed to carry the maximum horizontal shear force in the joint core V_j across the corner to corner crack using

$$V_j = nA_v f_y \qquad (1)$$

where n = number of layers of shear reinforcement, A_v = area of shear reinforcement in each layer, and f_y = yield strength of the shear reinforcement. The contribution of the concrete to the shear strength should be neglected because of the degradation of its contribution found during the previously described reversed loading tests.

It may be that neglecting the contribution of the concrete is conservative when either the column contains bars around the perimeter of the section which aid shear transfer (the test units had four bar columns), or when the column axial load is high, or when the joint core is crossed by several prestressing tendons down its depth, or when beams enter the columns on all four faces (the test units were plane frames). Further tests are required to establish the influence of these variables.

The test results from the ten test units may be seen reported in detail elsewhere (8,9).

3. THEORETICAL MOMENT-CURVATURE CHARACTERISTICS OF PRESTRESSED AND PARTIALLY PRESTRESSED CONCRETE SECTIONS SUBJECTED TO MONOTONIC FLEXURE

The available ductility of a section is illustrated by the shape of the moment-curvature ($M - \phi$) curve. The $M - \phi$ curve obtained for monotonic loading gives a good approximation for the envelope curve for cyclic loading providing strength degradation due to concrete damage is not significant. Theoretical $M - \phi$ curves can be obtained using idealized stress-strain relationships for concrete and steel by satisfying the requirements of strain compatibility and equilibrium for the section while incrementing the extreme fibre strain. The idealized stress-strain relationships for prestressing steel and nonprestressed mild steel used in this study were obtained by fitting equations to measured experimental curves. The stress-strain relationship used for the confined core concrete is that proposed by Kent and Park(10) which allows for the effect of transverse steel content on the ductility of the concrete. For the cover concrete (outside the transverse steel) a stress-strain curve was used which was closer to that for unconfined concrete. Good agreement was obtained between the theoretical and measured experimental $M - \phi$ curves for the ten test units(8).

The theoretical approach was used to study the effect of several beam section variables. The section studied was 18 in (457 mm) deep by 19 in (229 mm) wide containing No. 3 (9.5 mm dia.) stirrups at 3½ in (89 mm) centres, with 1½ in (38 mm) cover to the stirrups. The findings are discussed below.

Fig. 5 shows the theoretical $M - \phi$ curves obtained for the section eccentrically prestressed by a tendon near the extreme tension fibre. The reduction of the available curvature ductility with increasing content of prestressing steel has led the authors(8) and Blakeley(7) to recommend that for seismic design the following requirement should be observed

$$\rho_p \, f_{ps}/f'_c \leqslant 0.2 \qquad (2)$$

where $\rho_p = A_p/bd$, A_p = area of prestressing steel, b = width of section, d = effective depth of section, f_{ps} = stress in prestressing steel at the flexural strength, and f'_c = compressive cylinder strength of concrete. For the section shown in Fig. 5 Eq. 2 requires the gross area steel ratio $p_p = A_p/bt$, where t = section overall depth, to be $p_p \leqslant 0.0046$ and it is evident from the curves of that figure that this requirement will ensure reasonable curvature ductility. ACI 318-71(4) allows 50% more prestressing steel than Eq. 2 (i.e. $p_p \leqslant 0.0069$ for the section of Fig. 5), but it is evident from Fig. 5 that the requirement of Eq. 2 is more reasonable than the less severe ACI requirement if the moment is to remain near maximum over a large range of curvature. Eq. 2 has been adopted by the Seismic Committee of the N.Z.P.C.I.(5).

Fig. 6 shows that with prestressing steel present in the compression zone the theoretical curvature ductility of the section is not reduced by increase in prestressing steel content. This is a consequence of the prestressing steel acting as compression steel at large curvatures, providing that steel is restrained against buckling by the surrounding concrete and transverse steel. Successive cycles of reversed flexure may cause concrete damage leading to buckling. To avoid excessive concrete damage it would appear reasonable to require that all beam sections are capable of reaching a specified curvature at a given extreme fibre concrete strain. This in general requires a limitation on the maximum allowable neutral axis depth since for sections with tendons at various levels down the depth it is difficult to set a limiting value for $\rho_p f_{ps}/f'_c$ because tendons at different levels lead to different M - ϕ curves. It has been shown previously(7,8) that the curvature at ultimate for a section with any tendon arrangement will be at least equal to that of the section with all tendons placed near the extreme tension fibre if the following requirement is observed : $a \leqslant 0.2t$, where a = depth of equivalent concrete rectangular stress block and t = section overall depth. This requirement has been adopted by the Seismic Committee of the N.Z.P.C.I.(5).

It is of interest to compare M - ϕ curves for sections with different arrangements of prestressing steel. In seismic design moment reversals will require many sections to have both negative and positive moment strength and hence tendons will often exist near both extreme fibres of the section and near mid-depth. Fig. 7 shows theoretical M - ϕ curves for the section with up to five tendons symmetrically distributed down the depth. The total prestressing steel content is the same for each of the five cases, being 0.00696 of the gross concrete section. For the case of all steel concentrated in a single central tendon, N = 1, the moment capacity is more sensitive to a deterioration of the compressed concrete and a significant reduction of moment capacity occurs at high curvatures. However there is little difference in the moment capacity for two or more tendons and such sections are able to maintain near maximum moment capacity at high curvatures. Therefore two or more tendons are to be preferred.

The effect of increasing prestressing steel content on the curvature ductility of partially prestressed sections is also of interest. Fig. 8 shows the M - ϕ curves for the section with symmetrically placed nonprestressed steel top and bottom having a total area of 0.0124 of the gross concrete section, and various contents of prestressing steel. The figure shows that increase in the content of prestressing steel results in an increase in flexural strength without significant reduction in ductility.

Other M - ϕ analyses also have shown that for the section studied a 3 to 4 in (76 to 102 mm) spacing of closed stirrups gave reasonable curvature ductility and that greater spacing was undesirable, and that the concrete cover thickness to the stirrups should be made as small as possible to avoid a significant reduction in moment capacity when the cover concrete crushes.

4. MOMENT-CURVATURE CHARACTERISTICS OF PRESTRESSED AND PARTIALLY PRESTRESSED CONCRETE SECTIONS SUBJECTED TO CYCLIC FLEXURE

To obtain a better indication of the performance of sections subjected to seismic loading M - ϕ curves for cyclic flexure are necessary. Such theoretical M - ϕ curves can be obtained using idealized cyclic stress-strain curves for concrete and steel. In this study the idealization for concrete assumed the monotonic stress-strain curve as an envelope; the steel idealization took the Bauschinger effect into account. The theoretical cyclic M - ϕ curves were obtained by dividing the section into a number of discrete horizontal elements and tracing the M - ϕ loop by incrementing the extreme fibre strain and satisfying strain compatibility and equilibrium. The effect of buckling of prestressing steel at high compressive strains was taken into account using a tangent modulus approach.

Figs. 9 and 10 show compared theoretical and measured experimental cyclic M - ϕ curves for the beams in the plastic hinge regions of Units 4 and 7. Good accuracy results from the theoretical approach but the amount of computer time used can be considerable. The theoretical and experimental curves indicated that the inclusion of nonprestressed steel in a prestressed section results in a fattening of the M - ϕ hoops and thus may considerably increase the hysteretic energy dissipation.

The cyclic M - ϕ loops can be idealized for use in dynamic analyses to save computational time. The idealized model for reinforced concrete suggested in this study is based on the Ramberg Osgood relationship (Fig. 11a) and that for prestressed concrete is based on a series of straight lines (Fig. 11b). These two models can be combined to model partially prestressed sections (Fig. 11c). Such idealizations, with empirical factors to define the exact shape of the loops, give good agreement with measured experimental cyclic M - ϕ curves(8) and are suitable for use in non-linear dynamic analyses.

5. EARTHQUAKE RESPONSE OF SIMPLE PRESTRESSED, PARTIALLY PRESTRESSED AND REINFORCED CONCRETE SYSTEMS

A comparative study was made of the response of prestressed, partially prestressed and reinforced concrete single degree of freedom systems responding to various earthquake ground motions. The displacement response was calculated using a step-by-step numerical integration method(8). The idealized non-linear cyclic load-displacement curves used were of similar shape to the idealized cyclic M - ϕ curves of Fig. 11. The strength of the systems was calculated from the seismic loading given by NZSS 1900 Chapter 8(11) and the load factors given by NZS 3101P:1970(12). The first 15 seconds of three earthquake records were used: El Centro 1940 N-S component, and artificial earthquakes(13) A-2 and B-2. The displacement ductility factor μ is defined in this study as the displacement divided by the "first yield" displacement, where the "first yield" displacement is defined as the displacement at the point of intersection of the elastic slope of the load-deflection curve and the horizontal line at ultimate load. This definition gives an identical first yield displacement for all loop shapes with the same initial stiffness and strength and thus is a good basis for comparison of prestress, partially prestressed and reinforced concrete systems. Fig. 12

shows the variation of the maximum displacement ductility factor demand with period T and damping ratio λ for prestressed, partially prestressed and reinforced concrete systems responding to the El Centro 1940 N-S earthquake. The displacement ductility demand is shown to decrease with increasing T and increasing λ. Note the high displacement ductility demand for small period structures. The coefficient α is the ratio of the flexural strength contribution from the prestressing steel to the total flexural strength of the section; the coefficient β is the ratio of the flexural strength contribution from the nonprestressed steel to the total flexural strength of the section.

It is well known that the lower hysteretic energy dissipation of a prestressed concrete system compared with a reinforced concrete system of the same strength, initial stiffness and damping ratio, will generally result in a greater deflection response of the prestressed concrete system to a severe earthquake. This feature is illustrated in Figs. 12 and 13. In this study the maximum displacement of the prestressed concrete system was on average found to be 1.3 times that of the reinforced concrete system; however, this ratio showed wide variation from the average value of 1.3, ranging between 0.7 and 2.4 for the range of cases studied of $\lambda = 0.02$ to 0.10 and T = 0.3 to 2.1 seconds. The ratios of the average displacement ductility demand of the fully prestressed system for the A-2 and B-2 earthquakes to that for the El Centro earthquake was 2.14 and 0.6, respectively. The same two ratios for the reinforced concrete system were 2.11 and 0.90.

It can be concluded that the different load-displacement characteristics of prestressed and reinforced concrete structures do not affect the displacement ductility demand as much as the initial period of vibration, when responding to a given earthquake with given damping ratio. On this basis the use of different seismic loading for the design of prestressed and reinforced concrete structures seems unnecessary, providing the drift requirements are satisfied.

6. CONCLUSIONS

This study, along with previous studies, gives clear evidence that properly detailed prestressed concrete frames will give satisfactory seismic load resistance. The results of these studies have been incorporated in the design recommendations of the Seismic Committee of the N.Z.P.C.I.(5).

7. ACKNOWLEDGEMENTS

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9. SYSTEME INTERNATIONAL EQUIVALENTS

Imperial units are used in the figures in the following pages. SI equivalents of the imperial units used are:

1 in = 25.4 mm
1 kip in = 113 Nm
1 psi = 0.00689 MPa.

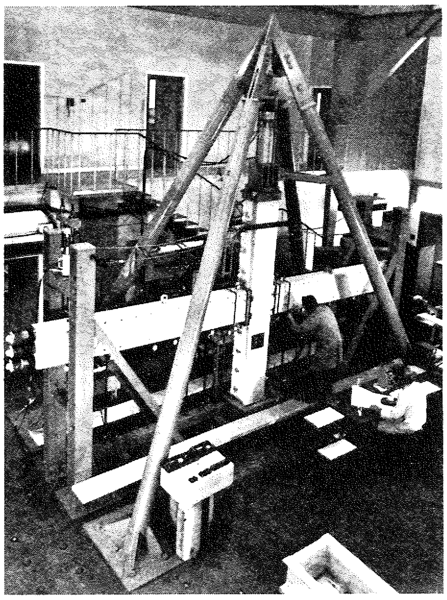
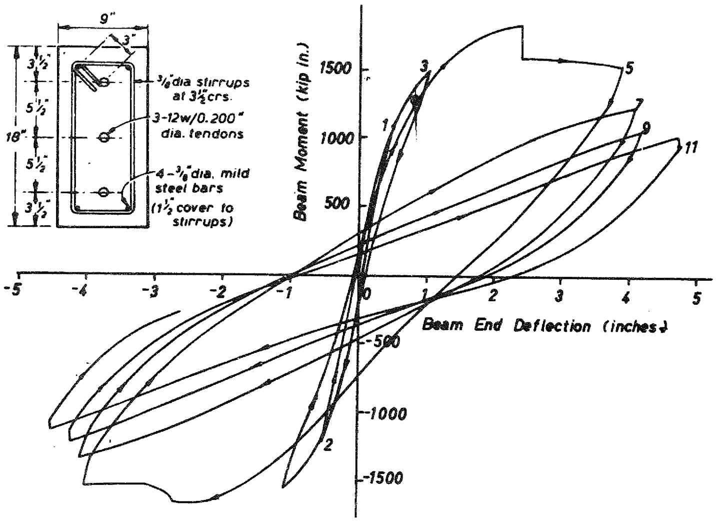
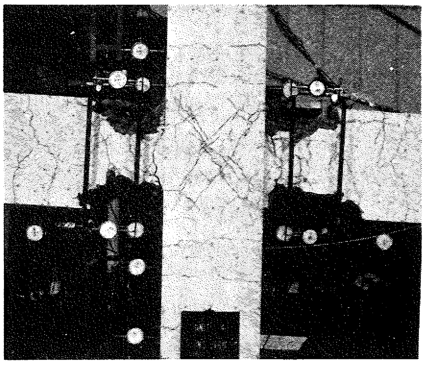


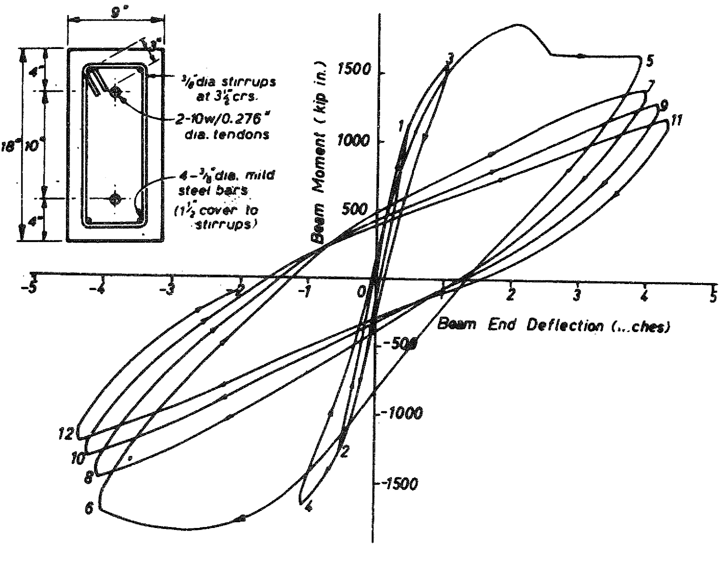
Fig. 1 Unit in Frame During Testing

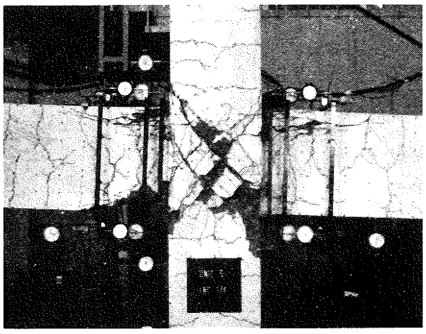




Unit 4 at Maximum Deflection (End of loading run 12)

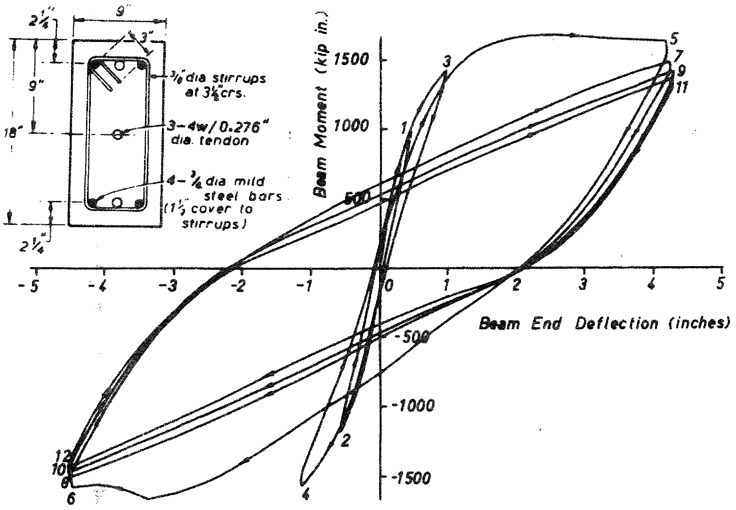
Fig. 2 Unit 4 : Beam Moment at Column Face Versus Beam End Deflection

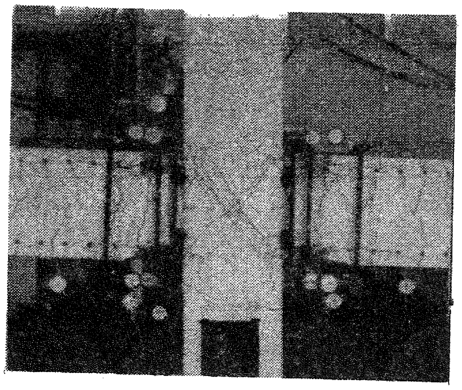




Unit 5 at Maximum Deflection (End of loading run 12)

Fig. 3 Unit 5 : Beam Moment at Column Face Versus Beam End Deflection





Unit 7 at Maximum Deflection (End of loading run 12)

Fig. 4 Unit 7 : Beam Moment at Column Face Versus Beam End Deflection

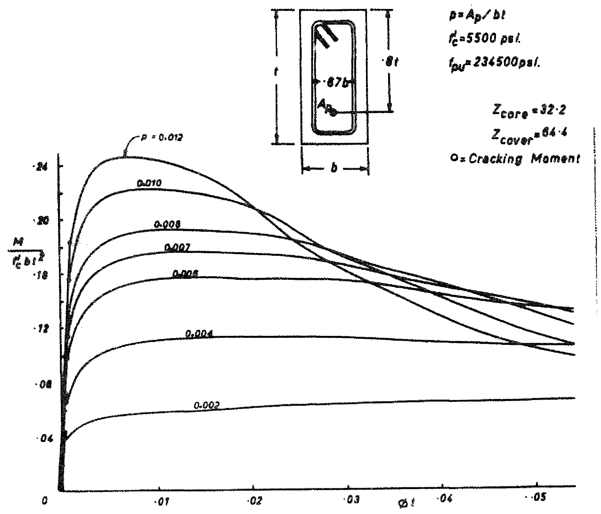


Fig. 5 Moment-Curvature Curves for Section With Various Contents of Eccentrically Placed Prestressing Steel

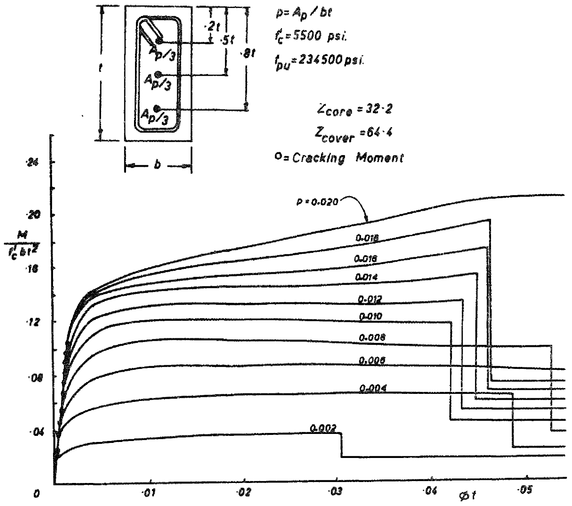
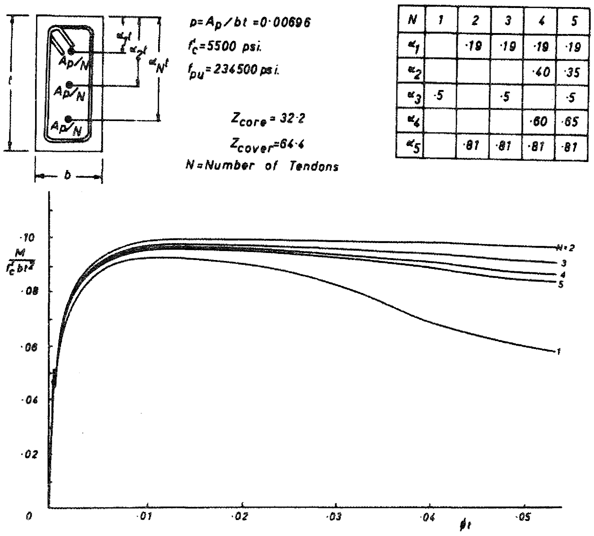


Fig. 6 Moment-Curvature Curves for Section With Three Symmetrically Placed Prestressing Tendons



N	1	2	3	4	5
α_1		·19	·19	·19	·19
α_2				·40	·35
α_3	·5		·5		·5
α_4				·60	·65
α_5		·81	·81	·81	·81

Fig. 7 Moment-Curvature For Section With Various Numbers of Symmetrically Placed Prestressing Tendons

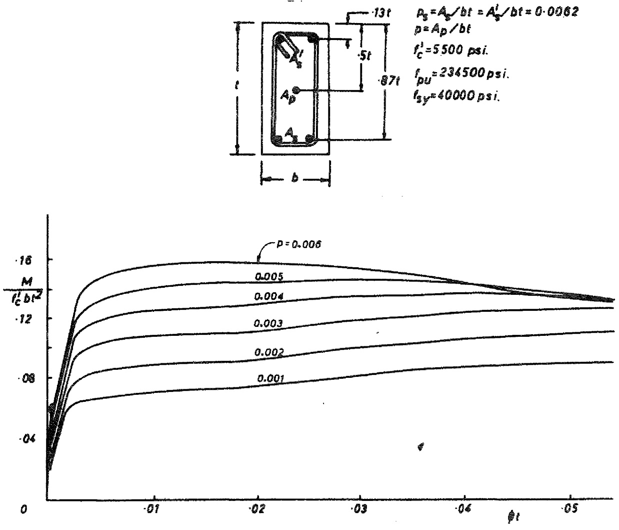


Fig. 8 Moment-Curvature Curves For Partially Prestressed Section

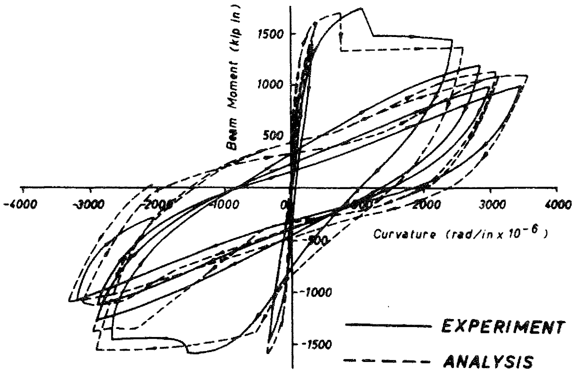


Fig. 9 Unit 4 : Theoretical and Experimental Moment-Curvature Curves

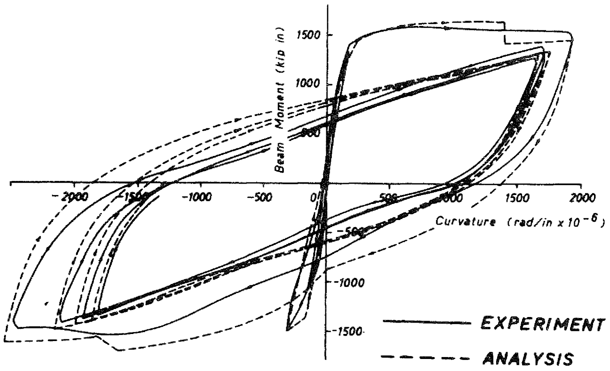
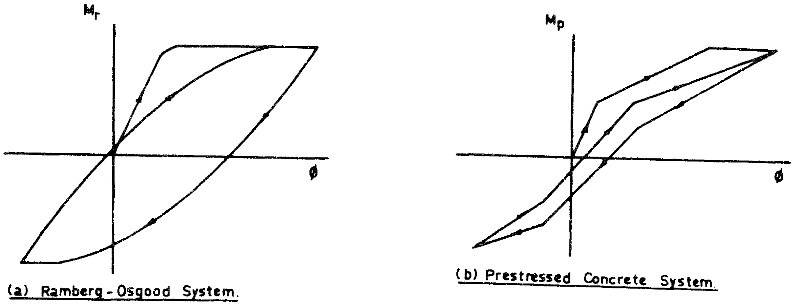
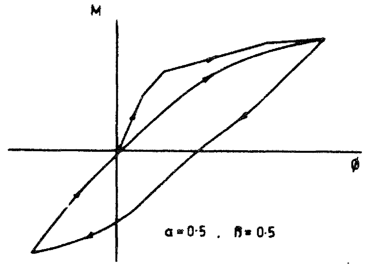


Fig. 10 Unit 7 : Theoretical and Experimental Moment-Curvature Curves



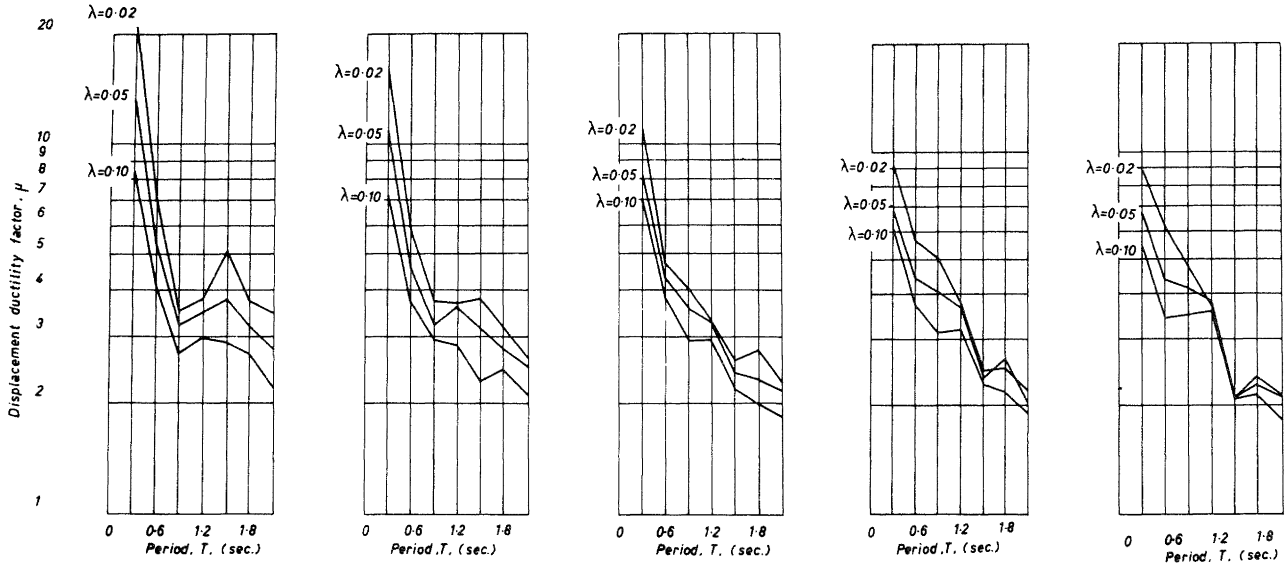
(a) Ramberg-Osgood System.

(b) Prestressed Concrete System.



(c) Partially Prestressed Concrete System.

Fig. 11 Moment-Curvature Idealizations For Reinforced, Partially Prestressed and Prestressed Concrete



(a) prestressed concrete ($\alpha=0$, $\beta=1$)

(b) partially prestressed concrete ($\alpha=0.25$, $\beta=0.75$)

(c) partially prestressed concrete ($\alpha=0.5$, $\beta=0.5$)

(d) partially prestressed concrete ($\alpha=0.75$, $\beta=0.25$)

(e) reinforced concrete ($\alpha=1$, $\beta=0$)

Fig. 12 Maximum Displacement Ductility Factor Demand for Prestressed, Partially Prestressed and Reinforced Concrete Systems for Zone A Non Public Buildings and El Centro 1940 N-S Earthquake

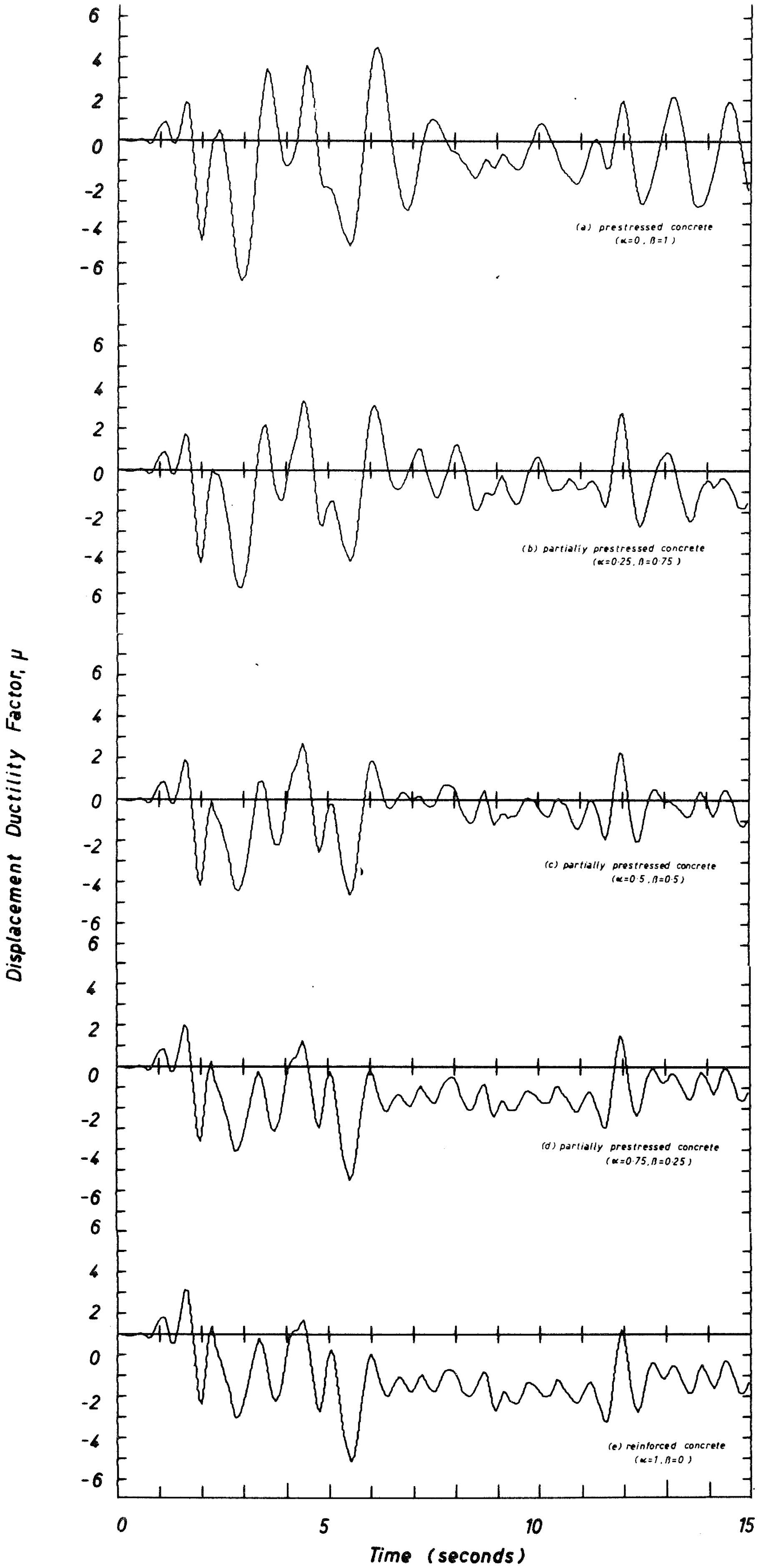


FIGURE 13: DISPLACEMENT DUCTILITY RESPONSE FOR ZONE A NON PUBLIC BUILDINGS AND EL CENTRO 1940 N-S EARTHQUAKE

PERIOD = .6 SECONDS AND DAMPING RATIO = .02