# BEHAVIOUR OF REINFORCED MASONRY SHEAR WALLS UNDER CYCLIC LOADING

J. C. Scrivener* and D. Williams**

A series of tests on reinforced brick and concrete block walls subjected to cyclic static loading applied in the plane of the walls is described. The parameters varied were the magnitude of bearing loads, wall aspect ratios and reinforcing percentages and distribution. The hysteresis loops from several cycles of load-deflection obtained by cycling at constant deformation, normally a multiple of the deformation at maximum load, were obtained and the walls loaded to failure. In particular, the ductility capability, stiffness degradation, load deterioration and the ductility requirements as determined by dynamic analyses are discussed in relation to the aseismic design of load bearing masonry walls.

## Introduction

In spite of being the oldest building material, the technological development of masonry in earthquake engineering has lagged behind other structural materials. The paucity of knowledge on the subject has led to a lack of confidence by engineers to use it for aseismic design of structures. This has not been helped by the frequent references to extreme damage which has occurred in masonry structures during earthquakes. However these masonry structures were of "unengineered" masonry and comparison with structures constructed in other materials, which were subjected to detailed seismic provisions, is grossly unfair. Even today, masonry structures are seldom designed using modern aseismic design methods.

Considerable research on many aspects of masonry structural performance has been undertaken but the behaviour under cyclic lateral loading, so necessary for consideration in aseismic design, has received very little attention. However in Mexico, Meli and Esteva (1) have conducted tests on reinforced masonry walls mainly of hollow concrete block construction. They obtained load-deflection hysteresis loops from cycling a horizontal load applied at the top of the walls. Their results showed a very marked loss of stiffness between the first and second cycles of load but a reasonably constant reduced stiffness from the second cycle onwards. The magnitude of the cyclic deformation and the amount and distribution of the reinforcing were the major parameters varied but in some tests the vertical bearing load was also altered. However all of their test walls were of aspect ratio (wall height : length) just below unity and of such magnitude of reinforcing and bearing load that shear effects dominated flexural effects.

The objects of this test series on reinforced masonry walls were as follows :

(a) to determine the ductility capabilities;
(b) to investigate the stiffness degradation and load deterioration characteristics;
(c) to observe the failure mechanisms and ultimate loads;
(d) to determine the effect on (a), (b) and (c) of varying the parameters (i) aspect ratio of walls, (ii) magnitude of bearing load, (iii) the amount and distribution of reinforcing.

Dynamic analyses were carried out to determine the effect of stiffness degradation on the ductility requirements of structural models.

Finally some recommendations for design of reinforced masonry walls are made.

## Test Walls and Testing Procedure

### Materials

In order to reduce the size and consequent difficulties in handling and testing, the walls were constructed in nominal 4" masonry units. As such the scale of the test walls was approximately one-half full size but is is contended that the results found will indicate the effect of the various parameters of the problem for prototype walls.

The masonry units used were $8^{5}$" x $4\frac{1}{4}$" x 2" bricks with 2 hollow cores each $2\frac{1}{2}$" square and 11" x 3" x 3" concrete blocks with 2 hollow cores each 3" x $1\frac{1}{4}$". In the first test series conducted, similar tests were performed on brick walls and on concrete block walls with comparable results. Later walls were all constructed in brick as the concrete block wall behaviour may be inferred directly from the brick wall results. Only test results on brick walls are reported herein.

The bricks used had compressive strengths in excess of 10,000 $lb/in^2$ and initial rates of absorption of approximately 30 gm/30 sq.in./min.

A lime mortar (5 sand : 1 cement : 0.5 lime by weight) of nominal ultimate cylinder strength 2000 $lb/in^2$, an initial flow of approximately 110% and a retentivity of approximately 70% was used in each wall.

All cores (except the unreinforced cores of BW4) were gravity filled with a very fluid grout (3 sand : 1 cement : 0.04 Onoda by weight) of nominal ultimate cylinder strength

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2000 lb/in$^2$. Considerable care was taken by the bricklayer to ensure that the cores were free of obstructions to grout flow.

Wall and Test Details

Table 1 gives the wall dimensions, amount and distribution of reinforcing and magnitude of bearing stresses. Only representative walls are discussed in this paper. The full series will be reported later(2).

It should be noted that the reinforcing percentages used are very low by reinforced concrete standards. The bearing loads are calculated on the gross sectional area and may be compared with the N.Z. Building Code allowable value of 250 lb/in$^2$ and with an estimated ultimate bearing capacity in excess of 2000 lb/in$^2$.

In every wall, the vertical reinforcing was welded to a steel base which was rigidly clamped to the test floor. The walls were constructed in stretcher bond on this steel base which had shear connectors welded to it.

In order to simulate earthquake loading on the wall element both the in-plane lateral load and the vertical bearing load were applied through a substantial reinforced concrete load-distributing beam at the wall top. The lateral load was applied through a ball seat with an hydraulic jack reacting against an exterior reaction frame. To keep the bearing load constant throughout the test, a system of jacks was used which was connected to an hydraulic pump capable of maintaining a constant oil pressure in all jacks. These jacks reacted against high tensile steel rods pinned at their other ends to the test base so that the loading system could move with the wall under the action of the lateral load.

The horizontal load was cycled, wherever possible, at a constant deformation representing some multiple of the initial deformation at maximum load. This constant deformation could not be the same for each wall as it was desired to cycle before load deterioration would occur, at least in the earlier cycles. After several cycles of load, the walls were taken to failure.

The lateral deformation as measured by a strain gauge extensometer at wall top mid-length and the lateral load as measured by a load cell were plotted continuously by an XY plotter.

A general view of the test set-up is shown in Fig. 1(a).

**Test Wall Results**

Although flexural and shear effects were present in all the walls tested, it is convenient to define two extreme states in order to be able to describe the behaviour of other walls in relation to these states.

In flexural behaviour, the initial cracking occurs mainly in the horizontal mortar joints near the base of the wall and is produced by the vertical movements necessary in the brickwork to achieve compatibility with the yielded deformations of the steel. After yielding, the load maintains the yield level while the deformation increases until failure is precipitated by crushing, usually accompanied by diagonal cracking, at the toe of the wall.

On the other hand, shear behaviour is characterized by initial diagonal cracking resulting in reduced stiffness, virtually no constant load plateau, some tendency for the load to reduce sharply from the maximum load with increasing deformation, extensive and sudden damage to the masonry causing loss of strength and eventually wall failure caused by disintegration of masonry at the toe of the wall.

The condition at failure of walls B3 and 3, exemplifying flexural and shear behaviour respectively, are shown in Figs. 1(b) and 1(c) respectively.

Some of the walls exhibited a behaviour which was initially flexural in character, and then, because of the overall deformation required of the panel while the wall displaced at yield load, they cracked along the compression diagonal. The behaviour thereafter was shear-like with one important difference in that the panels showed a constant load plateau not unlike flexural ductile behaviour. This apparent ductility was accompanied by load deterioration when the wall was cycled at constant amplitude. However the load was partially regained, almost back to the original yield load, provided the necessary deformation was reached. The degradation of load is presumably caused by progressive deterioration of the masonry along the shear cracks but increased deformation brings into effect undamaged material which allows an increase in load carrying capacity. Wall 1 (Fig. 2) is an excellent example of this behaviour which will be termed "transitional type".

Stiffness Degradation

All of the load-deflection curves (Figs. 2 and 3) show stiffness degradation with cycling. The major loss of stiffness occurs between the first and second cycles.

In the case of wall B3 (Fig. 3) behaving flexurally, the stiffness degradation was not pronounced and subsequent cycling at the same maximum deformation produced fairly constant hysteresis loops. For this wall, three cycles at a maximum deformation of 0.35" showed stable behaviour so the cyclic amplitude was increased to 0.65" where again the loops were stable justifying further increase in the deformation amplitude.

On the other hand, in the walls which failed predominantly in shear, illustrated by walls 3 and D1 (Fig. 2) and walls B2 and B4 (Fig. 3), the initial stiffness degradation was large and load deterioration and further stiffness degradation occurred on each subsequent cycle.

Effect of Bearing Load

The results of walls B3, B1 and B2 (Fig. 3) which have bearing stresses 125, 250 and 500 lb/in$^2$ respectively, show flexural, transitional and shear type behaviour respectively. Although there is a decrease in ductility as the bearing load increases, the ultimate strengths increase. Fig. 5 illustrates an idealization of this phenomenon.

It might be expected that increased bearing load would increase shear strength by the frictional action mode.

### Effect of Reinforcing

The static racking tests to failure on reinforced brick and concrete block walls reported by Schneider[3] and Scrivener[4,5] give, for any one masonry material, a reasonably constant shear strength provided a nominal amount of reinforcing is incorporated. Hence an increase of vertical reinforcing will increase the horizontal load to cause yielding of the steel (i.e. will raise the flexural strength of the wall) without altering the shear strength appreciably. Of course, the most effective vertical steel for flexural resistance is that positioned on the wall periphery. However, concentration of reinforcing at the wall edges may not necessarily produce the most suitable earthquake resistant structure, as will be discussed later. Comparison of walls B1 and B4 (Fig. 3) illustrates the above and shows that increasing the reinforcing, while other factors remain the same, has the effect of increasing the tendency towards shear failure.

Walls A1 and A2 differ only by the inclusion of horizontal reinforcing in A2. The effect of this reinforcing was to create a displacement plateau at maximum load in the first cycle. In subsequent cycles, the reinforcing was ineffective and the two walls behaved similarly. This agrees with the accepted view that shear reinforcing is ultimately ineffective in deep beams.

### Effect of Wall Geometry

In the walls of high aspect ratio, flexural behaviour occurred with wall B3 (Fig. 3) containing a low amount of reinforcing and supporting a low bearing stress (125 $lb/in^2$). A large ductility capability is evident.

Comparing walls B1 and 3 (Fig. 2), both with bearing stresses of 250 $lb/in^2$, a behaviour change from transitional to shear type occurs with reducing aspect ratio.

The behaviour of wall D1 (Fig. 2), with zero bearing load, indicates that for low aspect ratios it is virtually impossible to obtain a flexural condition. Such walls may be considered as deep beams which are known to have complex behaviour patterns.

It is important to appreciate that the ultimate strength of walls with low aspect ratio are relatively high and it is likely that, in practice, overall stability will be critical before any ductility is required.

### Ultimate Strength

In all cases of initial flexural behaviour the load to cause yielding of the wall could be predicted to within a few per cent by the algebraic summation of the moments of the bearing loads and of the yield forces in the vertical reinforcing taken about the reaction corner. This confirms the findings of Scrivener[4,5] and it shows that reinforced concrete ultimate flexural strength principles can be applied to reinforced masonry walls. It has been shown [2,8], in another part of the current research project, that the complete stress-strain curve for plain brickwork and concrete blockwork prisms closely resembles that of plain concrete specimens.

For those walls in which shear determined the maximum load, the shear strength (ultimate shear stress based on the gross horizontal section) varied from 100 $lb/in^2$ to 230 $lb/in^2$. The higher values were associated with walls containing large steel contents or supporting large bearing loads. The lowest value was obtained for the wall of aspect ratio 0.5.

## Dynamic Analyses

The important property of stiffness degradation was again brought to the attention of earthquake engineers following results of the PCA concrete frame ductility investigation[6]. For a given deformation amplitude, less energy is absorbed per cycle by a system with degrading stiffness behaviour than with an ordinary elasto-plastic system. However, until recently, practically all theoretical analyses on which the predicted earthquake ductility requirements in simple structures have been based assumed ordinary elasto-plastic behaviour. Thus the relative earthquake resistance of structures having a degrading stiffness property was questioned and, in particular, it was thought that the earthquake ductility requirements might be increased proportionately.

This was the background to a SEAOC sponsored investigation carried out by Clough[7] and aimed at determining the effect of stiffness degradation on earthquake ductility requirements. Simple single-degree-of-freedom structures with periods of vibration T ranging from 0.3 to 2.7 seconds were excited by earthquake ground motion records and their theoretical dynamic responses determined. For the flexible structures ($T > 0.6$ seconds), the ductility requirements were similar for equivalent elasto-plastic and degrading stiffness systems. However as the period approached 0.3 seconds, the ductility requirement for the degrading stiffness system became approximately twice that for the equivalent elasto-plastic system.

Load bearing masonry shear structures are typically stiffer than their reinforced concrete or steel frame equivalents, having a fundamental period of vibration of the order of 0.05N where N is the number of stories. Masonry structures of up to 6 stories will probably have fundamental periods below 0.3 seconds. Hence it was necessary to extend Clough's investigation to structures of lower periods. This extension is also applicable to the higher modes of vibration of multi-story buildings and the effect of stiffness degradation on them may be large enough to affect the overall structural behaviour.

The dynamic analyses, similar to Clough's, were undertaken on a digital computer using the numerical integration method. Only the El Centro 1940 N-S ground motion record was used for structural excitation since Clough showed that other less intense earthquakes produced the same trends. The simple single-degree-of-freedom system was used whose elastic properties are given by the period of vibration.

$$T = 2\pi \sqrt{M/k}$$

where M = vibrating mass

and $k$ = stiffness.
Damping ratios, $\lambda$, of 2, 5 and 10% were applied.

Clough's basic degrading stiffness model, as expressed in the force-deflection diagram of Fig.4a, was considered to be a reasonable, conservative first approximation to the experimental behaviour of those masonry walls which behaved in a ductile manner. Initial loading, yielding and unloading are identical to the elasto-plastic model. On further loading, subsequent stiffness is determined by two points, (i) the force-deflection condition at which the unloading terminated and (ii) the current yield point CYP, defined as the force-deflection condition of the maximum yielded displacement which has occurred at any previous time. In the event of no previous yielding in the direction concerned CYP is represented by IYP, the initial yield point which is the force-displacement condition that would be reached if the structure had yielded in this direction initially. Unloading takes place with the initial elastic stiffness.

The yield strength ratio,

$$\beta = V_y/W$$

where $V_y$ = yield strength of structure

$W$ = total weight of vibrating system,

is an important physical parameter in non-linear response analyses. Using the SEAOC code, the design base shear force $V_d$ for a structure located in the most severe zone, zone 3, is specified as

$$V_d = KCW$$

where K lies between 0.67 and 1.33 depending upon the type of framing system and seismic coefficient

$$C = \frac{0.05}{\sqrt[3]{T}}$$

Taking the yield load as twice the design load,

$$\beta = \frac{(0.1)K}{\sqrt[3]{T}} \qquad \ldots (1)$$

The highest value for $\beta$ equal to 0.3 is obtained for T = 0.1 sec. with K = 1.33. This maximum value of $\beta$ may be compared with the value based on the most severe basic seismic coefficient of NZS 1900 Chap.8. For public buildings in Zone A the coefficient is 0.16 and doubling to give a load factor of two, a yield strength ratio of 0.32 is obtained. All analyses used the value of $\beta$ = 0.3, as lower values gave ductility requirements which, as shown from the test results, would be difficult for masonry structures to attain. For all Clough's models, values of $\beta$ from equation (1) were used.

Fig. 6 shows the time-history displacement responses for the elasto-plastic and degrading stiffness systems with the structural properties indicated. Another view of the responses is shown in the force-deflection diagrams of Fig. 7. The times, after initiation of the earthquake, at which various points in the diagram were reached, are indicated.

It is evident from Figs. 6 and 7 that the degrading stiffness structure responds more actively than the ordinary elasto-plastic system. The frequency is reduced by the loss of stiffness but the total displacement from the initial position is greatly increased. This contrasts with the behaviour of flexible structures (as determined by Clough) where the initial response forthe two cases is almost identical but the loss of stiffness resulting from the large yield deformations greatly reduces subsequent response in the degrading system.

It is rewarding to consider these results with reference to the average response spectra shown in Fig. 8. With a flexible structure, (i.e. on the descending portion of the curve) as the stiffness degrades causing an increase in period, the structure becomes less responsive to earthquake excitation. Thus the build-up to the maximum response deformation is essentially an elastic phenomenon resulting ultimately in oscillations which cause yielding quickly followed by response stabilization. A further factor contributing to the desirable behaviour of flexible structures is that after any yielding has occurred the degrading stiffness mechanism gives rise to a hysteresis loop for all cycles of loading and unloading regardless of whether yielding takes place in the cycles. Of course, in the elasto-plastic system hysteretic energy losses result only from yielding during that cycle.

In the case of stiff structures ($T < 0.3$ seconds), as the period increases with stiffness degradation the structural response would be expected to increase along the ascending portion of the spectrum and may ultimately climb over the peak of the curve. Such reasoning explains the behaviour observed in Figs. 6 and 7.

Table 2 details typical results of the dynamic analyses. For the structure with 0.3 second period and properties of yield strength ratio 0.3, damping factor 10%, ductility factor (ratio of displacement at point of interest to displacement at yield) $\mu_o$ = 2.9 for the ordinary elasto-plastic system and ductility factor $\mu_d$ = 6.1 for the degrading stiffness system were required. For the equivalent structure with 0.1 second period, factors $\mu_o$ = 1.7 and $\mu_d$ = 3.8 were required. In fact, the ductility factor ratio $\mu_d/\mu_o$ just greater than 2 is typical of structures with periods 0.3 seconds and lower. This may be compared with ductility factor ratios in the range $0.8 < \mu_d/\mu_o < 1.2$ for longer period structures. An idealized average relationship for the relative ductility requirements (degrading/ ordinary elasto-plastic) of simple vibrating structures, expressed as a function of period, is given inFig. 9.

However in the short period range the actual ductility requirements, for both material types, generally decrease as the period decreases. This may be explained by the fact that the initial response for the low period elastic structure is less than for the longer period structure within the range up to 0.3 seconds. Structures with periods of vibrations near 0.3 seconds and exhibiting the stiffness degrading property will be the most susceptible to earthquake excitation, requiring ductilities of the order of 6 to withstand earthquakes of

El Centro intensity when designed with a yield strength ratio of 0.3.

Clough investigated the effect of bi-linear stiffness and showed that even slight strength degradation could be disastrous for short period structures but for flexible structures negative bi-linear characteristics had very little effect. Positive bi-linear behaviour has no deleterious effects on either stiff or flexible structures.

Observation of cyclic loading test results often indicate a stiffness degradation more severe than the basic degrading stiffness model. An hypothetical model, termed the total degrading stiffness model, was devised to represent the most extreme case of stiffness degradation. This model, illustrated in Fig. 4b, is the same as the basic degrading stiffness model for the first complete cycle of load. Thereafter the stiffness takes a zero value while the structure displaces at zero load until the next loading returns it with the original elastic stiffness to the current yield point. Under these extreme conditions, large ductilities are required (see Table 2) especially for low damping ratios as in this situation damping is the main source of energy dissipation. It must be emphasized that ductility factors from the hypothetical total degrading stiffness model are upper limits but they are valuable to indicate the necessity to produce designs in which this behaviour does not occur.

## Conclusions and Design Recommendations

Contrary to the generally held view, the experimental results indicated that it is possible to design reinforced load bearing masonry structural elements so that ductile behaviour is achieved. The tests also showed that low bearing loads, light reinforcing and high aspect ratios all enhance the prospect of flexural behaviour. Approximate boundaries beyond which such behaviour cannot be expected were found.

Theoretical dynamic response analyses (using the El Centro 1940 N-S earthquake) on stiff structures e.g. masonry walls, have shown that these are more responsive to earthquake excitation than flexible structures and that their ductility requirements are greater for a degrading system than for an elasto-plastic system. All of the masonry walls tested showed stiffness degradation, but with the least being shown by the ductile walls. The analyses showed that the greater the stiffness degradation, the larger is the ductility requirement. Only the flexural walls had sufficient ductility capability to meet the requirements. Less flexural walls exhibited some strength degradation and with stiff structures it has been shown that this can lead to collapse.

For ductile flexural walls it is appropriate and desirable to adopt the ultimate strength design philosophy as used in seismic-resistant reinforced concrete framed structures. The essence of the method is to limit reinforcing steel so that the load associated with the ultimate moment capacity does not exceed the shear strength of the structural element. Excess flexural steel may be disastrous. This is analogous to the restriction of steel content in a reinforced concrete flexural member to avoid a primary flexural compression failure. Reinforced masonry structures designed by this method should be able to withstand moderate earthquakes without major structural damage and should not collapse under the effect of the "severe" earthquake.

Of course it is assumed that a nominal amount of reinforcing is provided to give the walls structural integrity.

It must be emphasized that the wall aspect ratio must be sufficiently large to ensure enough flexural ductility before shear effects become predominant. In the test, true flexural behaviour with large ductility was not able to be achieved with walls of aspect ratio one or less. However it is conceivable that walls in this range which show apparent ductility (shear displacement at constant load) may be capable of withstanding earthquakes, when designed by the ultimate strength approach.

If ductile behaviour cannot be expected aseismic design by the working stress method is essential. For the stiff masonry wall this method, as currently used, is irrational. As no ductility can be assumed (unlike the situation in framed structures) the design loads must necessarily be the actual seismic loads likely to be experienced. These will be considerably larger than the normally used design loads. Lack of appreciation of this fact has lead to the mistaken belief that there is an inherent weakness in the masonry whereas in fact the earthquake forces which must be resisted by this type of structure have been underestimated. The same argument applies to reinforced concrete shear walls of this type. It should be realized that the load capacity of such walls in reinforced masonry (or concrete) may be very high. Hence if realistic stresses are used the above proposal may not be limiting.

The above comments have been based on the results from static cyclic tests. Similarly, dynamic behaviour of reinforced concrete structures has been assumed (conservatively) by static test results and this assumption is normally justified by the fact that both the yield and ultimate strength of reinforcing steel and the ultimate strength of concrete increase with increasing rates of strain.

This assumption is questioned for the following two reasons: (1) the strain-rate dependent properties have been obtained, by necessity, from non-cyclic tests and (2) no account has been taken of other structural properties which may be affected by dynamic conditions e.g. shear strengths. It is hoped to conduct dynamic tests on walls similar to those tested statically.

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TABLE 1 - TEST WALL DETAILS

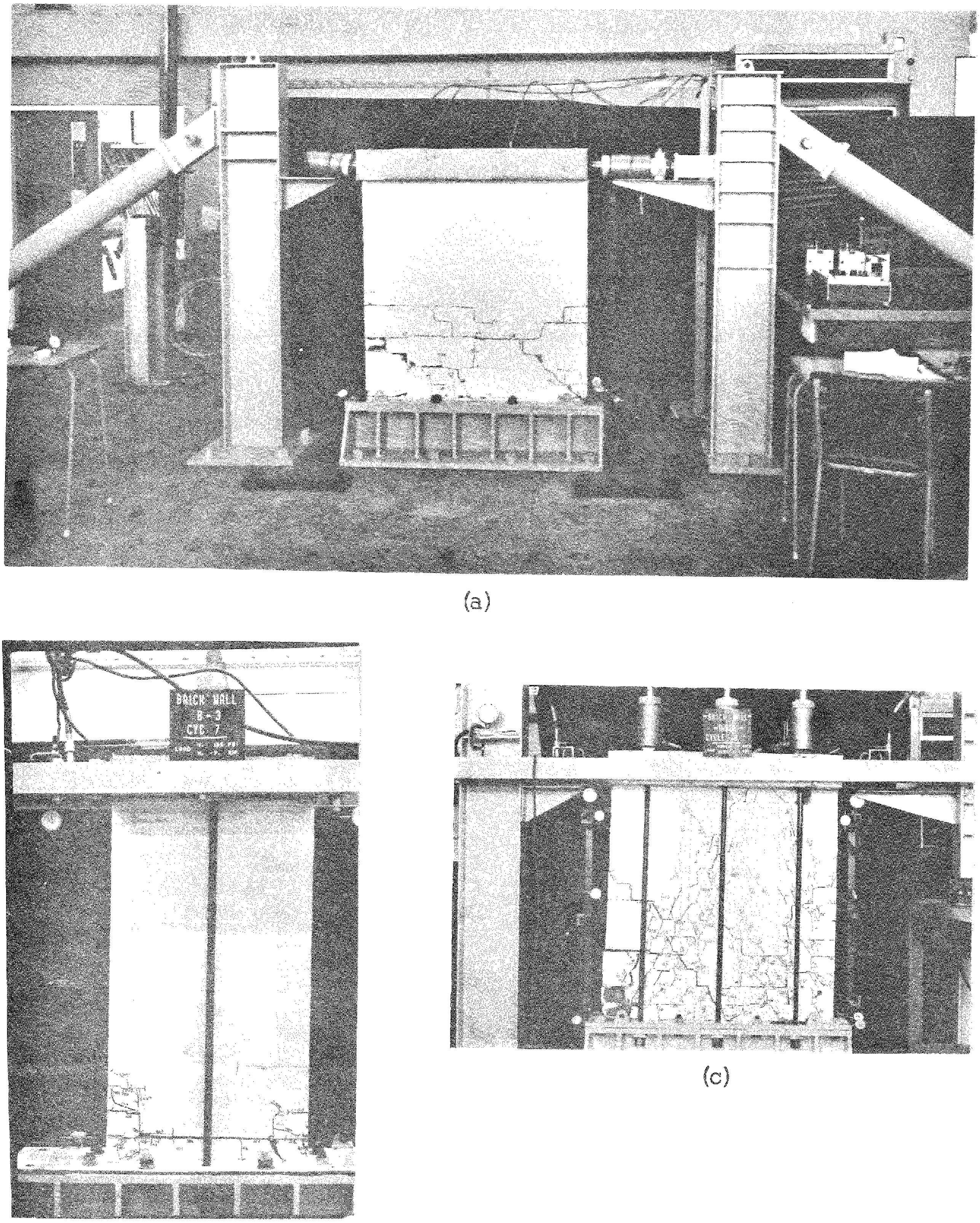
| Material | Designation | Height | Length | Nominal Aspect Ratio | Vertical Reinforcing | Reinforcing (%) | Bearing Pressure ($lb/in^2$) |
|---|---|---|---|---|---|---|---|
| Brick | 1 | 3'-9" | 3'-8" | 1 | $4/\frac{3}{8}$" bars uniformly distributed | 0.24 | 0 |
| " | 2 | " | " | 1 | " | " | 125 |
| " | 3 | " | " | 1 | " | " | 250 |
| " | 4 | " | " | 1 | " | " | 500 |
| " | 5 | " | " | 1 | " | " | 500 |
| Concrete Block | 1 | 4'-0" | 3'-11½" | 1 | $4/\frac{3}{8}$" bars uniformly distributed | 0.26 | 0 |
| " | 2 | " | " | 1 | " | " | 125 |
| " | 3 | " | " | 1 | " | " | 250 |
| " | 4 | " | " | 1 | " | " | 500 |
| Brick | A1 | 3'-9" | 3'-8" | 1 | $2/\frac{7}{8}$" bars on periphery | 0.67 | 250 |
| " | A2 | " | " | 1 | $2/\frac{7}{8}$" bars on periphery & $2/\frac{5}{8}$" bars horizontally | 0.67 & 0.33 | 250 |
| " | B1 | 3'-11" | 2'-2" | 2 | $2/\frac{3}{8}$" bars on periphery | 0.204 | 250 |
| " | B2 | " | " | 2 | " | " | 500 |
| " | B3 | " | " | 2 | " | " | 125 |
| " | B4 | " | " | 2 | $4/\frac{3}{4}$" bars on periphery | 1.63 | 250 |
| " | D1 | 3'-2" | 6'-1" | 0.5 | $6/\frac{3}{8}$" bars uniformly distributed | 0.225 | 0 |
| " | D2 | " | " | 0.5 | " | " | 250 |

Note: All reinforcing bars deformed.

## TABLE 2 - TYPICAL RESULTS OF DYNAMIC ANALYSES

| Period T (sec) | Model Type | Damping Ratio $\lambda$ (%) | Maximum Ductility Factor |
|---|---|---|---|
| 0.3 | Elasto-plastic | 10 | 2.9 |
| " | Basic degrading stiffness | " | 6.1 |
| 0.2 | " " " | " | 3.4 |
| 0.1 | Elasto-plastic | " | 1.7 |
| " | Basic degrading stiffness | " | 3.8 |
| " | " " " | 5 | 6.3 |
| " | " " " | 2 | 6.9 |
| " | Total degrading stiffness | 10 | 9.6 |
| " | " " " | 5 | 32 |
| " | " " " | 2 | 53 |

Note: $\beta$ = 0.3 for all cases.




(a)

(b)

(c)

Fig. 1.

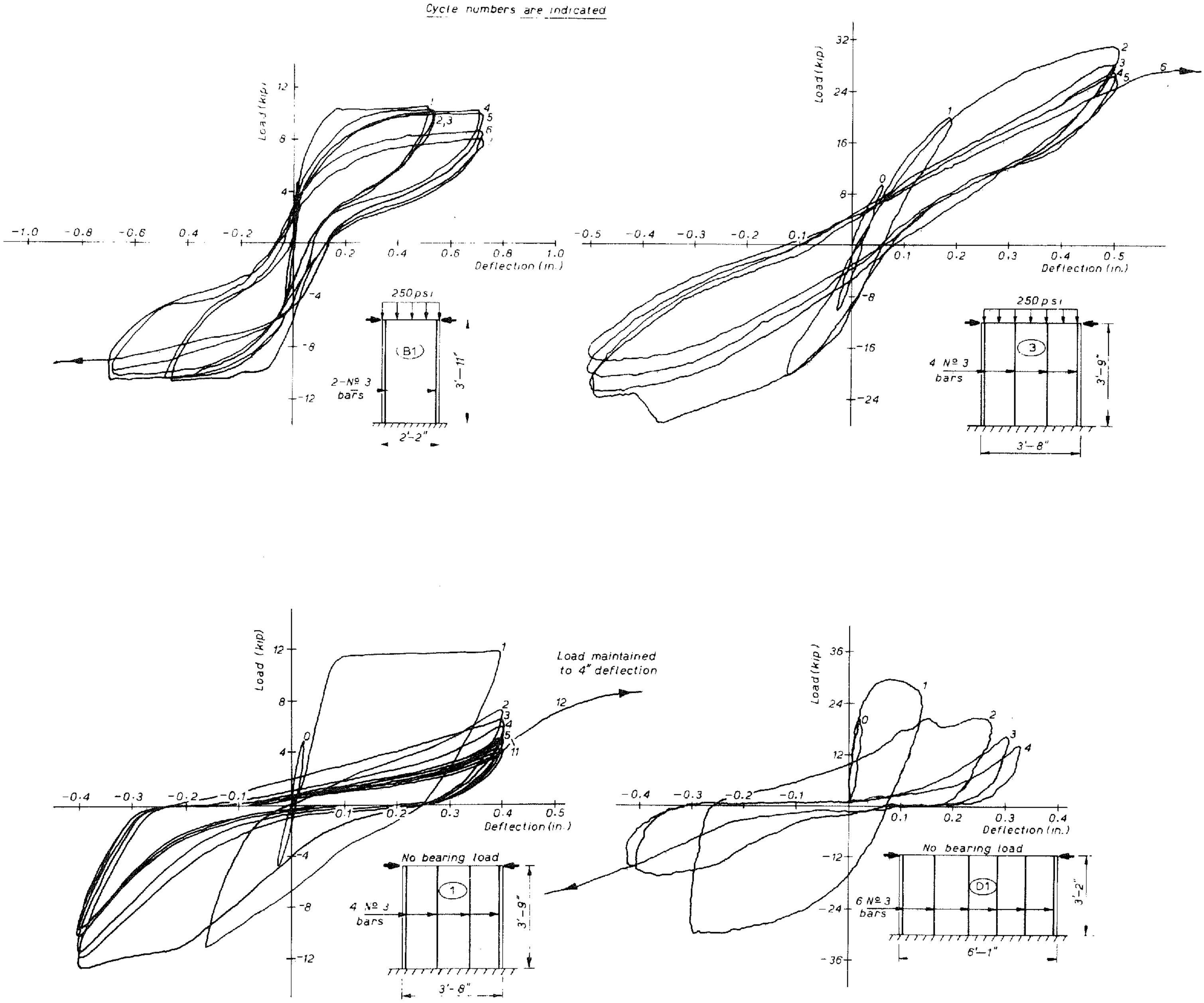



Fig.2 : LOAD DEFLECTION CYCLES

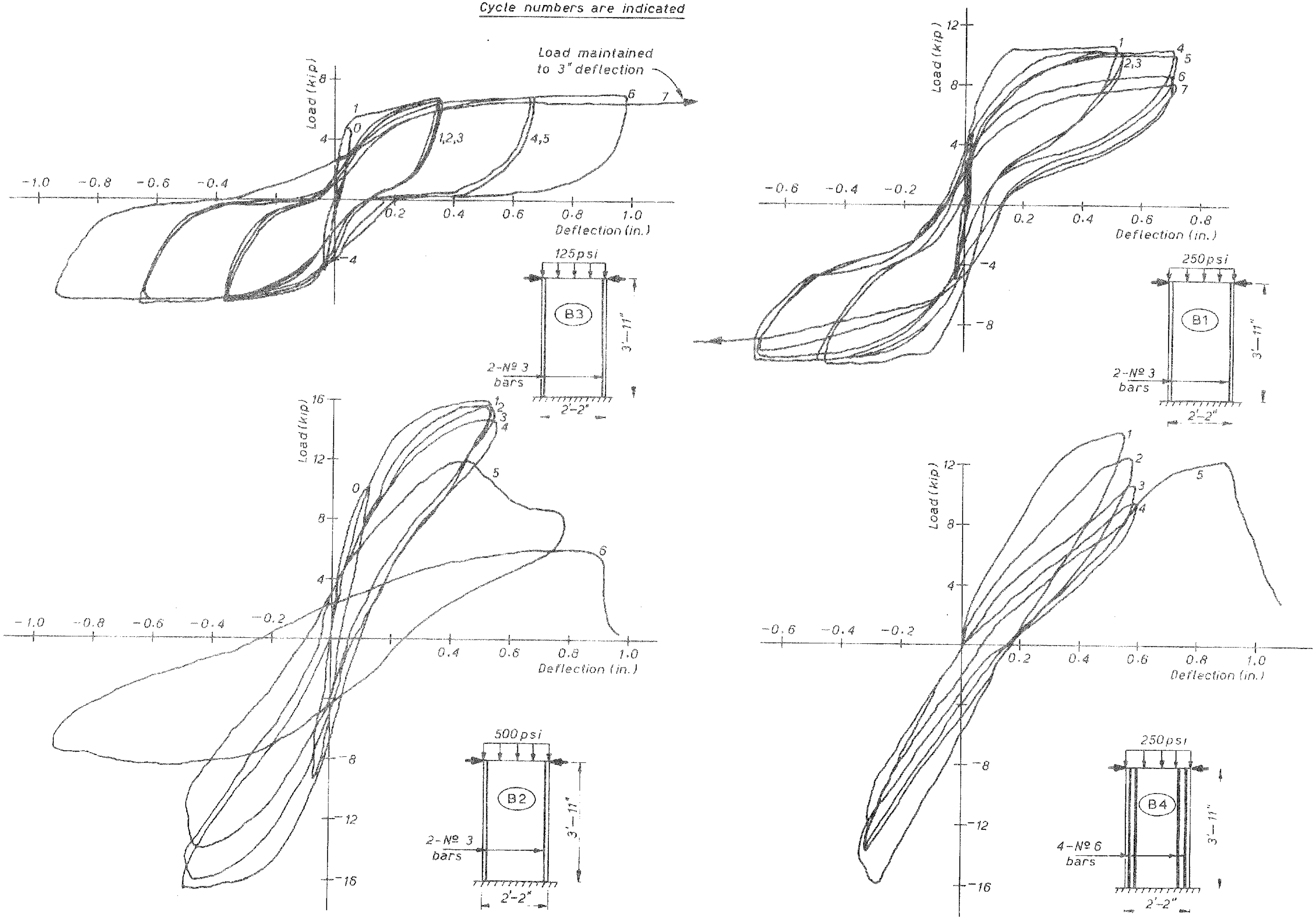



Fig. 3 : LOAD-DEFLECTION CYCLES

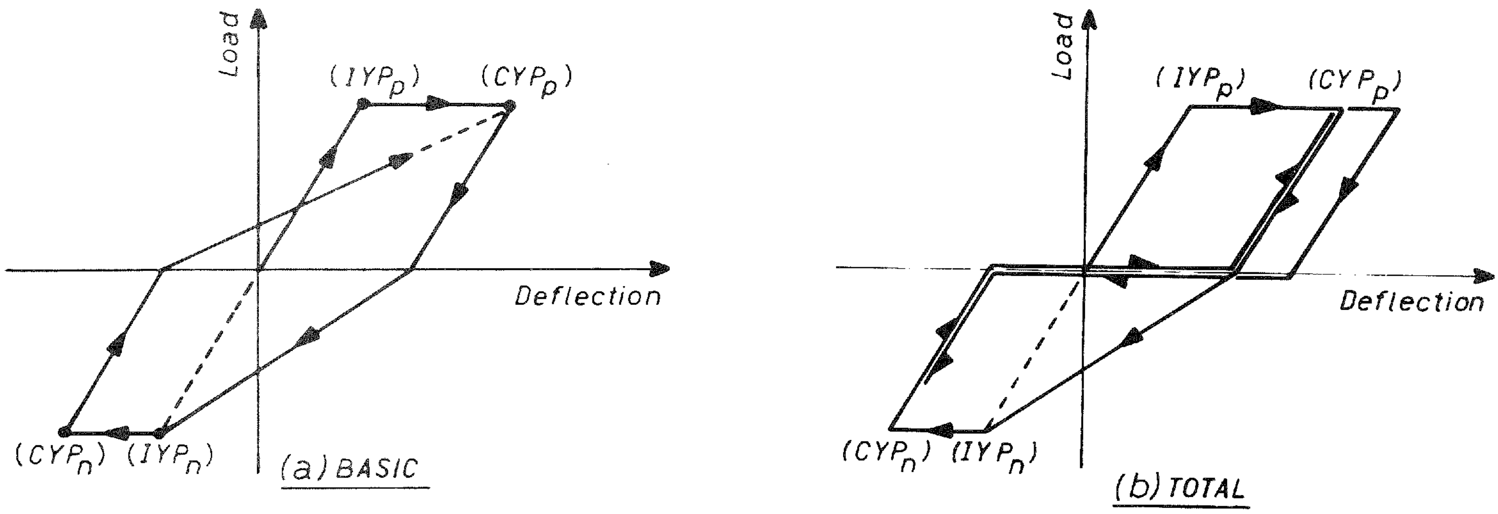



Fig. 4 : IDEALISED DEGRADING STIFFNESS MODELS

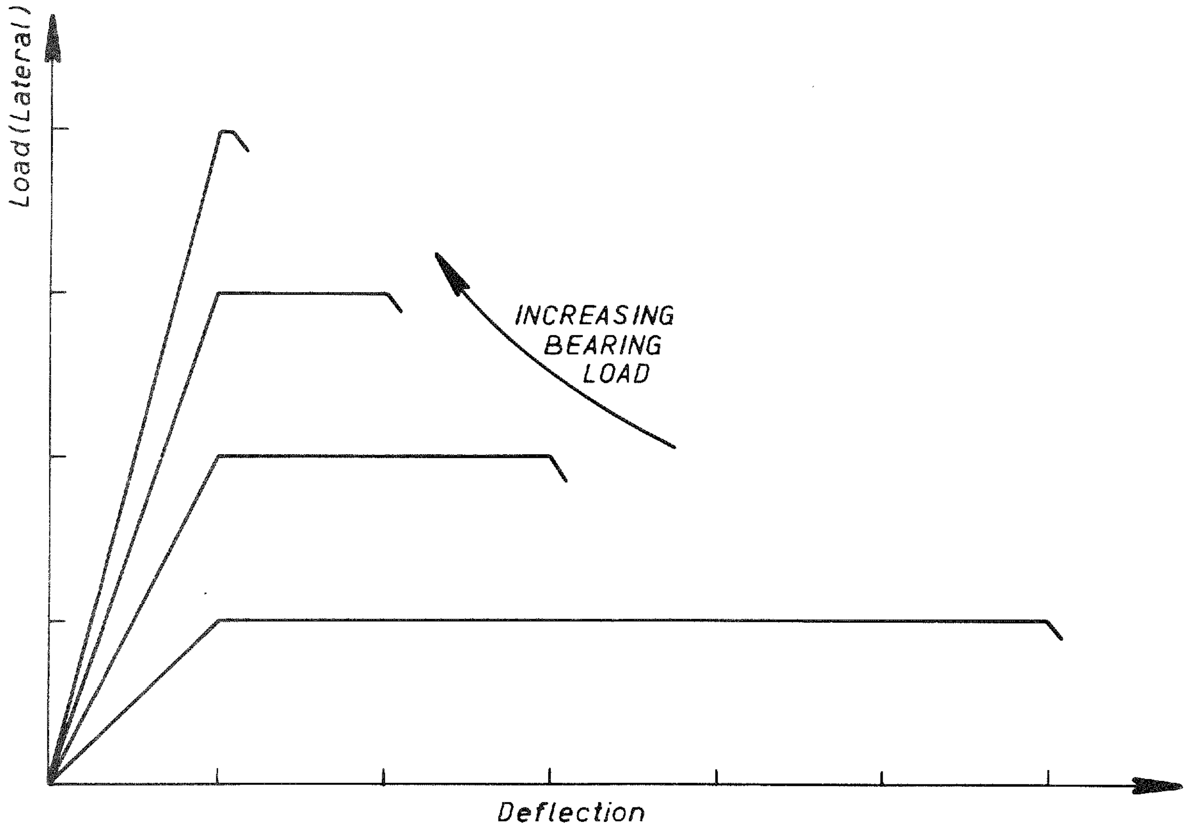



Fig. 5 : IDEALISED LOAD—DEFLECTION CURVES

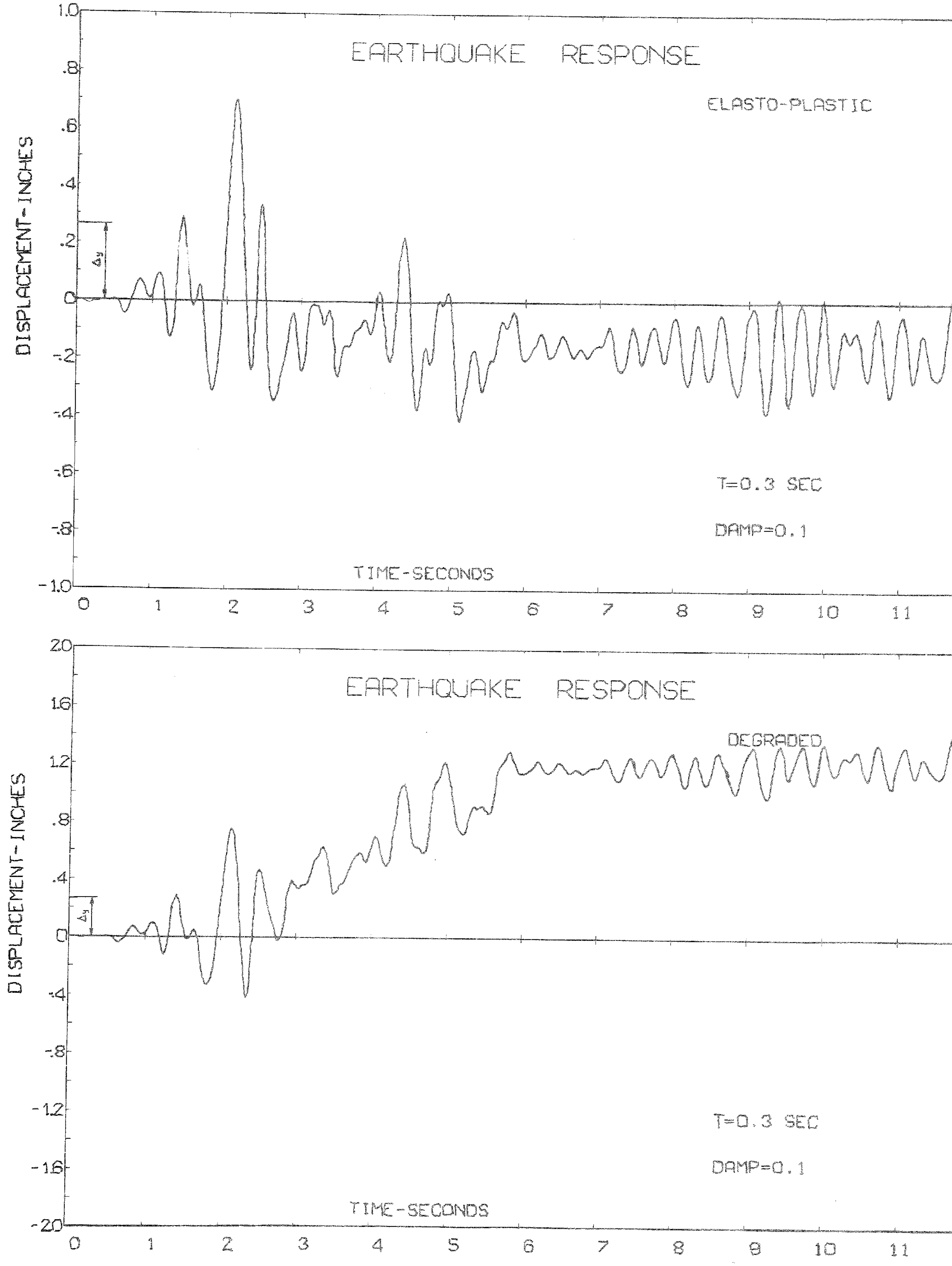
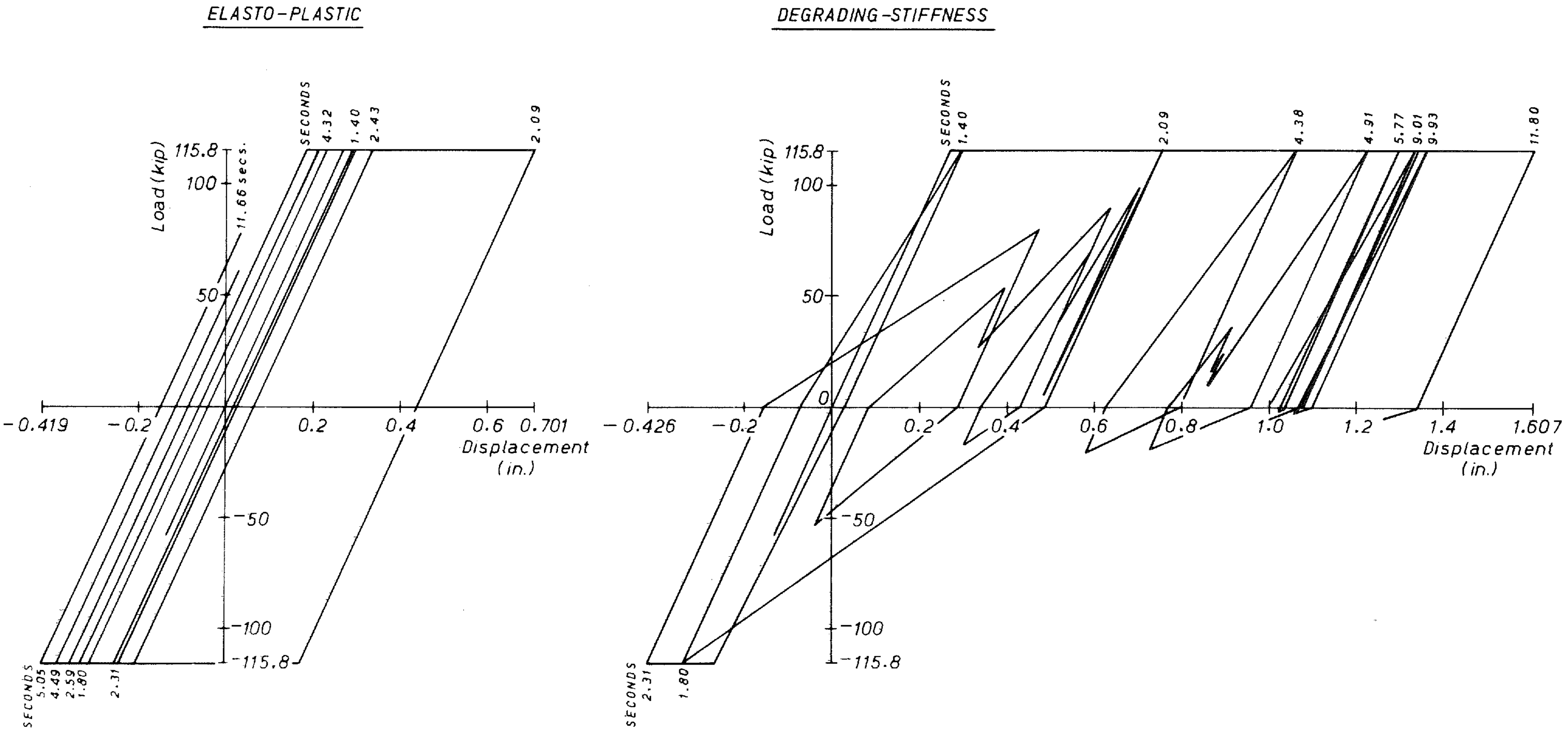

EARTHQUAKE RESPONSE
ELASTO-PLASTIC
DISPLACEMENT-INCHES
T=0.3 SEC
DAMP=0.1
TIME-SECONDS
EARTHQUAKE RESPONSE
DEGRADED
DISPLACEMENT-INCHES
T=0.3 SEC
DAMP=0.1
TIME-SECONDS

Fig. 6




STRUCTURE PROPERTIES :— T = 0.3 secs., λ = 10 %, β = 0.3

El Centro 1940 N-S Earthquake

Fig. 7 : FORCE-DISPLACEMENT DIAGRAMS

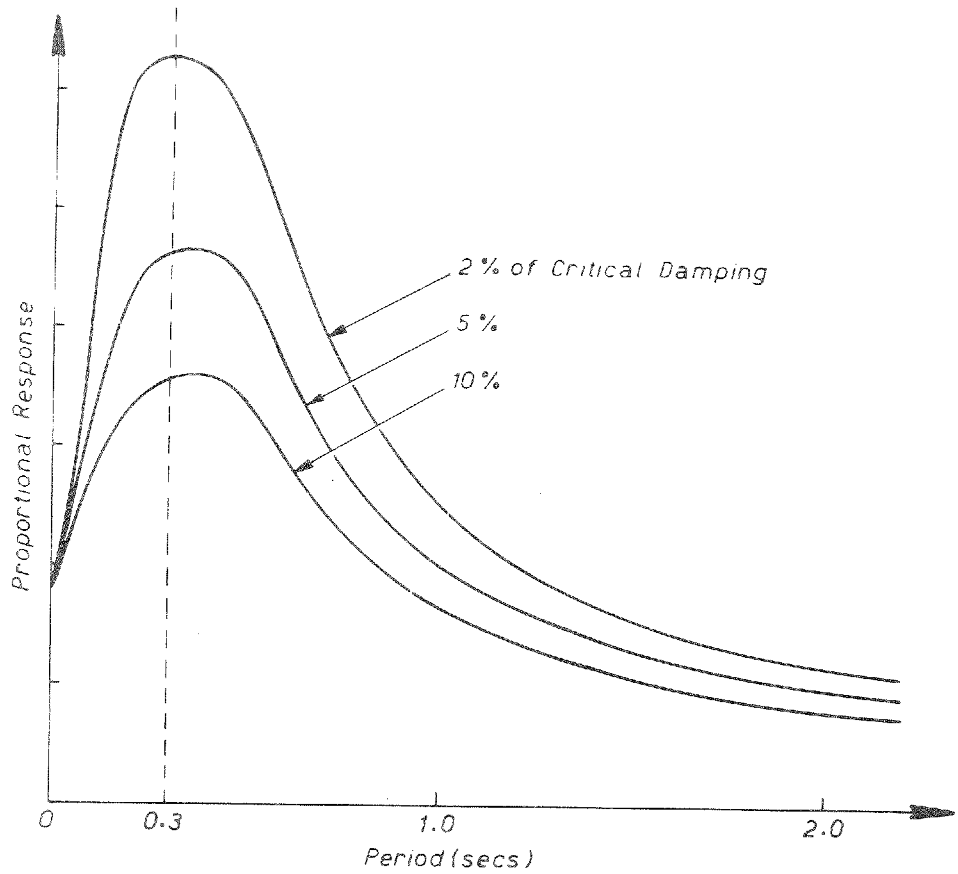



Fig. 8 : AVERAGE EARTHQUAKE RESPONSE SPECTRA

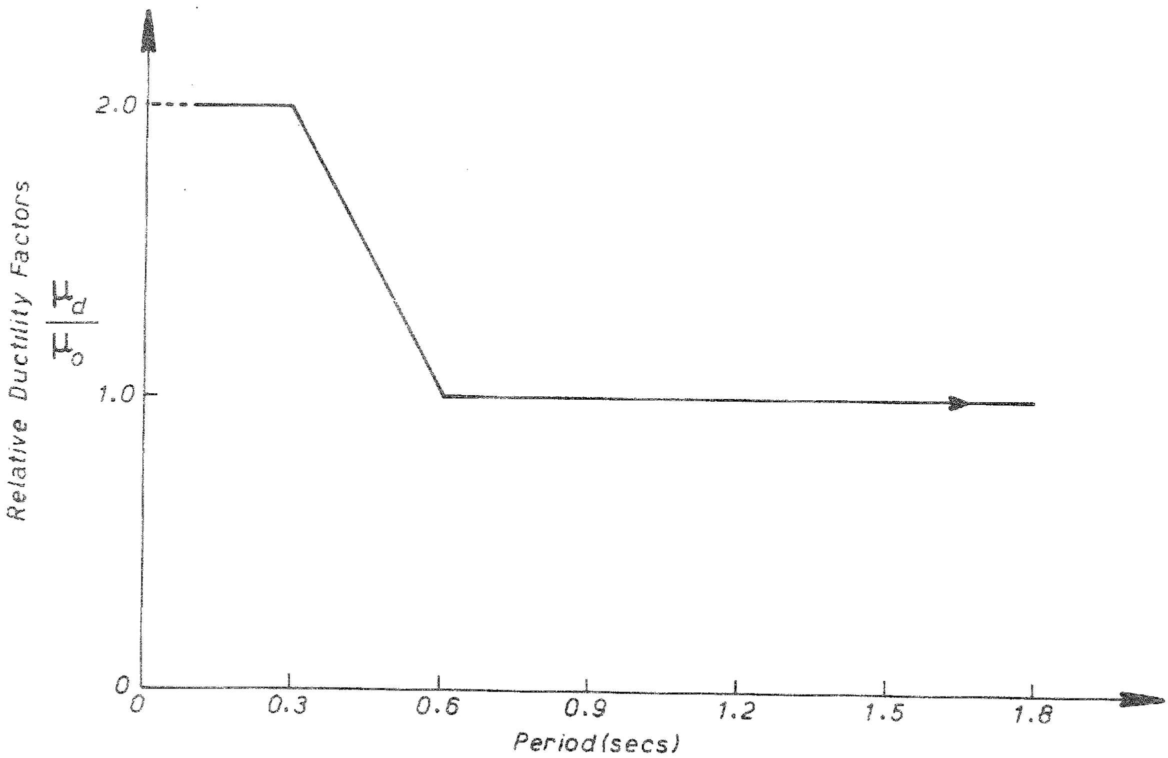



Fig. 9 : IDEALIZED RELATIVE DUCTILITY REQUIREMENTS DEGRADING/ORDINARY