EDITORIAL

ART? SCIENCE?

Whenever, in any field of endeavour, there is theory and practise, there is controversy. Usually this is harmless, trivial, good natured banter between practical people and theoretical "eggheads", the groups recognising their interdependence and acknowledging that when theory and practise differ there should be co-operation to resolve the difference, for that is progress.

So it is in engineering. But occasionally the extremist wing of the practical faction is provoked, generally by some more or less esoteric theory the eggheads have attempted to foist on it, into serious debate about whether engineering is predominantly art or science. Debate is futile; but it is not always sterile. Too much heat and discord is generated.

Consider, for example, the recent move by a dissident group of artists to oust researchers and academics from the ACI Building Code Committee. Perhaps this is a tongue-in-cheek exercise, not intended to be taken too seriously; but the instigators have been persistent enough to force it to a vote. (The result is not known at the time of writing.)

The group will not find it difficult to attract support from users of ACI 318-71, because this new edition of a respected Code contains among its many innovations some that are impractical and a few that are impracticable. Many of the new provisions have been explained in ably written papers and in the Commentary, available with the Code, and illustrative worked examples of design problem solution have been published. But the drafting committee apparently failed to recognise an essential difference between a text-book style treatment and real design work. Moreover, it was obviously insensitive to the mood of designers who resent such an enormous increase in design effort as is required for compliance with the new rules. Review authorities will consider whether insistence on compliance with some of the rules is not more likely to be damaging to design that it is to be constructive, because designers, compelled to track through a labyrinth of detail will lose something from the overall appreciation of the basic mechanics of their structures that it is so essential to keep intact.

Extremists amongst the practical engineers are those who use "egghead" derisively rather than affectionately, even perhaps enviously, as the moderates do when talking of academics and researchers. The typical extremist is proud of his intuitive ability and prefers intuition to scientific appraisal. He does not need many warnings, the most recent and clear of which is given in the Australian Commission report into the causes of a major bridge collapse, that engineering judgement, even when exercised by highly skilled and experienced people, can be tragically and fatally misleading. He would benefit from recognising that the structural intuition he lays claim to was developed from the science he once practiced, including those parts of it which were empirical, and were not necessarily less scientific for being that. Were he to do this, he might realise that, because his intuitive sense has not benefited from enough recently made rational studies, it will be quite as rusty as his science. Engineering intuition is not a native thing.

It is good to have frequent reminders that there is very much in engineering that is not yet rational, and some that might never be; but undue emphasis on the art content, particularly when it implies criticism of the science, simply encourages the incompetent.

Academics and researchers need our help in making the product of their work palatable to designers. Moves like that of the ACI dissidents and similar events in the history of the development of earthquake engineering are unhelpful.

L. Andrews

PRESTRESSED CONCRETE SEISMIC DESIGN

R.W.G. Blakeley

SUMMARY

The approach of design and research engineers to the use of prestressed concrete in primary seismic resistant structural elements is reviewed. Some results of recent research into the ductility of prestressed concrete members and the inelastic seismic response of prestressed concrete frames are presented. Also, the principles and recommendations of the FIP-CEB and others for seismic resistant design are given and current New Zealand design practice is summarised. Finally, the question of suitable load factors for prestressed concrete structures is discussed.

1. INTRODUCTION

Although prestressed concrete has gained popularity in recent years for use in structural components resisting gravity loading such as bridge decks and precast floor units, it has not met with such ready acceptance for primary seismic resistant elements such as shear walls and frames. From time to time investigators, for example Nakano[1] (1964), Sutherland[2] (1965), and Lin[3] (1970), have expressed confidence in the ability of prestressed concrete structures which have been designed according to a conventional code approach to withstand severe earthquakes, but design engineers have generally remained cautious. The principal reasons for caution have been firstly, the lack of information on the practical performance of such structures in earthquakes, secondly, a fear that prestressed concrete is a brittle material without significant available post-elastic deformations, and thirdly, the shortage of research evidence on the inelastic seismic response of prestressed concrete structures. These three points of concern will be discussed in subsequent sections.

2. EARTHQUAKE BEHAVIOUR OF PRESTRESSED CONCRETE STRUCTURES

A survey of the literature shows that structures incorporating significant amounts of prestressed concrete have been involved in few major earthquakes. However, the following observations have been made:

2.1 Skopje, 1963

One case of damage to a single storey prestressed concrete framed structure was reported. Crushing of the concrete in the pretensioned columns created difficulties in repair, for while the crushed concrete could be replaced to restore the vertical load carrying capacity, the prestress could not readily be restored. An auxiliary bracing system was therefore necessary to provide adequate lateral stiffness.

2.2 Alaska, 1964

Of some 28 structures in Anchorage employing precast prestressed concrete elements, five suffered partial or total collapse. However, there appeared to be no failures of the prestressed concrete members themselves. Instead the supporting structure, which was built of traditional materials, collapsed or connections were ineffective. Sutherland[2] observed 13 structures incorporating prestressing which suffered no damage other than minor cracking of the blockwork. Included in these were two buildings using prestressed concrete tees and columns as frames.

2.3 Niigata, 1964

Characteristic of this earthquake was the considerable settlements and displacements of foundations. Nevertheless, prestressed concrete structures in the area, predominantly bridges, behaved satisfactorily.

2.4 Caracas, 1968

None of the 12 structures in Caracas with significant amounts of prestressing were affected, although these structures were not in the area of high damage.

2.5 San Fernando, 1971

There appear to have been no prestressed concrete framed structures within the area of high damage from this earthquake. A number of bridges with prestressed concrete box girder superstructures were damaged including three bridges that collapsed. In general, failure was due either to the sliding of the superstructure off the supports at hinge joints due to the absence of holding-down or linkage bolts, or failure of the reinforced concrete piers. However, another possible reason for the distress shown arises because of the inability of prestressed concrete bridges to carry large upward forces. The vertical accelerations of the span may have been sufficient to effectively remove a large percentage of the dead load; thus causing upward deflections of the girder under prestressing uplift alone with resulting severe section cracking.

+ This paper was originally presented at the Seminar "Structural Design for Earthquakes" held at the University of Auckland in August 1972.

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3. DUCTILITY OF PRESTRESSED CONCRETE MEMBERS

It is well known that dynamic analyses of the elastic response of structures, using earthquake acceleration records, have shown that a structure is subjected to considerably greater loads than are provided for by the equivalent static load design coefficients recommended by codes. This means that structures must be capable of developing large post-elastic deformations if they are to survive severe earthquakes. However, as yet most building codes do not offer detailed provisions to ensure that adequate ductility may be obtained in prestressed concrete members. The following experimental and analytical studies of the ductility of prestressed concrete members should therefore be of interest to design engineers.

3.1 Beam - Column Assemblies Under Cyclic Load

A series of tests was conducted on four full size precast, prestressed concrete beam-column assemblies under reversed cyclic loading of high intensity[4]. The columns were pretensioned and the beams post-tensioned with grouted tendons which passed through the column into an external anchor block. Moist pack mortar joints were formed between the elements. A unit under test is shown in Fig. 1. A representative axial load was applied to the column, and moments were applied to the joint by loading the end of the beam with screw jacks. The consensus of research information appears to justify the use of this static cyclic loading to represent the rapid load reversals of an earthquake. Fig. 2 shows the experimental moment-displacement curves for a unit which formed a plastic hinge in the beam, and had transverse reinforcement satisfying the shear requirements in the commonly used codes for prestressed concrete, for example A.C.I. 318-71[5], with no special provision for ductility. The unit suffered little damage in the fifteen prescribed simulated earthquake loading cycles (see numbers on figure) and exhibited high ductility in the final cycle when the unit was loaded to the limit of available deflection in the test rig. The displacement sustained at the end of the beam on downward loading represents an inter-storey deflection of approximately 10 inches, with little loss of moment capacity. On the final upward loading the unit suffered a progressive failure due to the top beam cable in compression pushing through the column. This behaviour could be avoided in practice by using corrugated metal ducts to prevent the bond failure between the cable duct and the column. Other units designed to form plastic hinges in the columns also showed considerable ductility.

It was found that there was little difference between the behaviour of corresponding units with transverse reinforcement only for shear, and those with special transverse reinforcement for ductility, the requirements of the SEAOC code[6] being used as a guide in the latter case. However, the axial load levels on the column were reasonably low (less than 20% of the axial load capacity), and confinement would be more important at higher axial load levels. It was significant that in all cases the column region within the joint behaved very well (see Fig. 1), whereas in similar tests on reinforced concrete members[7] considerable distress has been suffered at this point. The confining effect of the prestress and the external anchor block no doubt helped the performance of the joint.

3.2 Analytical Study of Ductility

An analytical method was developed to determine the moment-curvature relationships of general prestressed concrete members under monotonic load[8]. It has been shown[9] that the curve for monotonic load is almost colinear with the envelope of cyclic loading curves, for a particular prestressed concrete member, and thus this analysis may be used to study the effect of practical section variables on the ductility of members under earthquake loading.

3.2.1 Prestressing Steel Area Ratio

The effect of variation of prestressing steel area ratio, $p=A_s/bD$, is shown in Fig. 3. As expected, the general pattern of the curves shows an increase in moment capacity but a decrease in ductility as the prestressing steel area increases. The A.C.I. 318-71[5] code gives the following maximum steel area which should not be exceeded if a brittle failure from over reinforcement is to be prevented:

$$\frac{A_s\, f_{su}}{bdf'_c} \leqslant 0.3 \tag{1}$$

This limit corresponds to beams with p=0.00694 in Fig. 3. A study of the curves of this figure indicates that this limiting p value results in reasonable ductility, but that to ensure adequate ductility for seismic design it may be desirable to reduce the 0.3 on the right hand side of equation (1) to say 0.2.

3.2.2 Transverse Reinforcement

The theoretical moment-curvature plot for a beam with two different stirrup contents is shown in Fig. 4. Crushing of the concrete begins at a surface strain of approximately 0.004 as marked on the figure, but substantial ductility is available beyond this point until failure occurs at an assumed fracture strain in the steel of 0.04. It is interesting to note that the effect of confinement of concrete is not large on beams with reasonably low prestressing steel contents. Even the beam with light stirruping showed considerable ductility.

3.2.3 Distribution of Prestressing Steel

The effect of distributing the prestressing steel within a section is demonstrated in Fig. 5. The number and position of the tendons were varied, while keeping the magnitude of the total prestressing force constant and retaining an axial line of action as is needed to resist seismic moment reversals at a frame joint. Clearly, for ductility considerations it is desirable to have the prestressing steel distributed into at least two locations within the section.

3.2.4 Axial Load

Fig. 6 shows the effect on the ductility of prestressed concrete column sections due to varying axial load levels. Of particular significance is the curve plotted for the axial load limit specified by the SEAOC code[6] for reinforced concrete:

$$\frac{P}{bD} \not> 0.12 f_c' \qquad (2)$$

above which the total ultimate moment capacity of the columns must be greater than the total ultimate moment capacity of the beams at a particular joint. From a study of Fig. 6 this limit appears to be a useful guide to ensure ductility in prestressed concrete structures also.

3.2.5 Nomograms for Design

The nomograms in Figs. 7 and 8 allow calculation of the available curvatures prior to crushing in prestressed concrete beams and columns, for varying prestressing steel area ratios and axial loads. Plots of the ratio of curvature at a surface concrete strain of 0.004 (approximate crushing strain) to curvature at first cracking against the dimensionless term pf_s'/f_c' are given for beams with eccentric and concentric prestress in Fig. 7 and columns with a range of axial loads in Fig. 8. The stress in the steel due to prestress alone was assumed to be approximately $0.6 f_s'$. The Z values in Fig. 7 reflect the degree of confinement of the concrete from transverse steel, and it is evident that up to a concrete strain of 0.004 the degree of confinement has little effect. It should be noted that considerably greater curvatures are available beyond crushing if required under a severe earthquake.

4. SEISMIC RESPONSE OF PRESTRESSED CONCRETE STRUCTURES

An important factor affecting the response of a vibrating structure is its capacity to dissipate energy. The sources of energy dissipation may be classified as either that arising from hysteresis under cyclic load in the structural members, or that represented by viscous damping. These characteristics for prestressed concrete are as follows:

4.1 Hysteretic Energy Dissipation

Cyclic loading tests on prestressed concrete members by Nakano[1] (1964) and Spencer[10] (1966) produced moment-rotation characteristics which could be idealised by the bilinear elastic system shown in Fig. 9 (a). Large elastic recovery is evident. For comparison, the traditional idealisation for reinforced concrete under cyclic loading has been an elasto-plastic system as shown in Fig. 9 (b). For loading in a particular direction the areas under the moment-rotation curves represent the energy absorption. On unloading the member this energy is largely released as kinetic energy for the bilinear elastic system, whereas in the elasto-plastic system it is largely dissipated by plasticity. The area within the hysteresis loop is a measure of the energy dissipation capacity of the system. Although both these idealisations are rather crude, they serve to illustrate that in this respect prestressed concrete is at a disadvantage relative to reinforced concrete.

Recent tests[4] have shown the cyclic flexural characteristics of prestressed concrete members at rather greater deformations than earlier tests[1,10]. Fig. 10 shows the shape of the experimental moment-rotation curves for the plastic hinges in the column of a beam-column unit which had only transverse reinforcement for shear. Plastic rotations occurred in the column hinge above the joint for downward loading at the end of the beam and in the hinge below the joint for upward loading. In the first five cycles there was little damage to the members, only a small area within the hysteresis loops was apparent, and the curves could in fact be idealised by a bilinear elastic system. Crushing of the cover concrete commenced during cycle 9 for both directions of loading and resulted in significant energy dissipation and stiffness degradation. It is interesting to note that in one theoretical study[11] the maximum rotations sustained by members in a large prestressed concrete frame under a severe earthquake were only five times the rotation at first cracking. For comparison, in Fig. 10 (a) the rotation at the peak of downward load half-cycle 5 is six times the rotation at first cracking. The figure shows that if greater rotations should in fact be sustained there would be a reserve of energy dissipation available, although the members would then be structurally damaged with consequent difficulty of repair back to a fully prestressed condition.

Inomata[12] (1969) and Entrican[13] (1969) have tested the behaviour under cyclic load of prestressed concrete beams containing additional mild steel reinforcement. The non-prestressed steel provided a valuable source of energy dissipation when it yielded. However, this type of member poses construction problems to make the non-prestressed steel continuous through the joints of precast frames.

4.2 Damping Tests

There have been few measurements of the equivalent viscous damping of full size prestressed concrete buildings. However, one test by Hisada and Nakagawa[14] (1956) on a two storey structure found values of from 3% to 5% critical viscous damping for vibrations of small amplitude, while for large non-linear vibrations the damping was from 6% to 10% of critical. The percentage critical viscous damping found for reinforced concrete structures under small amplitude vibrations is generally higher than the corresponding figure above, the difference being principally due to the delay in cracking in the prestressed concrete case.

4.3 Dynamic Response Analyses

It has generally been assumed that a prestressed concrete framed structure will undergo a greater response to a particular earthquake than a comparable reinforced concrete structure, because of the lower viscous damping ratio and energy dissipation capacity within the members of the former as discussed above. The following analytical studies have considered this question.

4.3.1 Multistorey Structures

Spencer[11] (1969) verified the above assumption in a study of the non-linear dynamic responses to the El Centro 1940, N-S, earthquake of reinforced and prestressed concrete versions of a twenty storey framed structure. However, although higher lateral displacements were obtained for the prestressed concrete structures, the section ductility requirements of their members were markedly lower than those of the reinforced concrete structures. Spencer attributed this in part to the longer plastic hinge lengths assumed for prestressed concrete. The results suggest that a prestressed concrete structure, similar to that analysed, would suffer no structural damage under a severe earthquake, but non-structural damage arising from larger lateral displacements may be greater than with a reinforced concrete frame. Evidence from more such analyses is required before this could be regarded as conclusive.

4.3.2 Single-Degree-of-Freedom Structures

A study was made[9] of prestressed concrete single-degree-of-freedom portal frame structures responding dynamically to the El Centro 1940 earthquake, N-S component, and compared with the response of similar reinforced concrete structures. Because more is known about the earthquake behaviour of reinforced concrete structures at this stage, such comparisons serve a useful purpose. The structural system which was considered for this analysis is shown in Fig. 11. It consists of a rigid girder of mass, M, weightless columns with a total lateral stiffness, k (load per unit deflection at top), and a viscous damping mechanism with coefficient c. Three non-linear idealisations for the load-displacement characteristics of the system were used as shown in Figs. 12 to 14. These were an elasto-plastic system, a degrading stiffness system typical of reinforced concrete[15], and a prestressed concrete system[9]. The load-displacement idealisation for prestressed concrete was obtained by fitting curves to experimental results. Stage 1 represents cycles before crushing in the members of the frame but after cracking has occurred, while stages 2 and 3 show the stiffness degradation that occurs after crushing in the members, first for one direction of loading and then the other. All structures were considered to have the same lateral strength for a given period of vibration as determined from the New Zealand code for seismic design loads[16] including a representative load factor.

(a) Structural Response Displacements

The clearest means of comparison of the three non-linear systems is through their displacement - time response histories. One example is shown in Fig. 15 for structures with periods of 0.6 seconds, damping ratios of 2%, and ultimate strengths determined from the lateral load requirements of Zone A Non-Public buildings. The response of an elastic system is also shown for comparison. The earthquake ground motions in this record lasted for 30 seconds. It is apparent that the prestressed concrete structure undergoes greater amplitudes of displacement than the reinforced concrete structures. In fact a comparison of the maximum displacements from a mean axis of vibration over the range of periods of interest gave an average of 40% greater displacements for the prestressed concrete structures relative to the reinforced concrete structures.

For design considerations, a convenient representation of the maximum displacements, which the structures are subjected to during the course of the earthquake, may be given by the maximum displacement ductility factors (maximum displacement / displacement at first yield for reinforced concrete or first cracking for prestressed concrete) as shown in Fig. 16. Clearly, the ductility demands decrease with increasing period when the strength for a given period is determined from N.Z.S.S. 1900 Chapter 8[16]. Note that a direct comparison between the ductility factor requirements of the three non-linear systems for a given period in Fig. 16 cannot be made because in the definition of the term the reference displacement is that at first cracking for the prestressed concrete system, whereas for the elasto-plastic and degrading stiffness systems it is the higher value of displacement at first yield.

(b) Section Curvature Requirements

The displacement ductility factor results for the prestressed concrete structure, as presented in Fig. 16 (a), need to be related to section curvature requirements in the members of the frame. For this purpose an approach was used for a portal frame which is essentially the same as that developed by Park[17] in a study of reinforced concrete frames. The analysis is necessarily approximate in that the simplifying assumption is made that plastic hinges form at all the critical sections at the same load and at sufficient sections to form a mechanism. The two possible mechanisms are a column sideway mechanism in which plastic hinges form at the top and at the base of the columns, and a beam sidesway mechanism in which plastic hinges form in the beam and the mechanism is completed by formation of plastic hinges at the base of the columns.

Table 1 presents the results of the analyses for a range of displacement ductility factors from 4 to 16, based on first cracking in the frame. For each case the ratio of the required maximum curvature at a plastic hinge section to the curvature at that section at first cracking, ϕ_{max}/ϕ_{cr}, is tabulated for both possible mechanisms. The lengths of the plastic hinges are assumed to be equal to 0.6 times the overall depth of the section, D, the distance between the plastic hinges in the beam is assumed to be 2/3 of the beam span, and the point of contraflexure in the columns for elastic deflections is assumed to occur at 0.6 times the height of the frame, H. Results for each mechanism are presented for two values of H in terms of D. The range of displacement ductility factors considered in Table 1 does not cover those found to be necessary for unusually short period structures of this type ($T<0.6$ seconds). However, the high ductility demands in such a case are common also to reinforced concrete structures and thus it appears that short, stiff portal frame structures present particular problems for seismic resistant design which warrant further research.

Having obtained the section curvature requirements of Table 1, the design procedure is completed by determining if these curvatures can in fact be sustained. Comparison of Table 1 and Figs. 7 and 8 indicates that it is possible to ensure adequate ductility of the members providing steel areas and axial loads are kept small. For example, a frame of period 0.9 seconds and a damping ratio of 2%, designed as a Zone A Non-Public Building[16] with the same load factors as for reinforced concrete, will require a displacement ductility factor of 7.3 from Fig. 16 (a). Considering a column sidesway mechanism and H=10D, Table 1 indicates a required column curvature of approximately 26 times the curvature at first cracking. Reference to Fig. 8 shows that crushing will not occur at this curvature ratio at a column load of $P/f_c'bD = 0.05$ for example, provided $p\, f_s'/f_c'$ is less than about 0.24. It is evident that this requirement could be met and hence adequate ductility for the structure to resist the earthquake without crushing of the concrete could be made available in this case. In other cases it may not be possible to avoid crushing of the members in an earthquake of this magnitude, but as has already been shown extensive further ductility can be available after crushing of the concrete has commenced and indeed a catastrophic failure of the members should not occur for any of the curvature ratios listed in Table 1.

5. PRINCIPLES AND RECOMMENDATIONS FOR SEISMIC DESIGN

The following recommendations for the seismic design of prestressed concrete structures have been published.

5.1 American P.C.I. Principles

The P.C.I.[18] (1966) presented "Principles of the Design and Construction of Earthquake Resistant Prestressed Concrete Structures". The recommendations allow for design by elastic theory using Chapter 26 A.C.I. 318-63[19] with a 25% reduction of all moments, shears, and loads in place of the normal 1/3 stress increase. Alternatively, ultimate strength theory may be used provided the service load deformations are checked. Special attention is devoted to connections.

It should be noted that ultimate strength design by the A.C.I. 318-63[19] or A.C.I. 318-71[5] codes requires the use of the same load factors under earthquake conditions for prestressed concrete as for reinforced concrete.

5.2 N.Z.P.C.I. Recommendations

N.Z.P.C.I.[20] (1966) published "Seismic Design Recommendations for Prestressed Concrete". The ultimate strength design method is recommended and minimum load factors which are greater than those of the A.C.I. codes[5,19] are suggested in recognition of the expected greater response of prestressed concrete structures. Other recommendations deal with practical details such as that all prestressing tendons should be fully grouted throughout their length, and that where structures incorporate mortar joints suitable binding or enclosure of the joint itself should be provided to prevent loss of material under seismic moment reversals.

5.3 F.I.P. Commission Report

The Commission of the Fédération Internationale de la Précontrainte on Seismic Structures[21] reported to the Congress of the F.I.P. in Prague 1970. The report acknowledges that to give complete protection against earthquakes is not economically feasible. Instead it suggests: firstly, that a structure should be designed in such a way that member deformations causing a significant loss of prestress, and hence reducing the serviceability for normal use, should be prevented in moderate earthquakes which occur occasionally (for example, once in 10 years). Secondly, collapse or serious damage should be avoided in severe earthquakes which very seldom occur (for example, once in 100 years).

The requirement for a "moderate" earthquake is satisfied provided the elastic limit of the prestressing steel is not exceeded. In these circumstances there would be no permanent set in the steel and the prestress would resume its original value after the earthquake motions ceased. The maximum deformations for the "severe" earthquake conform to the ultimate limit state specified by the F.I.P. - C.E.B. Joint Committee[22]. Thus the maximum allowable concrete compressive strain in flexure is 0.0035.

Attention is drawn to the prevention of non-ductile failure in the members. Adequate ultimate strength should be provided for both directions of loading in the joints of rigid framed structures. The use of extra lateral reinforcement to ensure adequate ductility in the members is also recommended, particularly to cope with shear within the beam-column joint under seismic load reversals. Confinement of the concrete by closely spaced spiral reinforcement or hoops in the column may be necessary under some circumstances, and the confining steel should be continued to a distance equal to the overall depth of the member (but not less than 20 inches) from the face of the joint.

It may be commented that the member curvature limits of the F.I.P.[21] for moderate and severe earthquakes are rather difficult criteria for the design engineer to implement with the current state of knowledge. A non-linear dynamic analysis would be necessary to determine the likely member curvatures in a frame under such earthquakes, in order to verify that the limits are not exceeded. Furthermore, the limits suggested appear rather severe as they will allow little reduction of response through inelastic deformations. A structure would be expected to behave virtually elastically under a "moderate" earthquake whereas the criterion for this earthquake, that of prevention of permanent set in the prestressing steel, is not critical provided care is taken in detailing as discussed in the next section. Also, large curvatures may be sustained subsequent to a surface concrete strain of 0.0035, in the member[8] if necessary under a severe earthquake.

6. CURRENT NEW ZEALAND PRACTICE

6.1 Framed Structures

In New Zealand a number of fully prestressed concrete framed buildings have been constructed, typically three storeys or less. Higher buildings have been constructed using combinations of prestressed concrete members with reinforced concrete members. Common design practice has been to balance the gravity loads which are considered to be present at the time of an earthquake (dead load plus generally one third of the live load) by prestressing uplift from draped tendons. This means that the earthquake moments are assumed to act on a frame with members in uniform compression only. The adequacy of the beam at the column face for the design earthquake moment has then been checked using the ultimate strength approach of the A.C.I. code[19], except that greater load factors for seismic loading have been adopted in some cases. The situation may well arise in a frame with a long span and consequent large prestressing force to balance gravity loads, that the moment capacity of the beams is considerably greater than that of the columns, where the latter are designed for code earthquake moments. In this case plastic hinges will form in the columns under earthquake loading. It is recognised that it is desirable to have plastic hinges forming in the beams rather than in the columns, because in this type of failure mechanism considerably greater energy absorption is possible[17]. However, this condition is not too critical for structures of three storeys or less.

6.1.1 Design Details

In order to resist seismic moment reversals, the prestress in the members at the joints in a frame is usually concentric with the tendons spread over the depth of the member. The advantage of locating some of the prestressing tendons near the top or bottom of the section for strength and ductility considerations has been shown in section 3.2.3. A further consideration is that of permanent set of the prestressing steel. Tendons located near the extremities of the section at a plastic hinge may undergo large strains under a severe earthquake with consequent permanent set and loss of prestress force when the earthquake motions cease. This should not affect the gravity load balancing capacity of a beam, with tendons concentric at the joints and draped over the span, as the permanent sets only occur at the joints. However, it must be ensured that there will not be a complete loss of prestressing force at the joint with consequent loss of shear capacity. This problem may be overcome by placing approximately one-third of the prestressing steel at the centre of the beam in an area of low flexural strain, and the remainder may be placed near the top and bottom. Even after exceptional curvatures the central tendon would retain its prestressing force and maintain the shear capacity at the joint.

Other common frame design details are the grouting of post-tensioned tendons and in some cases post-tensioned cables are anchored in external anchor blocks, thus relieving congestion in the column in the critical joint region.

6.2 Shear Walls

Vertical post-tensioning has been used in a number of shear walls, chimneys, and towers. Such construction can have an advantage over non-prestressed reinforcement because, for example, long tendons can obviate the need for splices at points of high tensile stress under seismic loading and avoid congestion of steel.

6.3 Attitude of M.O.W.

The current attitude of the Ministry of Works with respect to the use of prestressed concrete should be noted, not only because it involves the use of the material in Government buildings, but also because it affects the approach of local approving authorities. Prestressed concrete is acceptable in primary structural members resisting gravity loads only, and in seismic load resisting shear walls provided sufficient normal reinforcing steel is included to aid ductility. However, in primary seismic resistant frames the use of prestressed concrete alone is generally considered to be unacceptable in the present state of knowledge, but prestressing could be used to assist in providing overall structural integrity, with non-tensioned steel providing energy dissipation and ductility.

7. LOAD FACTORS FOR DESIGN

The results presented in this paper[11,9] suggest that a prestressed concrete frame when responding to a major earthquake will generally have a maximum displacement of about 1.4 times that of a reinforced concrete frame with the same strength, initial stiffness and percentage critical viscous damping. In practice a prestressed concrete structure is likely to have a lower percentage critical viscous damping than its reinforced concrete counterpart, a factor which will tend to increase the above displacement ratio. However, this may be counteracted by the greater flexibility of the prestressed concrete frame (due to smaller member sizes), which results in reduced ductility demands even if the design strength decreases with increasing period of vibration in accordance with code requirements.

It has been shown in this paper that in spite of these larger displacements prestressed concrete portal frame structures, designed using the same load factors currently in use for reinforced concrete structures, can be made sufficiently ductile providing tensile steel areas and axial load levels are kept reasonably small. This result is supported by research on multistorey structures[11]. Because of the greater displacements of a prestressed concrete structure responding to severe earthquake motions, some concern may be felt about non-structural damage and it may be desirable to increase the load factor for seismic loading to make the maximum displacements similar to those of a comparable reinforced concrete structure. The recommendations of the N.Z.P.C.I.[20] which effectively increases the earthquake load part of the ultimate strength design load by 20% in the case of prestressed concrete could be adopted. Whereas an increase in load factor of about 30% would be necessary to give equal maximum displacements, this may be unnecessarily severe on prestressed concrete because non-structural damage is often not of major significance. However, rather than increase the load factor in cases where non-structural damage is important a more logical approach would seem to be to limit the allowable drift (lateral deflection) under the code seismic loading to say 70% of that of a reinforced concrete frame. This would result in roughly similar deflections under a major earthquake.

8. FURTHER RESEARCH

The most urgent area requiring further research is analysis of the non-linear response of a range of prestressed concrete structures to earthquake ground motions. A range of earthquake records, including synthetic records, should be used to ensure that the worst effects are foreseen. This would enable further assessments of displacements and ductility demands to be made.

9. CONCLUSIONS

It is considered that earthquake resistant prestressed concrete frames can be safely designed providing care is taken in detailing to ensure adequate ductility in the members. To compensate for the expected greater seismic response and therefore greater non-structural damage for a prestressed concrete structure relative to a comparable reinforced concrete structure, it is recommended that either the earthquake load part of the ultimate design loads be increased by 20% in the case of prestressed concrete or the allowable drift under the code seismic loading be limited to 70% of that of a reinforced concrete frame.

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REFERENCES

1. Nakano, K. Experiment on Behaviour Under Lateral Force of Prestressed Concrete Portal Frames - Occasional Report No. 21, Building Research Institute, Ministry of Construction, Japan, Sept. 1964, 15 pp.
2. Sutherland, W. M. Prestressed Concrete Earthquake Resistant Structures - Development, Performance, and Current Research - Proceedings of the Third World Conference on Earthquake Engineering, New Zealand, Jan. 1965, Vol. III, pp. IV - 463 to 494.
3. Lin, T. Y., Prestressed and Precast Concrete Structures, Chapter 19, "Earthquake Engineering" - co-ordinating editor Robert L. Wiegel, Prentice Hall Inc., 1970.
4. Blakeley, R.W.G. and Park, R. Seismic Resistance of Prestressed Concrete Beam-Column Assemblies - Journal of the A.C.I., Vol. 68, No. 9 Sept. 1971, pp. 677-692.
5. A.C.I. Committee 318. Building Code Requirements for Reinforced Concrete (A.C.I. 318-71) - American Concrete Institute, 1971, 78 pp.
6. S.E.A.O.C. Recommended Lateral Force Requirements - Seismology Committee, Structural Engineers Association of California, 1968, 100 pp.
7. Hanson, N. W. and Conner, H. W. Seismic Resistance of Reinforced Concrete Beam-Column Joints - Journal of the Structural Division, A.S.C.E., Vol. 93, No. ST5, Oct. 1967, pp, 533-560.
8. Blakeley, R.W.G. and Park, R. Ductility of Prestressed Concrete Members - Bulletin of the N.Z. Society for Earthquake Engineering, Vol. 4, No. 1, March 1971, pp. 145-170.
9. Blakeley, R.W.G. Ductility of Prestressed Concrete Frames Under Seismic Loading - Ph.D. Thesis, University of Canterbury, 230 pp.
10. Spencer, R.A. The Damping and Flexural Properties of Prestressed Concrete Members subjected to Reversed Cyclic Loading - Ph.D. Thesis, University of Auckland, 1966.
11. Spencer, R.A. The Non-linear Response of a Multistorey Prestressed Concrete Structure to Earthquake Excitation - Proceedings of the Fourth World Conference on Earthquake Engineering, Chile, Jan. 1969, Vol. II, pp. A4 - 139 to 154.
12. Inomata, S. Comparative Study on Behaviour of Prestressed and Reinforced Concrete Beams Subjected to Reversed Loading - Journal of Japan Prestressed Concrete Engineering Association, Vol. 11, No. 1, March 1969.
13. Entrican, G.C. Reinforced Concrete Frames with Prestress Behaviour under Repeated Reversed Loading Beyond the Yield - Private Communication, New Zealand, March 1969, 16 pp.
14. Hisada, T. and Nakagawa, K. Vibrations of Buildings in Japan - Part II, Proceedings of World Conference on Earthquake Engineering, California, 1956, pp. 7_{II} - 1 to 10.
15. Clough, R.W. Effect of Stiffness Degradation on Earthquake Ductility Requirements - University of California Structural Engineering Laboratory, Report No. 66-16, 66 pp.
16. N.Z.S.S. 1900 Chapter 8. Basic Design Loads - N.Z. Standard Model Building Bylaw, 1965, 39 pp.
17. Park, R. Ductility of Reinforced Concrete Frames Under Seismic Loading - New Zealand Engineering, Vol. 23, No. 11, Nov. 1968, pp. 427-435.
18. P.C.I. Seismic Committee. Principles of the Design and Construction of Earthquake Resistant Prestressed Concrete Structures - Journal of the P.C.I., Vol. 11, No. 3, June 1966.
19. A.C.I. Committee 318. Building Code Requirements for Reinforced Concrete (A.C.I. 318-63) - American Concrete Institute, 1963, 144 pp.
20. N.Z.P.C.I. Seismic Design Recommendations for Prestressed Concrete - New Zealand Prestressed Concrete Institute, Sept. 1966, 9 pp.
21. F.I.P. Report of the F.I.P. Commission on Seismic Structures - Proceedings of the Sixth Congress of La Fédération Internationale de la Précontrainte, Prague, 1970, 6 pp.
22. Joint F.I.P.-C.E.B. Committee. Practical Recommendations for the Design and Construction of Prestressed Concrete Structures - Fédération Internationale de la Précontrainte, London, June 1966, 82 pp.

NOTATION

A_s = area of prestressing steel

b = width of cross section

c = viscous damping coefficient

D = overall depth of section

d = distance from extreme compression fibre to centroid of prestressing steel

f'_c = concrete cylinder compressive strength

f'_s = ultimate tensile strength of steel

f_{su} = stress in prestressing steel at maximum moment as defined by A.C.I. code[5,19].

H = height of portal frame

k = stiffness (load to produce unit deflection)

k_e = elastic stiffness

k_1, k_2 = post-cracking and post-crushing stiffness

M = bending moment or mass

M_{cr} = moment at first cracking

N = number of tendon positions

P = column axial load

p = ratio of total area of prestressing steel to area of cross section

T = period of vibration

V = lateral force

V_{cr} = lateral force at first cracking in the frame

$V_{\ell i}, V_{\ell d}$ = initial and degraded loop depth load values

V_u = maximum lateral load capacity

V_y = yield strength

x = displacement

x_{cr} = displacement at first cracking

$x_g, \ddot{x}_g$ = displacement and acceleration of ground

x_m = maximum displacement from initial zero

x_{mn}, x_{mp} = current maximum displacements from initial zero in negative and positive directions

x_u = displacement at maximum load capacity

x_y = initial yield displacement

Z = slope of falling branch of stress-strain curve for concrete

$\alpha 1, \alpha 2 \ldots$ = distance to tendon position 1, 2... from extreme compression fibre/D

ε_{su} = ultimate steel strain

λ = damping ratio

μ = displacement ductility factor as defined in Figs. 12 and 14

ϕ = curvature

ϕ_{cr} = curvature at first cracking in a section

ϕ_{crc}, ϕ_{crb} = curvature at first cracking in a column section, beam section

$\phi_{max\ c}$, $\phi_{max\ b}$ = maximum curvature sustained in a column section, beam section

$\phi_{.004}$ = curvature at a concrete strain of 0.004 in the extreme compression fibre (assumed crushing strain)

TABLE 1

SECTION CURVATURE RATIOS

Displacement Ductility Factor μ	Column Sidesway Mechanism: Column Curvature Ratio ϕ_{maxc}/ϕ_{crc}		Beam Sidesway Mechanism: Beam Curvature Ratio ϕ_{maxb}/ϕ_{crb}		Beam Sidesway Mechanism: Column Curvature Ratio ϕ_{maxc}/ϕ_{crc}	
	H = 10D	H = 16D	H = 8D	H = 14D	H = 8D	H = 14D
4	12.5	18.5	13.0	21.4	9.2	14.7
6	20.3	30.8	22.2	37.2	15.4	25.3
8	28.2	43.0	31.5	53.1	21.5	35.9
10	36.1	55.3	40.7	69.0	27.7	46.5
12	44.0	67.9	49.9	84.6	33.8	57.0
14	51.8	80.1	59.1	100.6	40.0	67.6
16	59.7	92.4	68.4	116.4	50.7	78.3

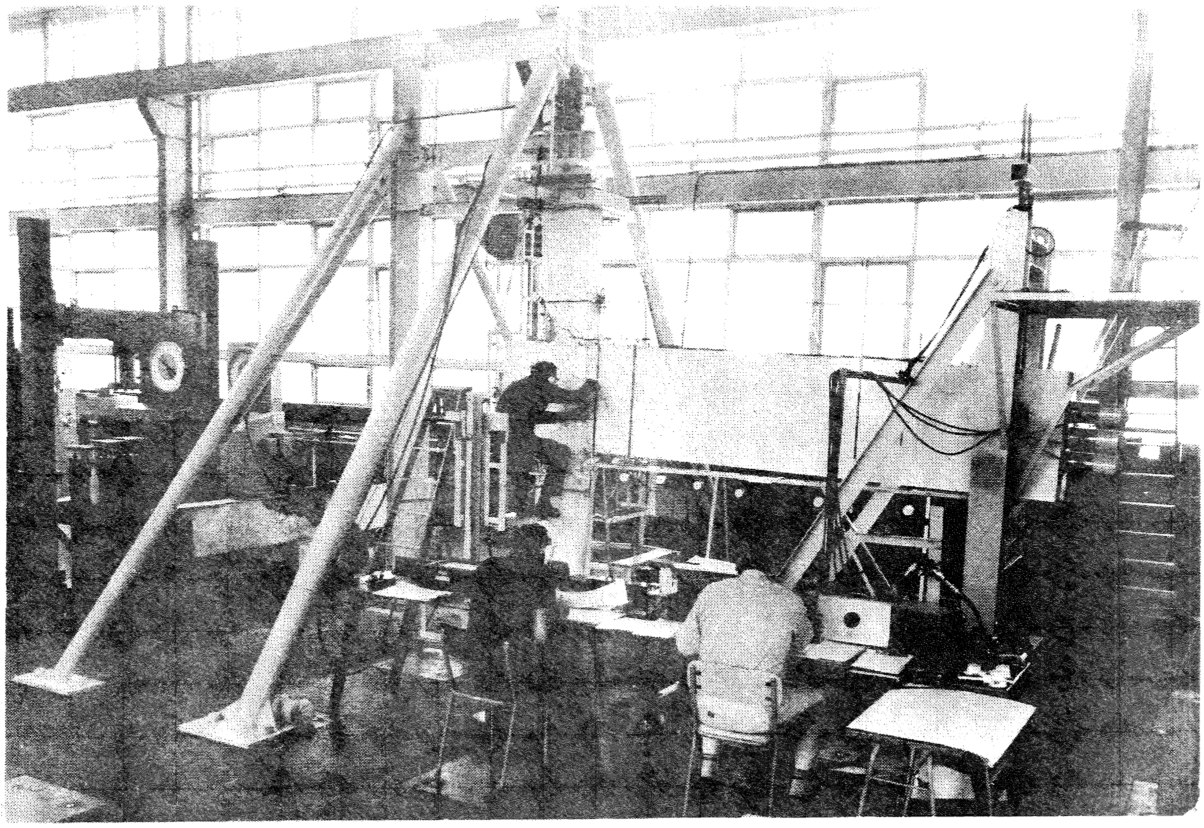


Fig.1: Unit Under Test

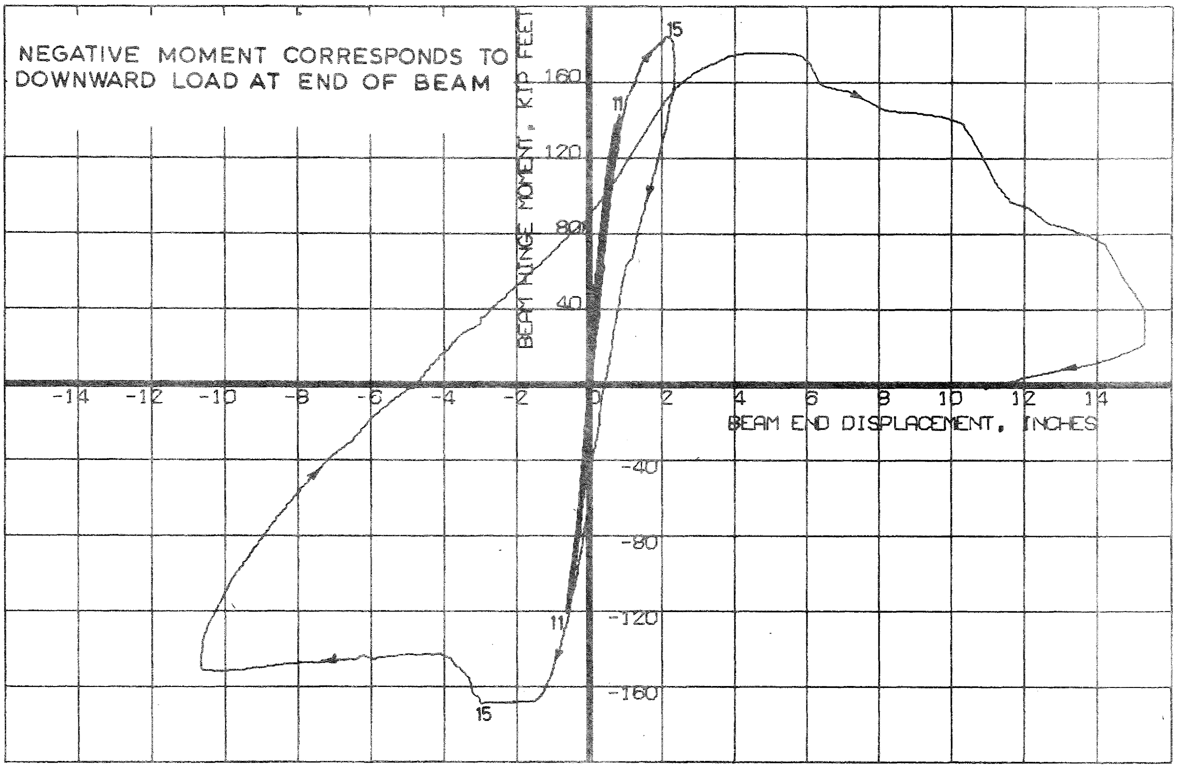


Fig 2: Experimental Moment -- Displacement Curves

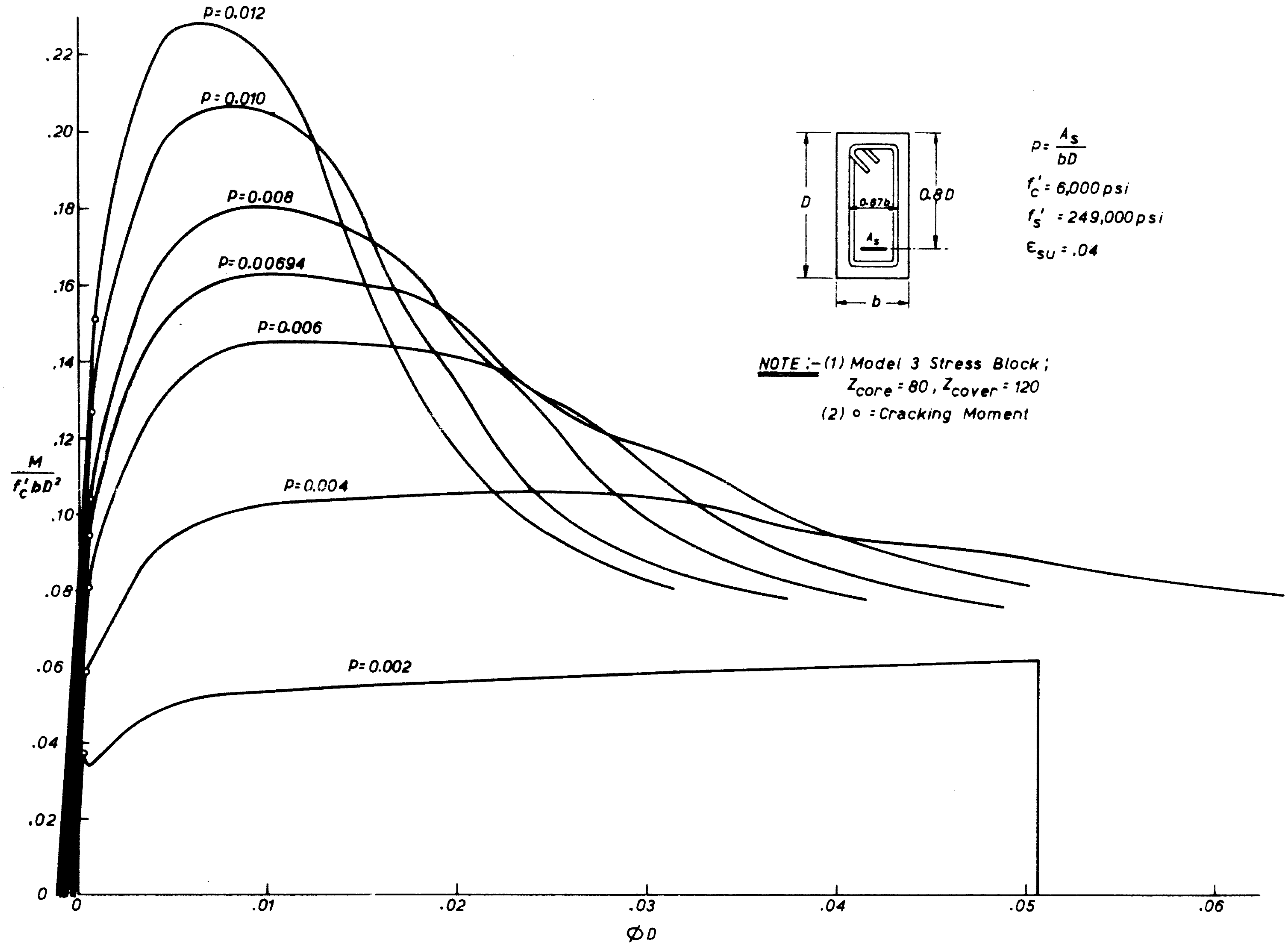


Fig 3: Effect of Variation of Steel Area Ratio

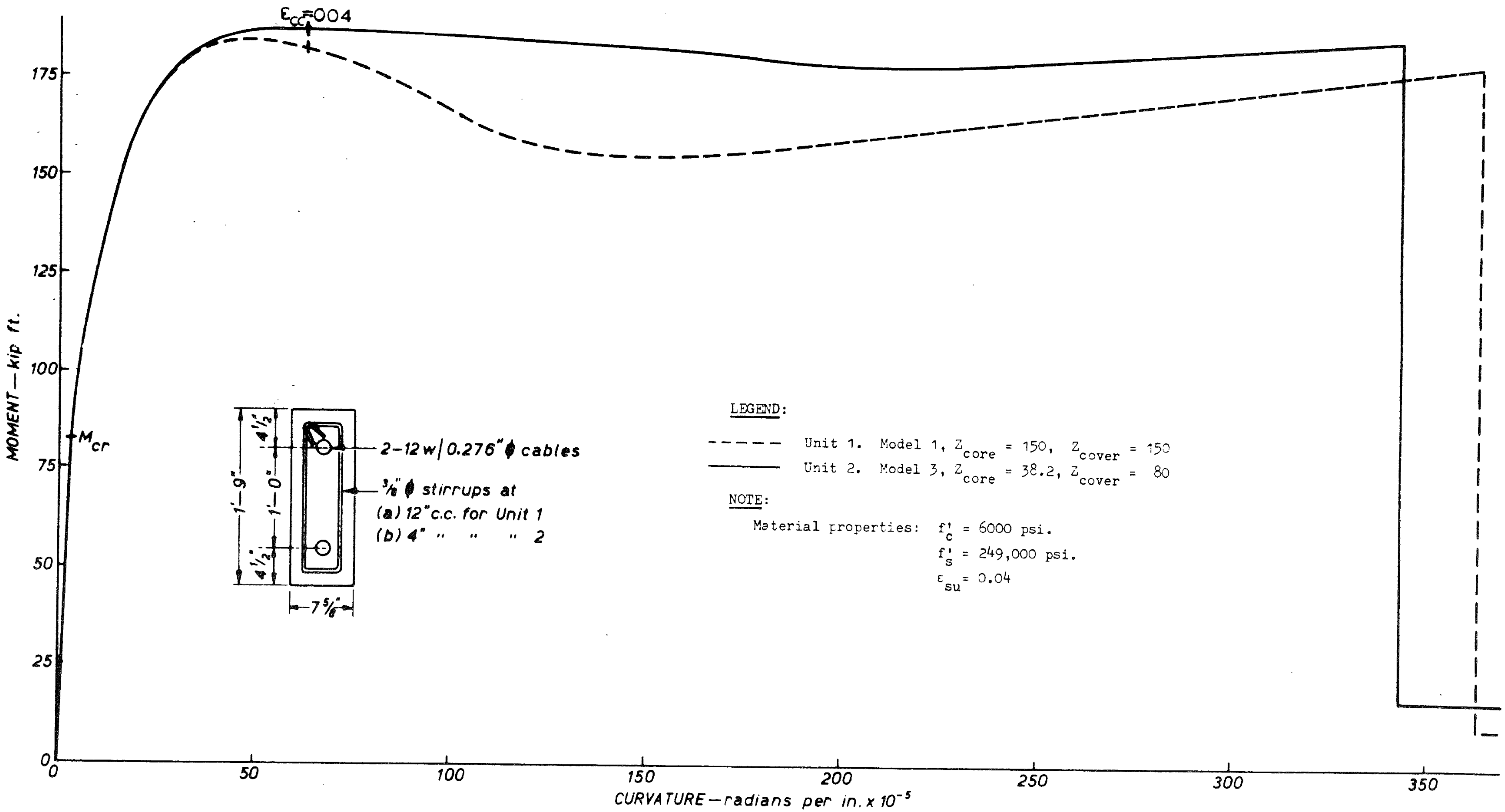
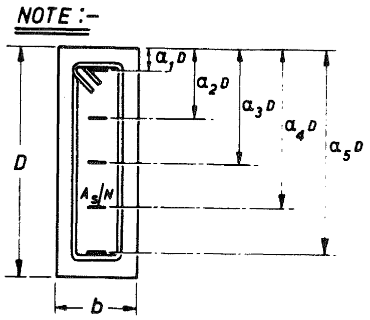


Fig 4: Effect of Different Transverse Reinforcement

NOTE :–



1. SECTION GEOMETRY

No. of tendon positions, N	1	2	3	4	5
α_1			0.1	0.1	0.1
α_2		0.2		0.367	0.3
α_3	0.5		0.5		0.5
α_4		0.8		0.633	0.7
α_5			0.9	0.9	0.9
A_s/N sq.ins.	1.435	0.718	0.478	0.359	0.287

p = 0.00896 for all cases.

2. MATERIAL PROPERTIES: f'_c = 6000 psi.
f'_s = 249,000 psi.
ε_{su} = 0.04

3. Model 3 stress block; Z_{core} = 38.2, Z_{cover} = 80.0

4. Analysis was based on $7\frac{5}{8}$" x 21" section with $1\frac{1}{2}$" cover to the stirrups.

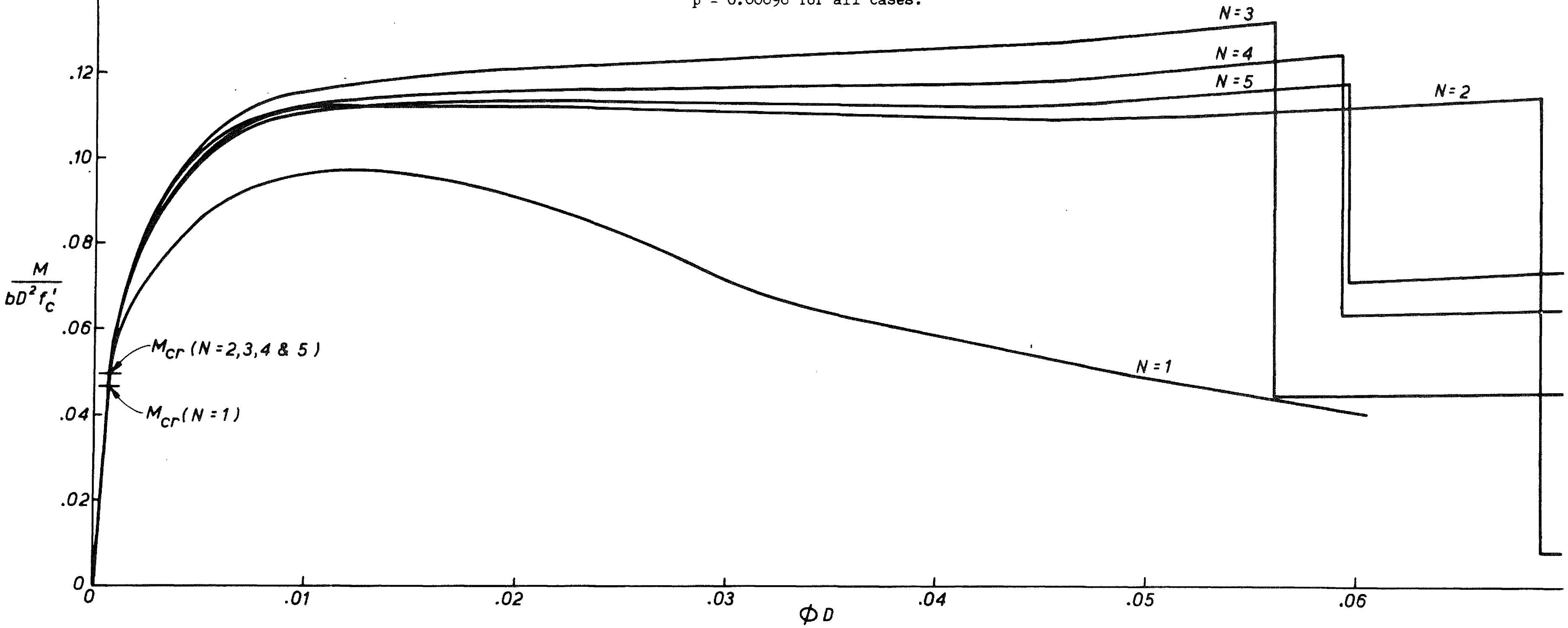


Fig 5: Effect of Distribution of Prestressing Steel

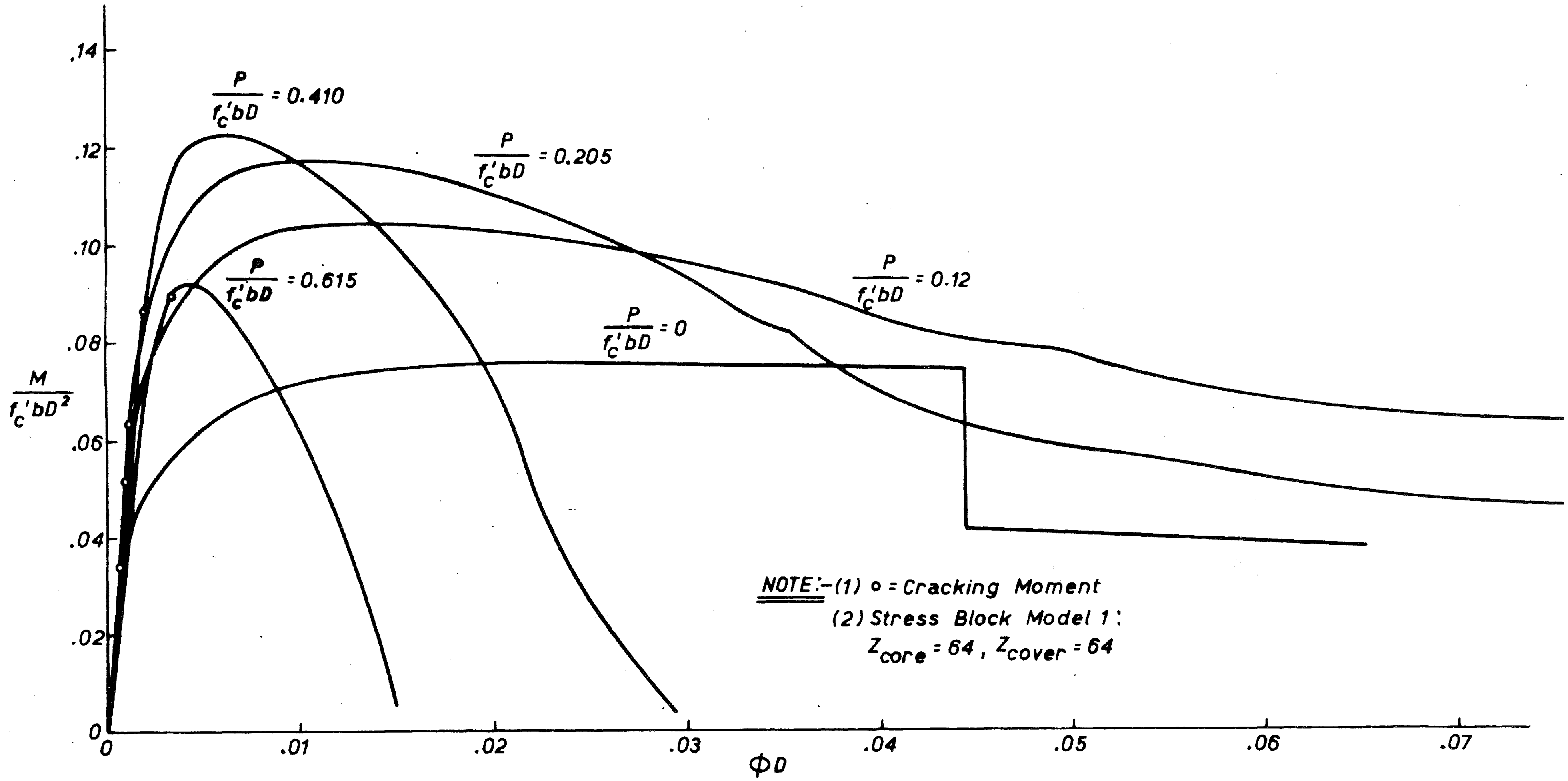


Fig 6: Effect of Variation of Axial Load

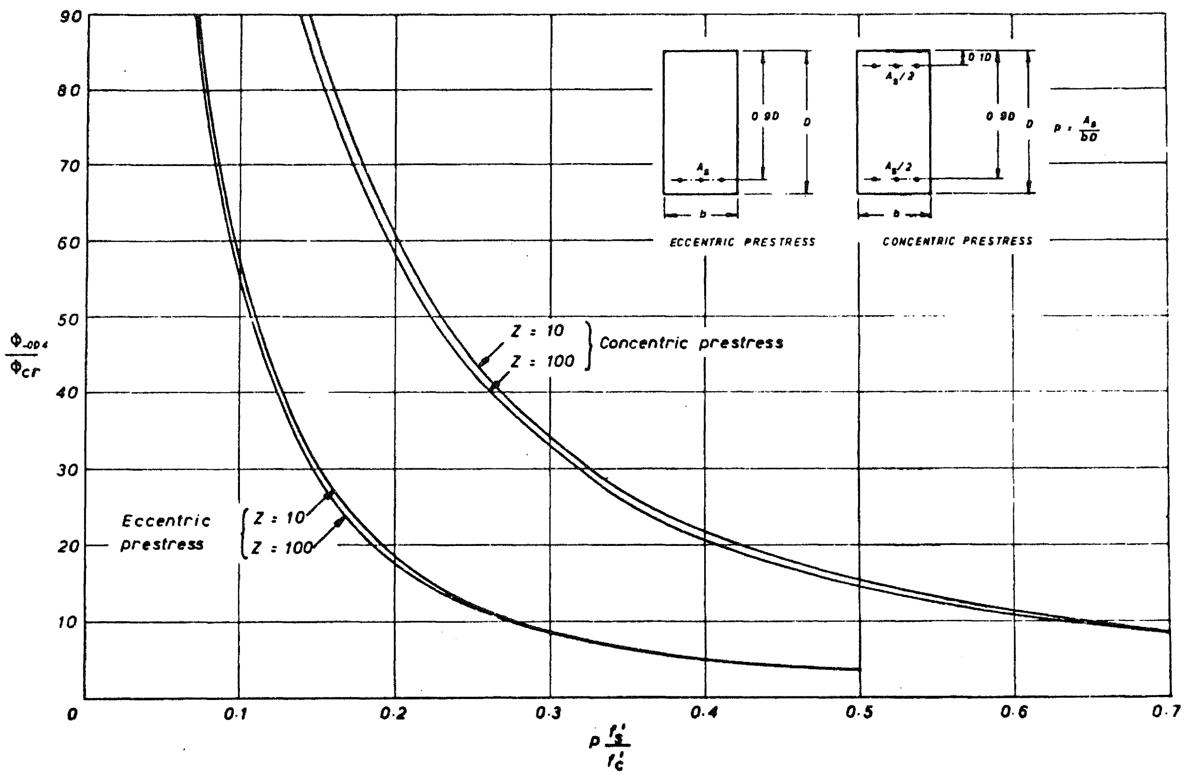


Fig 7: Variation of Curvature Ratio at Crushing for Beams

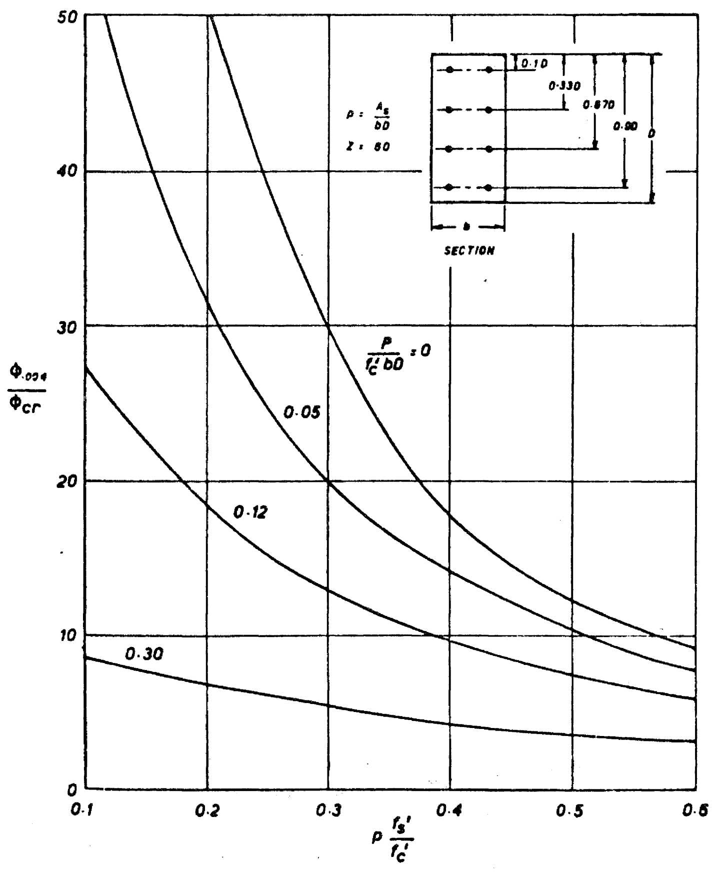
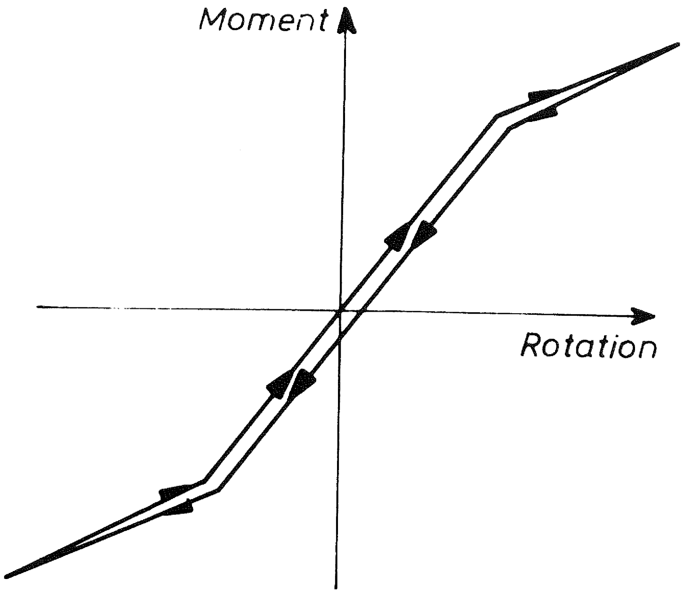
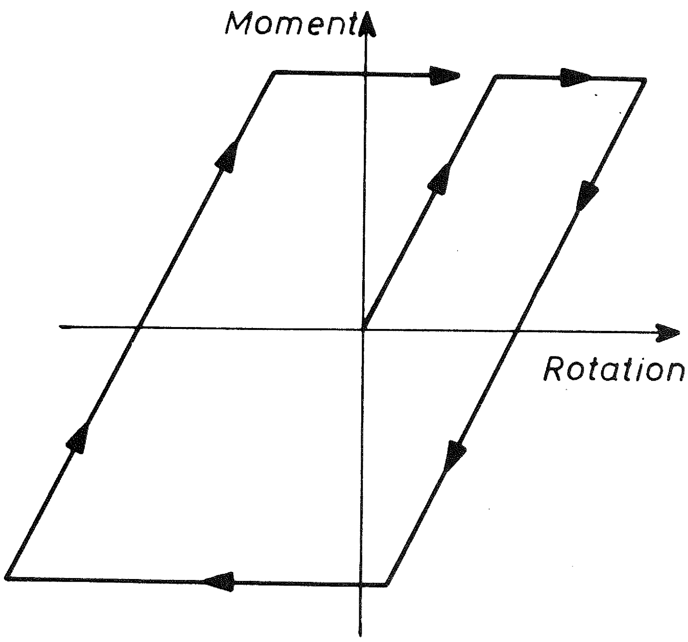


Fig 8: Variation of Curvature Ratio at Crushing for Columns

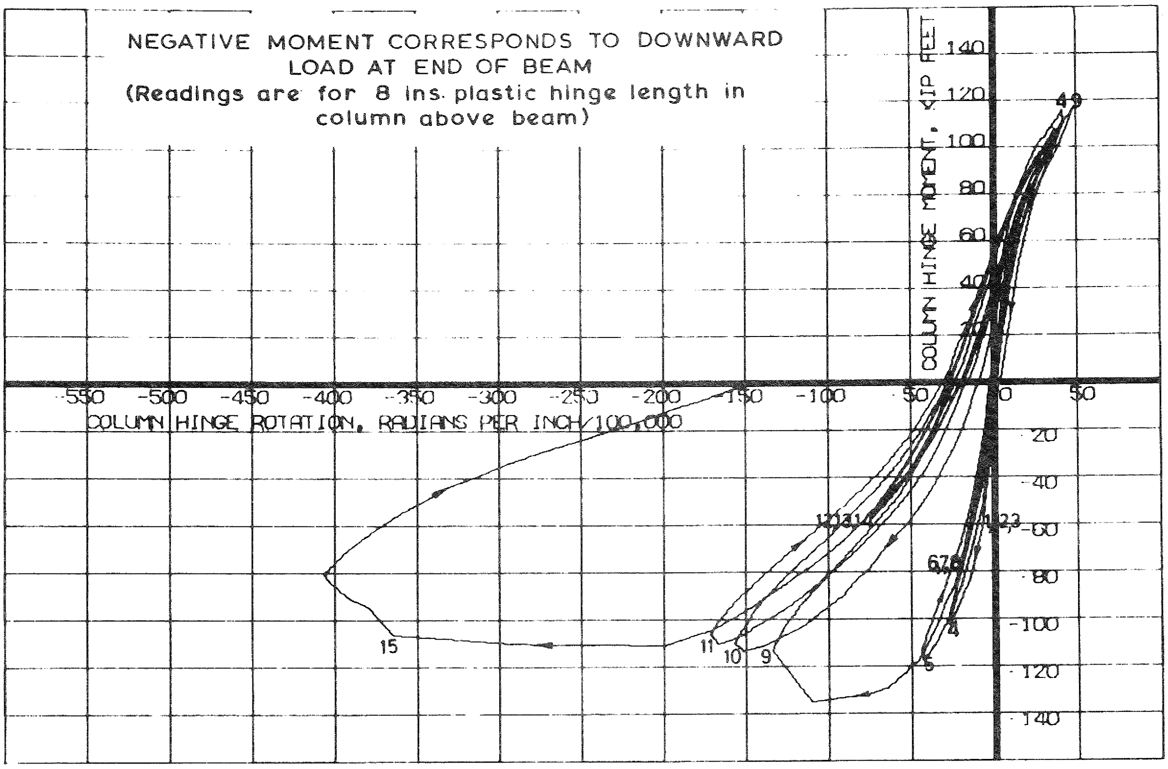
Fig 9: Structural Idealisations



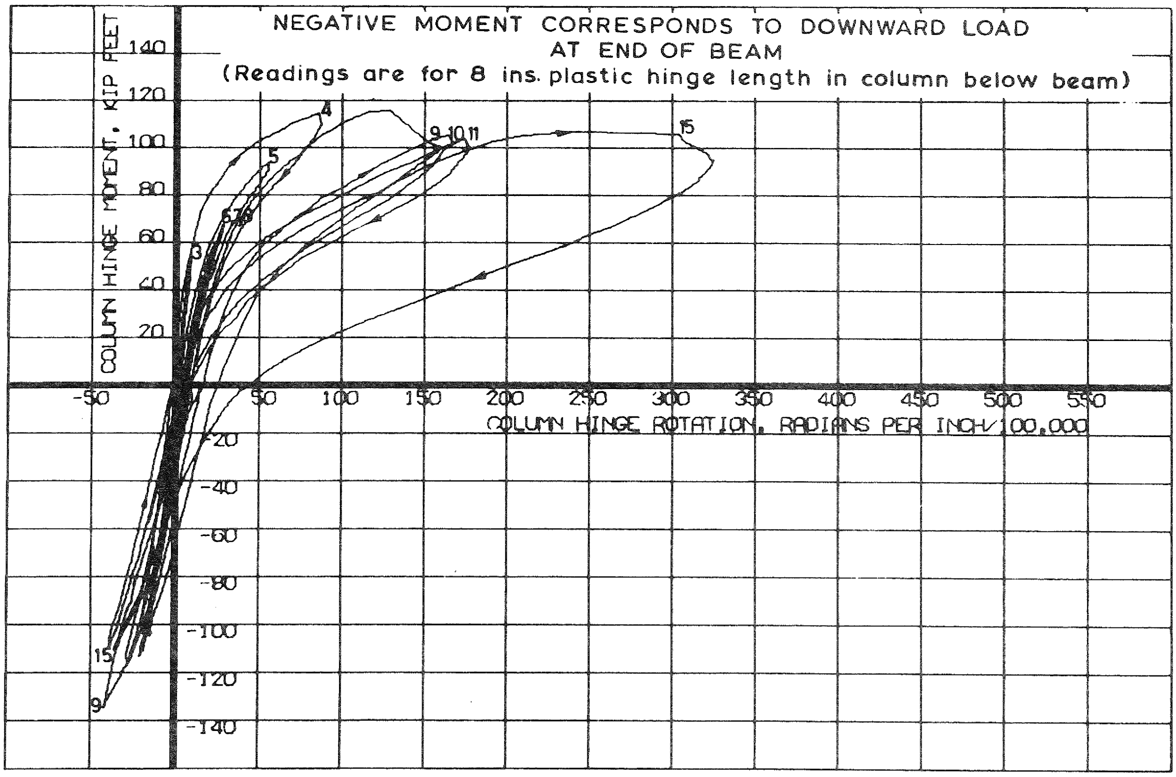
(a) Idealized Bilinear Elastic System



(b) Idealized Elasto-Plastic System



(a) Upper Column Plastic Hinge



(b) Lower Column Plastic Hinge

Fig 10: Experimental Moment-Rotation Curves

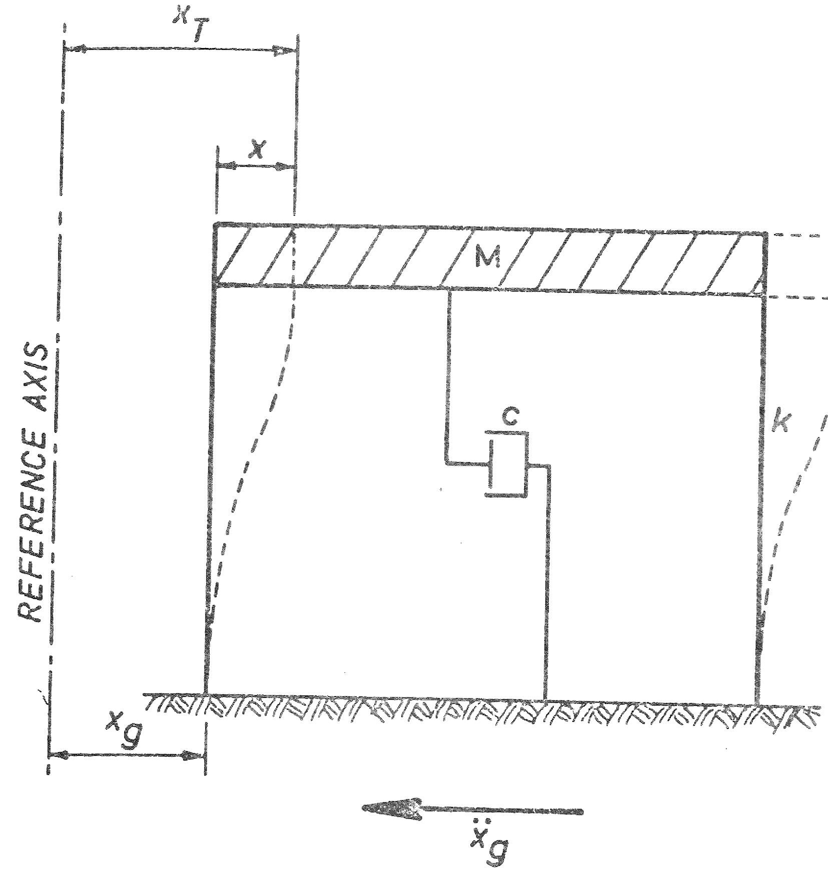


Fig 11: Structural System

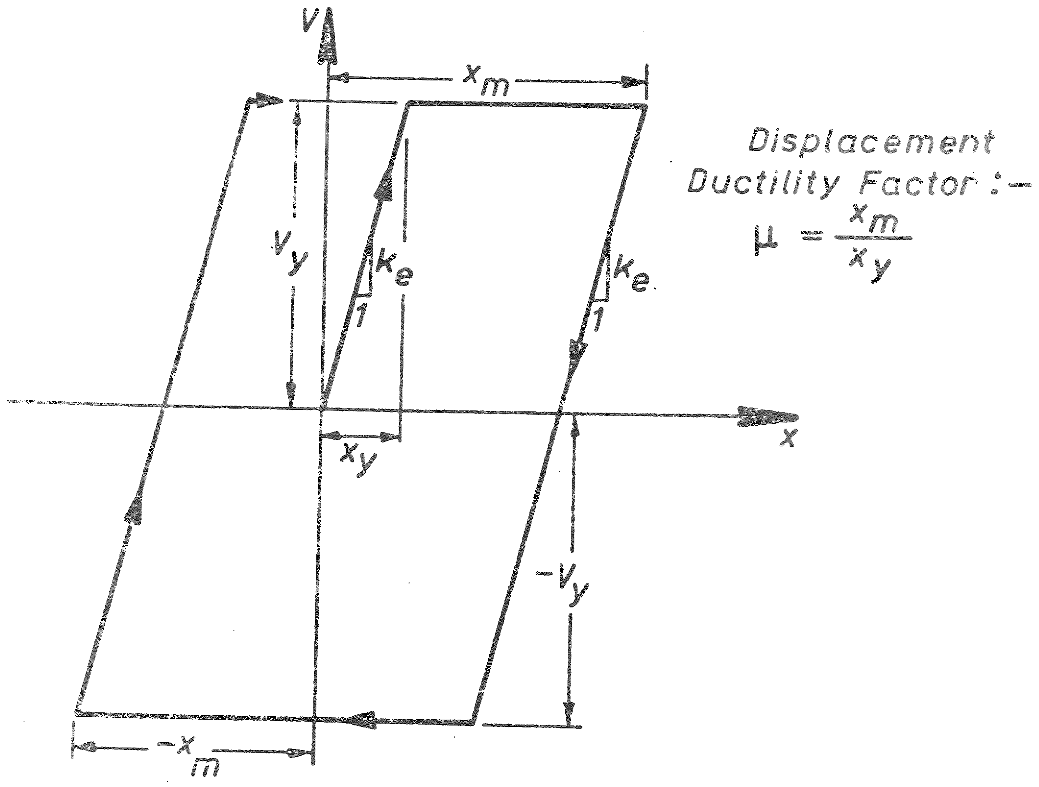


Fig 12: Elasto-Plastic Model

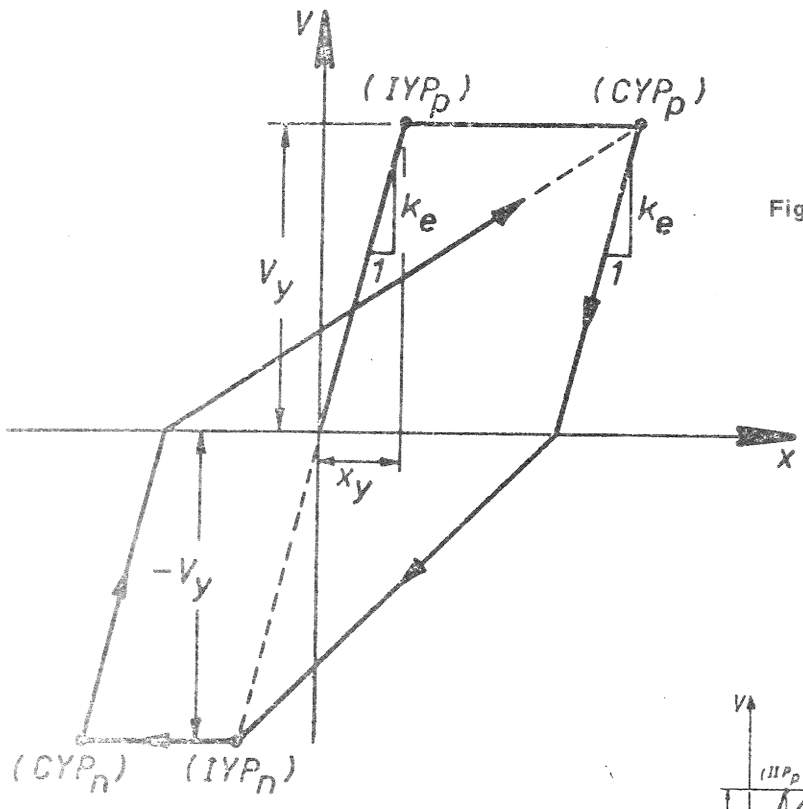


Fig 13: Clough's [15] Degrading Stiffness Model

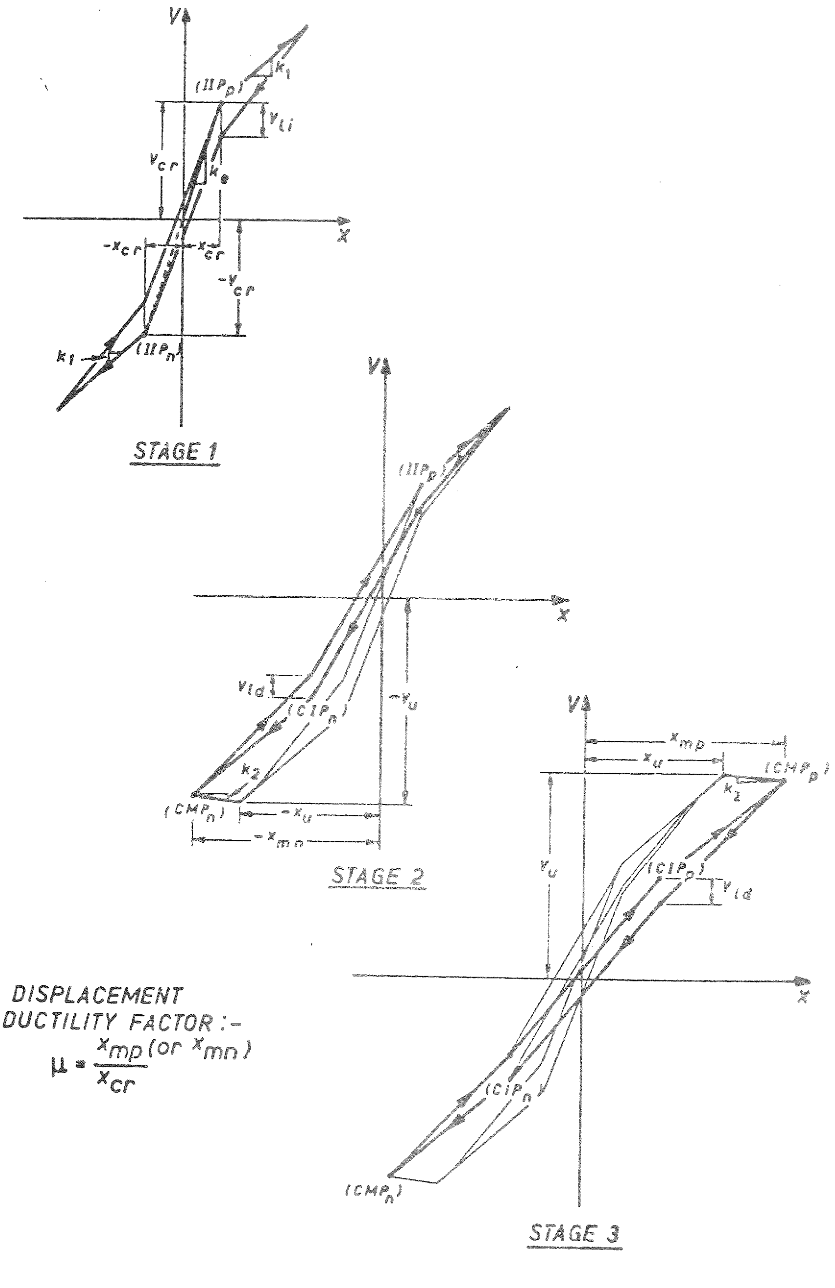


Fig 14: Prestressed Concrete Model

DISPLACEMENT DUCTILITY FACTOR :-

$$\mu = \frac{x_{mp} \text{ (or } x_{mn})}{x_{cr}}$$

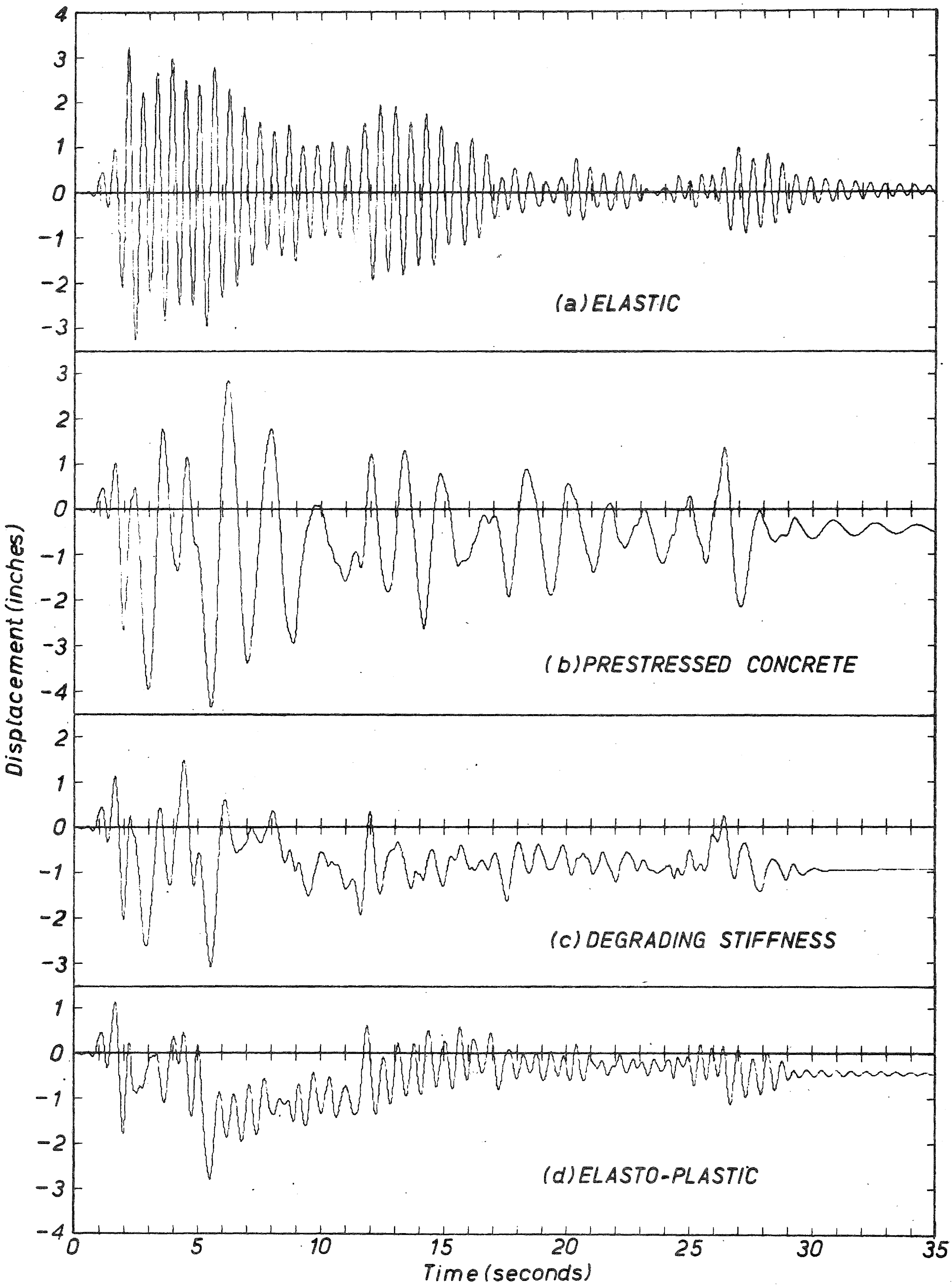
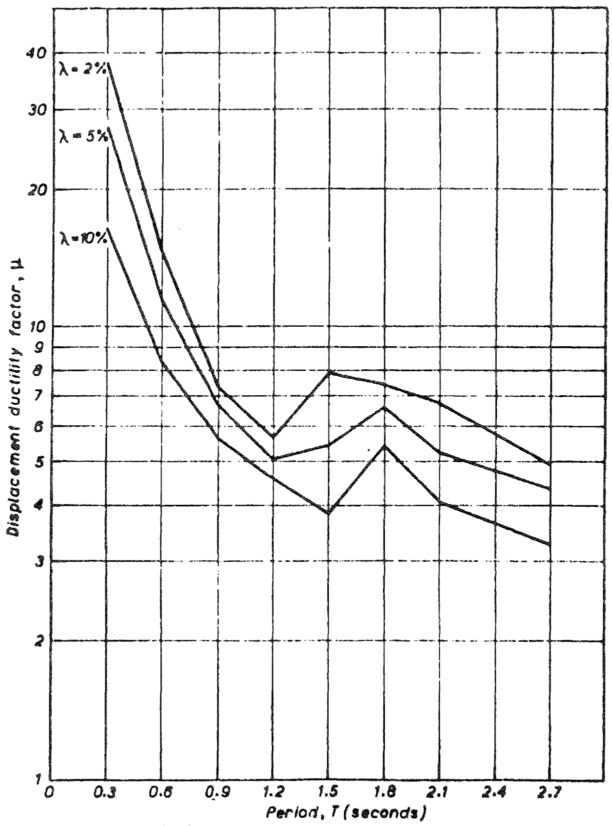
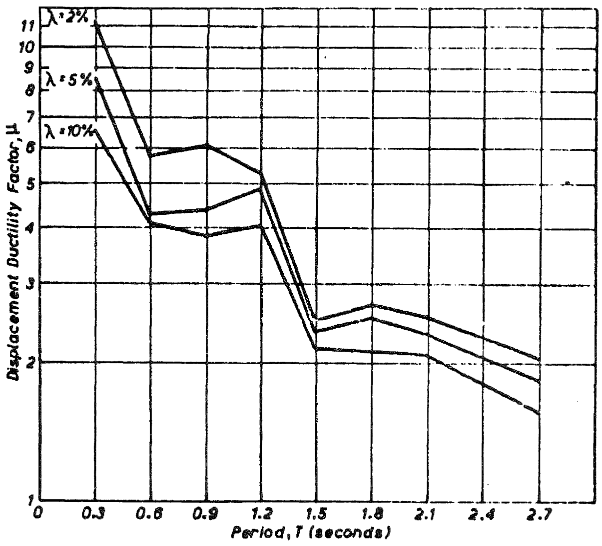


Fig 15: Earthquake Displacement Responses (T = 0.6 Seconds, λ = 2%, Zone A Non-Public Buildings, El Centro 1940 N-S Earthquake)

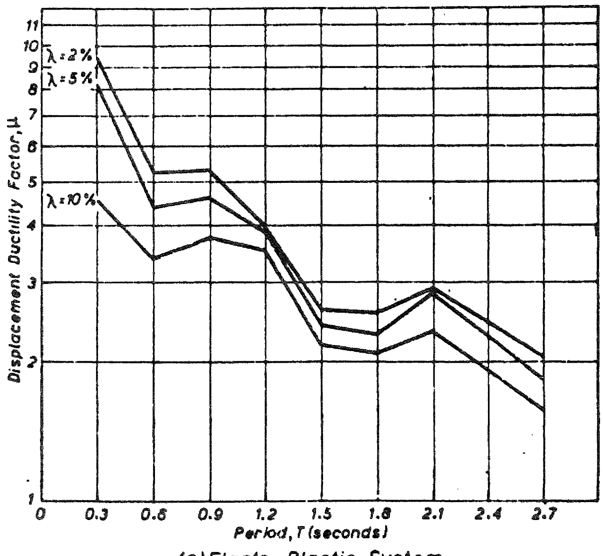
Fig 16: Maximum Displacement Ductility Factors (Zone A Non-Public Buildings, El Centro 1940 N-S Earthquake)



(a) Prestressed Concrete System



(b) Degrading Stiffness System



(c) Elasto-Plastic System