SEISMIC DESIGN OF TRADITIONAL AND PREFABRICATED REINFORCED CONCRETE BUILDINGS

by

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Synopsis

Results concerning the idealization of seismic loads, static and dynamic behaviour of different types of building structures and safety principles are combined to derive simple design rules for usual types of traditional and prefabricated buildings.

The study generalizes concepts derived in the paper "Seismic Design Criteria for Reinforced Concrete Buildings" presented at the III World Conference.

Attention is called to the necessity of improving: i) the statistical definition of seismicity, ii) the information on displacements of non-linear multidegree of freedom systems and iii) the definition of allowable ultimate displacements and ductility factors for the different types of structures.

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1 - INTRODUCTION

The main purpose of the present paper is to derive and justify simple rules for the seismic design of traditional and prefabricated reinforced concrete buildings, based on the present knowledge concerning earthquake loading, static and dynamic behaviour of linear and non-linear systems and on the structural behaviour of reinforced concrete. As the behaviour of the structures is influenced by a large number of factors, it is tried to choose the paramount ones in order to obtain design rules that are simultaneously simple and accurate.

Statistical concepts used for defining the loading and the structural behaviour are combined in order to define the overall safety. The basic safety conditions are established by comparing the displacements due to earthquake loading and the allowable displacements. For this purpose, approximate expressions for the displacements undergone by one-degree and multidegree of freedom systems and for the allowable displacements in reinforced concrete structures are derived.

Two main types of reinforced concrete structures are studied : framed structures and panel structures. The panel structures that correspond to the most usual types of prefabricated buildings are considered in more detail. The force displacement diagrams for these types of structures are obtained on the basis of previous studies. Particular attention is paid to the allowable ductility that corresponds to each of the above mentioned types of structures.

The simplified design rules presented are judged on the basis of the available information concerning the behaviour of structures during earthquakes.

2 - IDEALIZATION OF SEISMIC LOADING

Two main lines are usually followed to obtain a convenient idealization of seismic loading.

i) The study of the maximum response of single-degree of freedom oscillators (with different natural frequencies and viscous damping) to recorded accelerograms using the results thus obtained as a representation of the earthquake (Housner's response spectra).

ii) The assimilation of the recorded vibrations to Gaussian or Poisson processes, stationary or non-stationary, and the direct computation of the quantities that describe these processes using the fundamental concepts of the Random Vibration Theory.

One advantage of this second method is the possibility of numerical or experimental generation of artificial earthquakes that can be used for research purposes.

A correspondence between the two methods can be established. This is particularly simple if the vibration is assumed to be a white noise within a limited band of frequencies (defined by a constant power spectral density of acceleration, S_o) and having a given duration, t.
In fact, the root-mean-square value of the response (expressed in displacement, $\bar{\delta}$) is given by[1]

$$\bar{\delta} = \sqrt{\frac{S}{8\ \eta\ \omega_o^3}} \quad \ldots\ldots\ldots\ldots 1)$$

where η - fraction of critical damping and $\omega_o = 2\pi f$ - circular fre quency of the system.

For a duration $t = 30$ s and for systems within the range 0.5 to 3 Hz* the maximum displacement, δ_{max} can be approximately expressed by[2]

$$\delta_{max} = 2.9\ \delta$$

and thus δ_{max} can be expressed in function of the frequency by

$$\delta_{max} = 0.066\ S^{1/2}\ \eta^{-1/2}\ \bar{f}^{3/2} \quad \ldots\ldots\ 2)$$

For the power spectral density of acceleration indicated by Bycroft[4] to represent N-S 1940 El Centro accelerogram, $S = 700\ cm^2 s^{-4}/Hz$, and for $\eta = 0.05$ (eq.2) comes

$$\delta_{max} \simeq 8\ \bar{f}^{3/2} \quad \ldots\ldots\ldots\ldots\ 3)$$

δ being expressed in cm and f in Hz.

In a previous paper[3] the expression

$$\delta_{max} = 6\ \bar{f}^{1} \quad \ldots\ldots\ldots\ldots\ 4)$$

was used : As it shall be seen expression 3) corresponds to a better approximation than expression 4)

The maximum displacements computed according to expression 3) can be directly compared with those indicated by the response spectra. Fig. 1 gives in logarithmic diagram the response spectra for N-S 1940 El Centro accelerogram[5] and also the spectra that correspond to the accelerograms simulated by Jennings, Housner and Tsai[5]. All these response spectra are reduced to the same intensity as that of El Centro Earthquake.

As is well known response spectra are often idealized by two lines, $\delta_{max} = k'\ f^{-2}$ and $\delta_{max} = k''\ f^{-1}$. Fig. 2 shows that the first one of these lines is convenient for frequencies higher then 3 Hz and the second one for frequencies smaller than 0.5 Hz. The disadvantage of this bi-linear representation results from the fact that the lowest accuracy is obtained in the region 0.5 to 3 Hz (T = 0.3 to 2 s), the most important one for the design of structures.

The line that corresponds to $\delta_{max} = k\ f^{-3/2}$, is tangent to the mean response spectra at the point f = ~ 2 Hz (fig.2). It is thus convenient for representating the spectra within the nearby frequencies.

Another advantage of the expression $\delta_{max} = k\ f^{-3/2}$ results from the fact that the coefficient k can be directly related to the power spectral density of acceleration according to expression 2).

Conversely the power spectral density of acceleration can be expressed as a function of k by

$$S = 230\ \eta\ k^2 \quad \ldots\ldots\ldots\ldots\ 5)$$

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* - Hz - Herz - cycles per second

For η=0.05 and k=12 cm $s^{-3/2}$ it is obtained S=1700 $cm^2\ s^{-4}$/Hz, more than twice the value S = 700 $cm^2 s^{-4}$/Hz proposed by Bycroft to represent the N-S 1940 El Centro accelerogram. The reason for this discrepancy derives from the non-stationarity of the record. In fact, using the first 6.5, 13 and 30 s of 1940 El Centro accelerogram, the power spectral densities indicated in fig.3 are obtained. Obviously the shorter the interval used to estimate the power spectral density of acceleration the larger the interval of confidence of the corresponding estimate. Even so, it is evident that within the 30 s duration the first few seconds correspond to a more intense vibration. Although it is in fact so, the computed variations of power spectral density still fall within the interval of confidence corresponding to the probabilities 0.1 and 0.9. Thus from a statistical point of view these variations could be accepted as due to sampling within a stationary random process.

In the present paper the power spectral density 700 $cm^2 s^{-4}$/Hz is retained as a standard value (for an earthquake with 30 s duration). The maximum displacement for linear one-degree of freedom oscillators (0.5 < f < 3 Hz, η = 0.05) is then computed by the expression

$$\delta_{max} = 8\ f^{-3/2} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad 6)$$

δ expressed in cm and f in Hz.

3 - IDEALIZATION OF STRUCTURAL BEHAVIOUR

As with the seismic loading the structural behaviour has to be conveniently idealized. A sufficiently accurate representation of the structural behaviour is difficult to obtain due to the following reasons:

i) Buildings are in general formed by both structural and non--structural elements (such as partitions) the latter being often disregarded when the resistance to horizontal forces is computed. The influence of these non-structural elements on the dynamic behaviour may be paramount. During intense earthquakes partitions may be heavily damaged at one or several stories and the type of structural behaviour is directly affected by these damages.

ii) Under strong earthquakes the behaviour of the structure itself is no longer linear. Thus the force displacement relations cannot be expressed by simple stiffness matrices and non-linear relations have to be introduced.

iii) Dynamic behaviour may be much influenced by the interaction between the soil and the structure.

For instance, in the case of a building formed by a framed structure infilled by partitions, initial behaviour corresponds to that of a cantilever on an elastic support. The transverse section of the cantilever is obtained by considering the contribution of the frame and of the walls. As such a cantilever is very stiff the deformation of the elastic support (soil) in general represents a large part of the total displacements. If the partitions suffer severe damage at one level, the behaviour immediatly changes to a shear type. The displacements at the damaged level are dominant and the behaviour is then well represented by a one-degree of freedom oscillator. As the damage in the partitions increases pure frame behaviour becomes dominant.

Thus the dynamic characteristics of the building, and particular ly the vibration modes, change greatly from stage to stage. It may well be that the smallest safety against collapse does not correspond to the last of the mentioned stages but to an intermediate one. This is the reason why when designing a structure the safety at different stages has to be studied.

4 - SAFETY CONDITIONS

It is considered that safety conditions can be expressed in terms of ultimate displacements. The limit states to be associated with this ultimate displacements are:

i) local ruptures in structural elements

ii) excessive damage in non-structural elements

iii) overturning

The last condition in general does not apply to buildings and thus only the two first ones are studied.

The ultimate displacements that correspond to local ruptures in structures of shear type are relatively simple to establish. In fact the displacements are independent from story to story. This is not the case in structures where the stiffness of the beams or the floor structure is insufficient to prevent rotation of the joint blocks. In this case the interaction between stories has to be considered.

The non-linear behaviour of reinforced concrete structures can be studied under simple hypotheses (7,8). When the deformations due to bending are predominant it is sufficient to define moment-curvature diagrams and to compute the displacements by integrating the curvatures. Rupture is considered to be attained when compressive strain of $3.5^{o}/oo$ in the concrete or tension strain of $10^{o}/oo$ in the steel are reached. These limits thus correspond to ultimate curvatures, and consequently, to ultimate displacements.

The limitation of displacement at one story to avoid excessive damage on non-structural elements has to be established taking into consideration the type of non-structural elements used. In the case of brick masonry partitions, recent tests have confirmed that displacements of more than 3 cm between stories of usual height correspond to very severe damage (9).

5 - STATISTICAL CONCEPTS

In discussing on a statistical basis the problem of structural safety for earthquake loading, the following main aspects have to be considered.

i) The seismicity of the region has to be statistically defined by the probabilities of exceeding given spectral densities of acceleration, during a given interval of time.

ii) To each value of the power spectral density of acceleration of the soil corresponds a statistical distribution of the response of the structure.

iii) The condition of rupture (or of attaining an ultimate state) is also defined by statistical distributions.

The probability of rupture during a given interval of time has thus to be computed by combining the randomness arising from the three mentioned sources.

It can be shown that the first one is the most important source of randomness and, thus, the probability of rupture is approximately given by the probability of occurrence of a power spectral density of acceleration that produces mean maximum displacements that exceed the mean value of the displacements that correspond to rupture(10,11,12). This consideration simplifies the problem considerably. In fact, the response of the structures can be simply studied by analysing the mean maximum values, where maximum refers to the maximum during an earthquake and mean to the mean of the maxima corresponding to several earthquakes. The expressions indicated in § 2 give mean maximum values. By the same reasoning it is also justifiable to disregard the statistical distribution of the mechanical properties and to retain only the mean values. Note that this is no longer so if the randomness of behaviour during a given earthquake is to be studied.

The data at present available for studying the statistical distribution of earthquakes in a given region are scarce. This is particularly true if the power spectral density of acceleration or the response spectra are taken as a measure of this intensity. For the magnitude, there exist already satisfactory estimates of the statistical distributions for several regions. However the relationship between magnitude and intensity of vibration of the soil cannot yet be satisfactorily established (13). This relationship is influenced by many parameters of which local soil conditions are of fundamental importance. For instance the paramount influence of soil conditions is quite apparent in the 1967 Caracas earthquake (14).

Data expressed in Mercalli or analogous scales also can be treated statiscally but their qualitative character cannot be forgotten.

A Bayesian approach that uses all the available information seems to be the most convenient one. A Bayesian estimate based only on the Mercalli scale was recently presented by Benjamin (15).

6 - RESPONSE OF ONE-DEGREE OF FREEDOM SYSTEMS

Housner's spectra are very often used as an idealization of seismic loading directly giving the response of linear oscillators. A simple relation between the mean maximum displacements and the power spectral density of acceleration was derived in § 2.

It is now particularly important to analyse the influence of non-linear behaviour. Numerous studies on this problem have been performed. The following results obtained by Ravara (16) once more confirm the well known conclusion that within the range of ductilities of pratical interest the displacement of linear and non-linear oscillators having the same natural frequency are approximately equal.

To represent the behaviour of reinforced concrete accurately, bi-linear force displacement diagrams, with parabolic transitions, indicated in fig.4, were used. The oscillators, with a fraction of critical damping η =0.05, were acted by the 1940 and the 1934 El Centro, N-S accelerograms reduced to the same power spectral density of acceleration computed in 30 s. Thus the accelerations of 1934 record were multiplied by 1.06 in order to get $S = 700\ cm^2s^{-4}/Hz$. The results obtained are indicated in fig.5.

To interpret the results it is important to consider the ductility factor, μ , defined as

$$\mu = \frac{\delta_{max}}{F_{max/K}} \quad \ldots\ldots\ldots\ldots \quad 7)$$

where F_{max} is the force that corresponds to the ultimate displacement δ_{max}, fig. 6. Note that according to the adopted definition, the ductility factor also corresponds to

$$\mu = \frac{F_e}{F_{max}} \quad \ldots\ldots\ldots\ldots \quad 8)$$

Thus, the maximum force F_{max}, in a non-linear system, may be obtained by dividing the elastic force F_e by the ductility factor.

The analysis of fig.5 shows that for 1934 El Centro accelerogram amplified to the power spectral density of 700 $cm^2 s^{-4}/Hz$ the maximum displacement of bi-linear systems having ductility factors below about 4 are well represented by the lines $\delta_{max} = 8\ f^{-3/2}$. For the 1940 El Centro accelerogram the line $\delta_{max} = 12\ f^{-3/2}$ gives a better fit. These results are similar to those obtained in § 2 for linear oscillators.

Assuming that the displacements of linear and non-linear systems having the same frequency with ductilities lower than 4 are equal, a simple expression can be deduced for the maximum force, F_{max}, and hence also for the correspondent seismic coefficient.

The seismic coeffficient c is given by

$$c = \frac{F_{max}}{g\ M} = \frac{F_e}{\mu\ g\ M} = \frac{K\ \delta_{max}}{\mu\ g\ M} \quad \ldots\ldots\ldots \quad 9)$$

where $g\ M$ is the weight of the structure. But

$$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}} \quad \ldots\ldots\ldots\ldots \quad 10)$$

and thus

$$\frac{K}{M} = 4\ \pi^2\ f^2$$

Taking $\delta_{max} = 8\ f^{-3/2}$ the seismic factor can be expressed in function of the natural frequency, f, and the ductility factor, μ, by

$$c = \frac{32\ \pi^2}{g}\ \frac{\sqrt{f}}{\mu} = \sim 0.32\ \frac{\sqrt{f}}{\mu} \quad \ldots\ldots \quad 11)$$

This seismic factor corresponds to $S = 700\ cm^2 s^{-4}/Hz$ and $\eta = 0.05$. For different values of S and η the following expression is easily derived (see equation 2))

$$c = \frac{0.264\ \pi^2}{g} \cdot \frac{\sqrt{S}}{\sqrt{\eta}}\ \frac{\sqrt{f}}{\mu} = 2.65\times10^{-3}\ \frac{\sqrt{S}}{\sqrt{\eta}}\ \frac{\sqrt{f}}{\mu} \quad \ldots\ldots \quad 12)$$

S being expressed in $cm^2 s^{-4}/Hz$ and f in Hz.

The above expressions only apply for $f < 3$ Hz. For $f > 3$ Hz the maximum displacement can be taken as $\delta_{max} = k' f^{-2}$ and thus c is independent of f. The value of c that corresponds to 3 Hz, $c = \frac{0.55}{\mu}$, can thus be adopted for $f > 3$ Hz.

Expressions 11) and 12) show that the seismic factor is inversely proportional to the ductility factor. The influence of the ductility is thus very important. As the maximum displacements and natural frequencies are directly related, the seismic factor can also be expressed as a function of δ_{max} by

$$c = \frac{0.64}{\mu} \delta_{max}^{-1/3} \qquad \text{13)}$$

7 - RESPONSE OF MULTIDEGREE OF FREEDOM SYSTEMS

Simple and general results expressing the behaviour of linear and non-linear multidegree of freedom systems are difficult to derive. In fact, a large number of parameters influence the behaviour of such systems.

Computations performed for typical multistory buildings show that the displacements at the top for shear type structures are about 30% higher than those that would be expected for a single-degree of freedom system having the same natural frequency. Thus, for the considered standard earthquake, displacements at the top can be roughly estimated by

$$\delta_{max} = 10 \, f^{-3/2} \qquad \text{14)}$$

δ expressed in cm and f in Hz.

For other types of structures the percentage of increase must be established considering the shape of the deformed structure.

The influence of non-linear behaviour does not much affect the maximum displacements, just as was mentioned for one-degree of freedom systems. Results obtained by Clough and Benuska (17) for bending type structures and by Serac Committee (18) and others for shear type structures confirm this assertion.

Note that although the maximum displacements are not much affected by the non-linear behaviour, variations of stiffness of the different elements and of the relations between ultimate strengths very much influence the shape of the deformed structure and the points where non-linear deformations concentrate.

Results obtained by Ravara (16) concerning the behaviour of typical shear type structures of reinforced concrete buildings (fig.7) confirm this statement. Bi-linear force displacement diagrams with parabolic transitions are adopted at each story in order to reproduce the behaviour that corresponds to the assumed dimensions and percentages of reinforcement of the columns. Materials are considered to be concrete with a cube strength of 300 kg/cm^2 and steel with a limit of proportionality at 0.2% of 4 000 kg/cm^2. The structures were preliminarily designed for a seismic factor of 0.15.

Variations of stiffness from one level to the other greatly influence the distribution of the maximum displacements. The overall ductility of the structure is thus much affected by this uneven distribu tion of plastic deformations.

The notion of ductility factor can be generalized for multidegree of freedom systems by taking for reference the displacements at the top of the buildings. The overall ductility of the structure, μ_s , is defined as the relation between the maximum non-linear displacement at the top and the elastic displacements that correspond to the maximum force. It is easy to understand that this ductility is in general much smaller than the ductility of the elements that form the structure, μ_e.

The seismic factors to be adopted for multidegree of freedom systems can then be computed by

$$c = \frac{10}{8}\, 0.32 \, \frac{\sqrt{f}}{\mu_s} = 0.40 \, \frac{\sqrt{f}}{\mu_s} \quad . \; . \; . \; . \; . \quad 15)$$

where μ_s is now the overall ductility of the system.

8 - SIMPLIFIED DESIGN RULES FOR FRAMED STRUCTURES

Two main types of structural behaviour have to be considered: i) shear at any story level and ii) general frame behaviour.

Rules for studying shear of reinforced concrete buildings at one-story level were indicated in former papers (3, 19). There it was shown that for not exceeding the allowable displacements of the reinforced concrete columns it is necessary to limit the normal stress in the columns to about 0.2 of the strength of the concrete. If this rule is followed the ductility of the columns can be taken as about 3.5, for the usual percentages of reinforcement and qualities of materials. In fact the ductility of reinforced concrete columns perfectly built-in decreases for increasing percentage of reinforcement from about 4 to about 2. In practice, this decrease of ductility can be disregarded since for high percentages of reinforcement ultimate concrete strains higher than 3.5°/oo can be allowed.

The maximum displacements between two stories have also to be limited as a function of the type of partitions, to avoid excessive damage. As mentioned in § 3, for the usual type of brick masonry partitions , the maximum value of 3 cm can be adopted. Thus, in order not to exceed this limit, the natural frequency (expression 3) must be higher than

$$f > \sqrt[3]{\left(\frac{8}{3}\right)^2} \simeq 2 \text{ Hz}$$

For f = 2 Hz and μ = 3.5 the seismic factor (expression 11) becomes

$$c = 0.32 \, \frac{\sqrt{2}}{3.5} = 0.13 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad 16)$$

As it was seen in § 6, no increase of the seismic factor in function of the frequency has to be considered for frequencies higher than 3 Hz. Thus for designing buildings for shear at one story level it is not important to consider the influence of the natural frequency and a mean value of 0.15 corresponding to frequencies between 2 and 3 Hz can be adopted for the assumed standard earthquake. Note that the elastic stiffness must be sufficient to guarantee a natural frequency higher than 2 Hz for the behaviour corresponding to shear at a single level.

For studying general frame behaviour the frequency of only the first mode need be considered and the seismic factor is given by expression 15).

As the number of stories increases the natural frequency and the overall ductility decrease. The example of fig.7 show that the ductility of 6 to 8 story frames varies between 1.6 and 2.1 when the ductility of the elements vary between 2 and 5. The consequence of this simultaneous variation of frequency and ductility is that the final values of the seismic factors vary very little with the number of stories.

To improve the design of multistory buildings it is necessary to define their ductility conveniently. The solution of considering $\mu_s = 1$ may be too much on the safe side, particularly if the buildings are not very high. Therefore an improvement in seismic design depends considerably on the knowledge of the ductilities of the elements and on the determination of the overall ductility of the structure.

9 - SIMPLIFIED DESIGN RULES FOR PANEL STRUCTURES

Although many studies on panel structures and particularly on prefabricated panel structures have been recently performed, only a limited knowledge of the static behaviour of a few types of these structures is available. The information concerning dynamic behaviour is very scarce (20).

The generalization to panel structures of the rules presented above implies a knowledge of their natural frequencies, damping and ductility factors. If these elements are known the seismic factors can be computed and the problem is then transformed to the design for static horizontal forces.

As the structures are formed by panels these elements have to be studied deeply in order to forecast their force displacement diagrams. By assembling panels of known behaviour the overall behaviour of the structure can be analysed.

Recent studies by Trigo (9) concerning brick masonry and concrete panels, interconnected by reinforced concrete bracing, have shown that the following main phases of the force displacement diagrams have to be considered: i) behaviour prior to cracking, ii) perimetral cracking between panels and bracing, iii) diagonal cracking of the panels, iv) plastic deformation of the panels or bracing. The importance of each of these phases depends largely on the geometry of the panels, on the mechanical properties of the materials and on the normal force acting on the panels. For each of the referred phases a different structural idealization has to be adopted in order to obtain sufficiently accurate force displacement diagrams.

Fig. 8 refers to model tests of panels and shows a typical force displacement diagram. For single panels, as their elastic stiffness is very high, very high values of the ductility factor are obtained even for small maximum displacements. In this case the ductility factor is meaningless.

Force displacement diagrams for two prefabricated buildings are exemplified in fig.9. The structure of the buildings is formed by 3 × × 3 m brick masonry panels interconnected by reinforced concrete brac ing.

The displacements at the top are mainly due to bending of the composite cantilever and to rotation of the foundation. Thus the plastic deformation by shear at one level contributes very little to the total deformation of the structure. For the 5 story building the ductility factor takes the value 1.5 and for the 10 story building the value 1.2.

10 - CONCLUSIONS

The results presented provide the following conclusions:

10.1 - For an improved definition of the seismic loading it is particularly important to define the seismicity in probabilistic terms, even using subjective probabilities. A deeper knowledge of the relation between magnitude and power spectral density of acceleration in the bed-rock and of the amplification due to the overlying soils is also of paramount importance. For the design of buildings, the principal types of idealizations of seismic loading used at present are pratically equivalent.

10.2 - The randomness of seismic loading exceeds all the other sources of randomness. This fact simplifies the problem, as the safety conditions can be established by comparing mean maximum values of the response with mean values of the ultimate displacements. Analytical and experimental methods at present available allow the accurate determination of the response of the structure. On the other hand the information at present available concerning allowable ultimate displacements is still very scarce.

10.3 - The simplified expressions derived show that seismic factors are inversely proportional to the ductility factors. Thus the correct definition of ductility factors both for traditional and prefabricated structures is of foremost importance.

10.4 - An increase of the number of stories corresponds to a decrease in the natural frequency and also to a decrease in the overall ductility. This explains why in a first approach, for the usual range of variation of the number of stories, it is not justifiable to vary the seismic factors by much. The interpretation of the damage undergone by buildings during the 1960 Agadir (21) and the 1967 Caracas (14) earthquake confirms this assertion.

10.5 - Although for usual cases simplified design rules are adequate, it cannot be forgotten that these rules can only be improved by special studies. For important structures and particularly for tall buildings specific dynamic analysis or model tests lead to significant benefits in safety and in economy.

10.6 - For improving seismic design the following themes of research are considered among the most promising: definition of seismic loading in order to estimate the probability of exceeding given seismic intensities; systematic dynamic analysis of multidegree of freedom systems to establish the influence of the main typical parameters on the mean maximum displacements; studies on the principal types of elements and structures to determine their allowable ultimate displacements and ductilities; and observation of the behaviour of structures during earthquakes including interpretation of the damage as a function of the actual characteristics of the structures.

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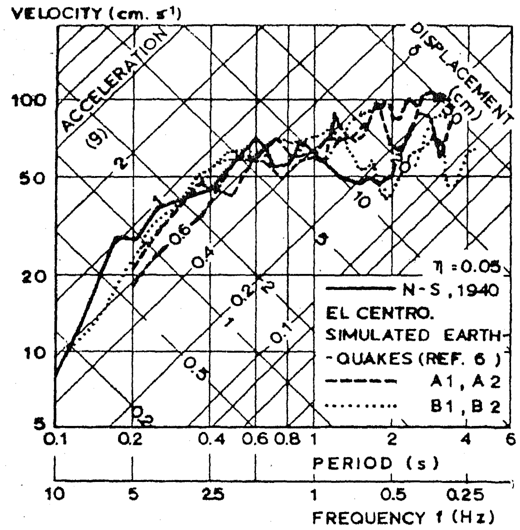


Fig.1 - LOGARITHMIC SPECTRA FOR 1940 EL-CENTRO AND SIMULATED EARTHQUAKES

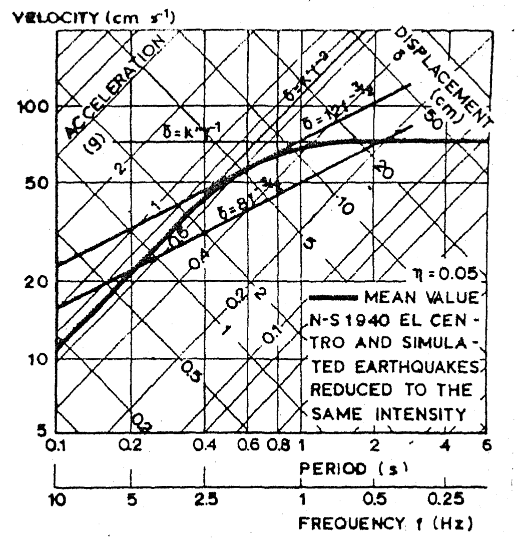


Fig. 2 - ANALYTICAL REPRESENTATION OF MEAN VALUE OF SPECTRA

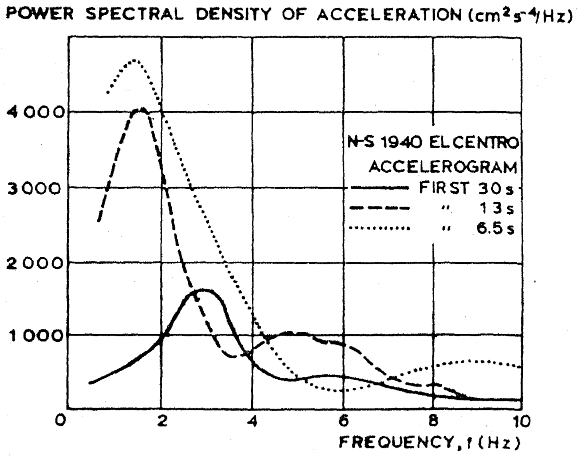


Fig. 3 - POWER SPECTRAL DENSITY OF ACCELERATION, COMPUTED FOR THE FIRST 30, 13 AND 6.5 s OF N-S 1940 EL CENTRO ACCELEROGRAM

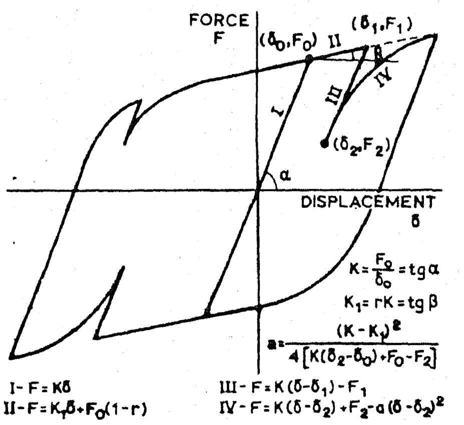


Fig. 4 - BI-LINEAR FORCE DISPLACEMENT DIAGRAM WITH PARABOLIC TRANSITIONS

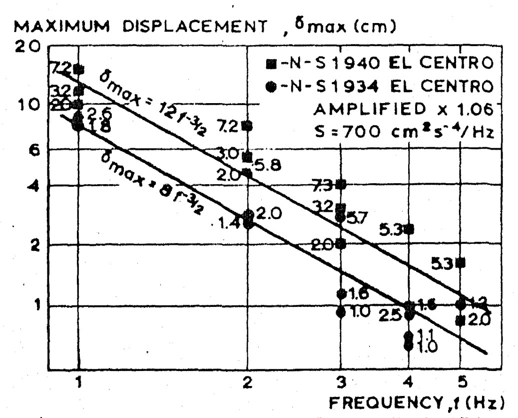
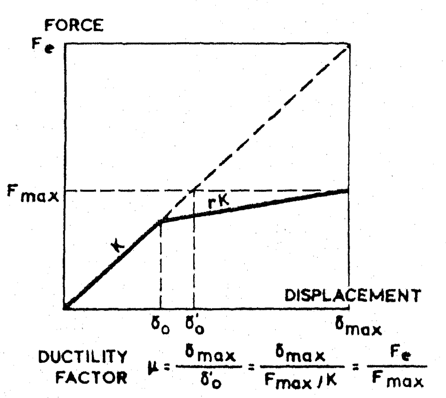
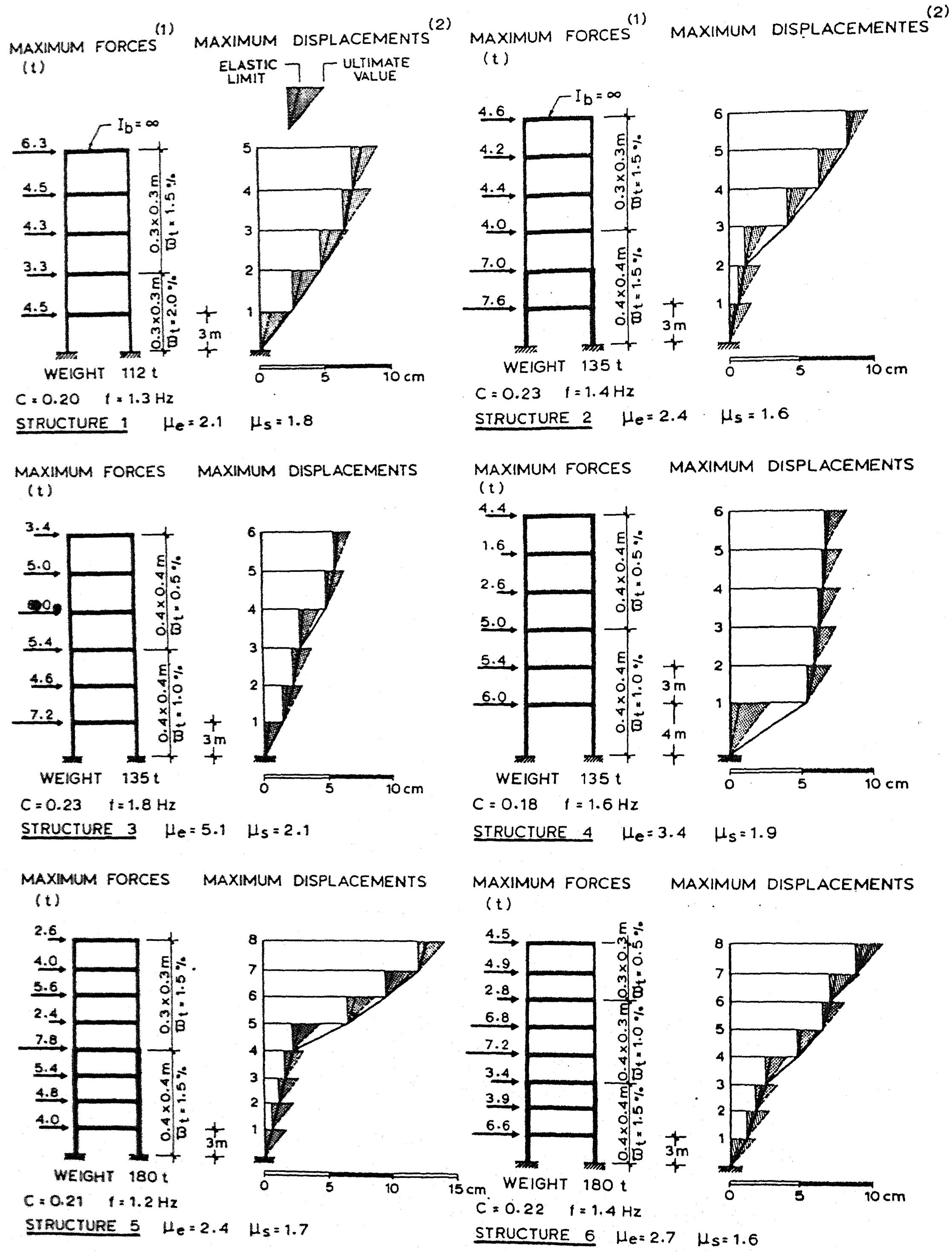


Fig. 5 - MAXIMUM DISPLACEMENTS AND DUCTILITIES OF BI-LINEAR OSCILLATORS WITH PARABOLIC TRANSITIONS



$$\text{DUCTILITY FACTOR}\quad \mu = \frac{\delta_{max}}{\delta'_o} = \frac{\delta_{max}}{F_{max}/K} = \frac{F_e}{F_{max}}$$

Fig. 6 - DEFINITION OF DUCTILITY FACTOR



1) SIMULTANEOUS FORCES THAT CORRESPOND TO MAXIMUM SHEAR AT THE BASE

2) MAXIMUM RELATIVE DISPLACEMENTS DURING THE EARTHQUAKE (NON – SIMULTANEOUS)

FIG. 7 – BEHAVIOUR OF SHEAR TYPE STRUTURES

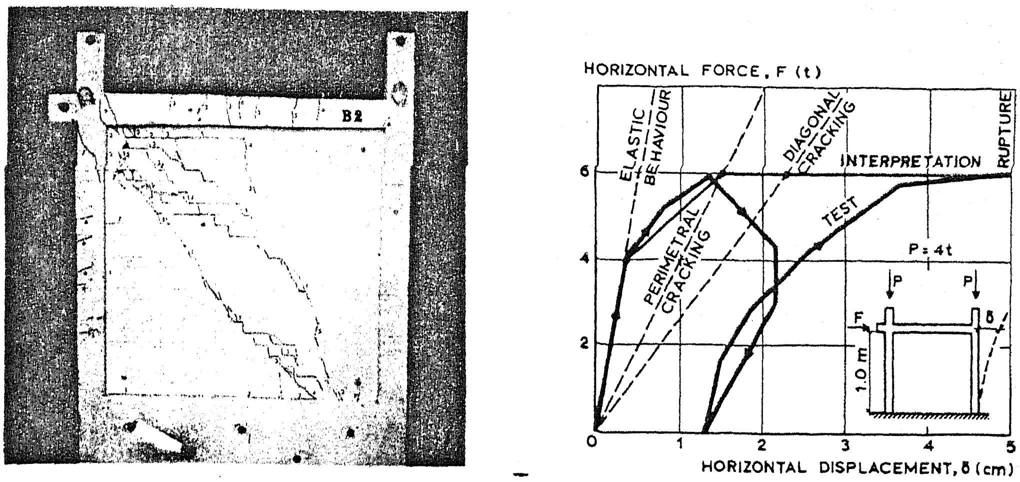
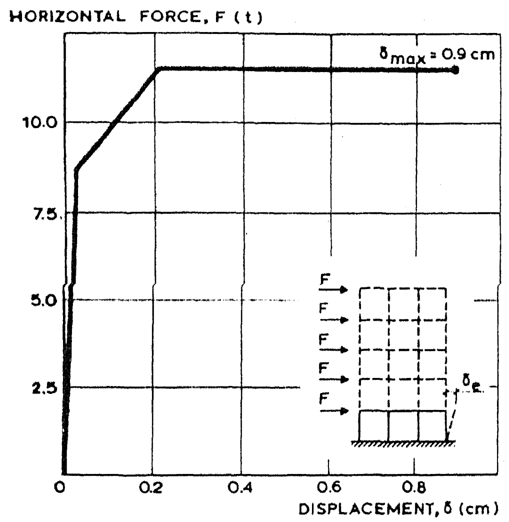
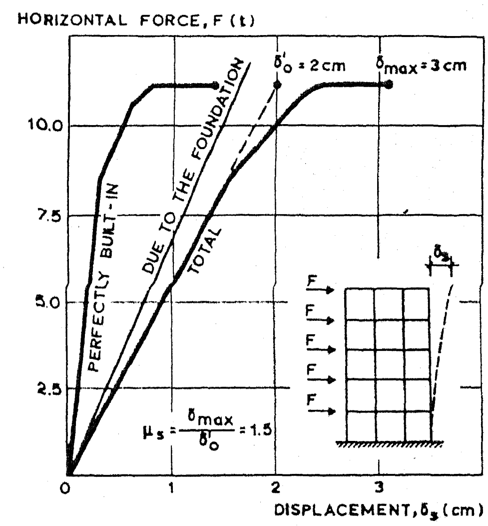


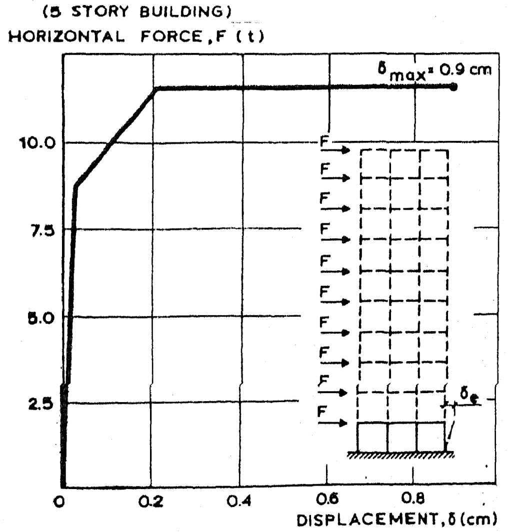
Fig. 8 - MODEL TEST OF BRICK MASONRY PANEL WITH REINFORCED CONCRETE BRACING



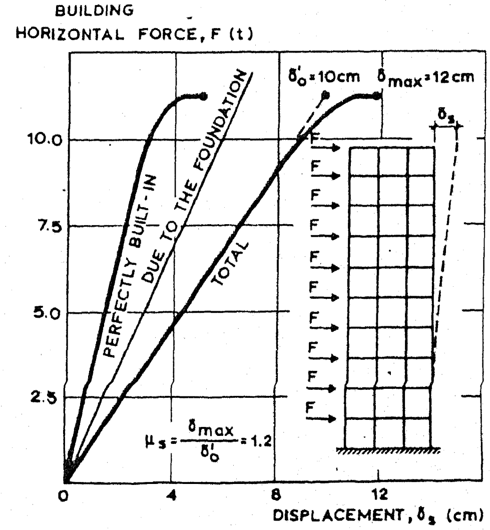
a) FORCE DISPLACEMENT DIAGRAM FOR FIRST STORY (5 STORY BUILDING)



b) FORCE DISPLACEMENT DIAGRAM FOR 5 STORY BUILDING



c) FORCE DISPLACEMENT DIAGRAM FOR FIRST STORY (10 STORY BUILDING)



d) FORCE DISPLACEMENT DIAGRAM FOR 10 STORY BUILDING

Fig. 9 - BEHAVIOUR OF PANEL STRUCTURES