SEISMIC DESIGN OF BUILDINGS USING A TIME-HISTORY METHOD

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SYNOPSIS

A study is made of the feasibility and usefulness of a seismic design method for multistorey frame buildings. The method employs a time-series elastic response analysis of the structure for a ground motion compatible with a design response spectrum. The correlation between the elastic and elasto-plastic response is investigated and it is suggested that the design forces in the members of an elasto-plastic structure can be obtained by applying appropriate reduction factors to the forces obtained in an elastic analysis.

A design example is presented in which a multistorey steel-frame building with a rather large setback is designed for seismic forces by using the results of an elastic dynamic analysis for a selected ground motion and reducing the forces so obtained by applying one uniform force reduction factor to all girder moments and another, a smaller one, to all column moments and axial loads.

RESUME

Dans cet article on discute d'une méthode de calcul sismique des cadres rigides multi-étagés. On étudie la corrélation qui existe entre la réponse élastique et la réponse élasto-plastique de la charpente et on suggère que les forces agissant sur une ossature élasto-plastique soient déterminées en appliquant des facteurs de réduction sur les forces obtenues par une analyse élastique.

Pour illustrer la méthode proposée, on présente les calculs d'un bâtiment multi-étagé en acier comportant un décrochement assez important. On dimensionne les cadres en utilisant les résultats d'une analyse dynamique de la réponse du bâtiment à une accélération déterminée du sol. Les forces qu'on obtient de cette analyse sont réduites en utilisant un facteur de réduction uniforme pour tous les moments dans les poutres, et un autre facteur, un peu plus petit, pour tous les moments et les charges axiales dans les poteaux.

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INTRODUCTION

For the seismic design of building structures that are regular in shape and in their distribution of mass and stiffness, the codes generally prescribe an equivalent static load approach which converts the dynamic problem into a problem of static analysis. At the same time it is recognized that a dynamic analysis is essential for the design of irregular structures where the equivalent static load approach may not be quite adequate.

Commentary K to the National Building Code of Canada (5) gives specific recommendations on the use of dynamic analysis for seismic design. The recommended procedure is to conduct an elastic dynamic analysis of the structure for an appropriate design ground motion. The ground motion can either be in the form of a response spectrum, in which case the elastic dynamic forces are obtained by the mode superposition method, or it could be in the form of an acceleration record, when a time series analysis must be used. It is accepted that most buildings need be designed for only a fraction of the forces obtained through elastic analysis, provided care is taken in the choice of material and detailing of the structure, so that it can absorb energy through inelastic deformation.

The inelastic deformation of a structure is customarily measured in terms of a ductility factor, which is defined as the ratio of the total elasto-plastic deformation to the yield deformation of the structure. Any representative deformation, such as a typical storey displacement or an interstorey displacement, can be used in this definition. Obviously, ductility demand should not exceed the ductility capacity of the structure.

Studies carried out on single degree of freedom systems tend to show that the maximum displacement in an elastic system is approximately equal to that in an elasto-plastic system with the same mass and the same initial stiffness. It is argued, on the basis of these results, that the forces induced in an elasto-plastic system can be obtained by dividing the elastic forces by a reduction factor that is equal to the ductility factor defined above. It is essential that one recognizes the limitations in this approach. First, even for a single degree of freedom system, the assumption that the total elasto-plastic displacement is the same as the displacement of the corresponding elastic system is only an approximation. In fact, the relationship is very much dependent on the ground motion, and large differences may exist between the two displacements. Second, the results of a single degree of freedom system cannot be extrapolated to a multi-degree of freedom system. Third, the application of a uniform force reduction factor in obtaining the member design forces does not guarantee a uniform ductility demand in all members. In fact, member ductility requirements may be larger than the force reduction factor used in the design. Finally, gravity loads materially alter ductility demands. Some of these factors are studied in this paper through the analyses of a single storey and a three storey frame.

While inelastic deformations are accepted in a building subjected to a severe seismic excitation, it is considered desirable that the inelasticity be confined to girders and that the columns remain elastic. Intuitively, one would think that for this objective to be fulfilled the reduction factors to be applied to the forces obtained from an elastic dynamic analysis should be different for columns and girders. However, the recommended practice (NBC 1977) has been to apply a uniform reduction factor to all the member forces. It is shown in this study that the use of a uniform reduction factor for both the girders and the columns may not be appropriate. In general, the columns that remain elastic will be required to carry additional forces beyond those present at the stage when the girders have started to yield. Therefore, it may be preferable to use a lower force reduction factor for the column moments and axial loads than for the girder moments. Evidence is presented in this paper in support of this viewpoint. The paper also presents an example in which a building with a large setback is designed using a time-series dynamic analysis and two different force reduction factors, one for the girder forces and another for the column forces.

This study is concerned primarily with steel framed buildings, although many of the results are equally applicable to framed buildings of reinforced concrete. Modelling of inelastic behaviour in reinforced concrete members is far more complex on account of cracking and stiffness degradation. Additional work is therefore required to establish a correlation between elastic and inelastic seismic response of concrete frames.

DESIGN GROUND MOTION

The choice of a suitable ground motion is probably the most difficult part of seismic design. One solution is to use a previously recorded or artificially generated ground motion that has been appropriately scaled to account for the characteristics of the site. However, different records, even though of the same intensity, may give widely differing structural responses; and the design values obtained by using a single record may not be very useful. Obviously, it is preferable to use some form of an average earthquake which is free of any particular bias. The most effective way of defining an average earthquake is by means of its response spectra. Newmark and Blume (6) have derived such average response spectra by taking statistical means of the spectrum curves of several earthquake records normalized to give the same level of intensity. These design spectra are now used by the U.S. Atomic Energy Commission for the design of nuclear power plants and also form the basis for the design spectra given in the National Building Code.

Although a design response spectrum can be used in proportioning a structure, it is necessary to have an explicit description of the ground motion history if a time series analysis is to be carried out. A ground motion that would produce a response spectrum similar to the design spectrum referred to above would be appropriate for this purpose. A method of generating such spectrum-compatible motion is described by Tsai (7). In this method, an existing ground motion record having a response spectrum roughly similar in shape to the design response spectrum is selected and then progressively modified until its response spectrum is rather close to the design spectrum.

For the purpose of this study, two spectrum compatible motions were generated using the above method. The target response spectra chosen for the purpose were those specified by the U.S. Atomic Energy Regulatory Commission (8) and 5% damping. The north-south component of El-Centro 1940 ground motion was used as the base ground motion. This ground motion was first scaled and then progressively modified till its spectrum was reasonably close to a target spectrum. Figure 1 shows a comparison of the target spectra and the response spectra obtained from the compatible motion. All time series dynamic analyses reported in this paper were carried out for the compatible motion for 5% damping; this motion will henceforth be referred to as the standard ground motion.

METHOD OF ANALYSIS USED

The dynamic analyses reported here were carried out by a step-by-step numerical integration of the equations of motion. The computations were performed with the aid of an elastic and an elasto-plastic dynamic analysis program, both developed by the author. The P-Δ effect was not included in the analysis because previous studies (1, 3) had shown that it did not materially alter the response. The maximum displacements, storey shears and member forces generated during the entire history of excitation were automatically computed by the program and printed out at the end of response calculations. Computations for the response spectra referred to above had indicated that the response always reached its maximum value within the first 15 s of the ground motion history. The time series analyses were therefore restricted to a 15 s duration of earthquake.

EFFECT OF INELASTICITY ON RESPONSE

Single Degree of Freedom System

A building frame is expected to become inelastic when vibrating under the design ground motion. Strictly speaking, therefore, the design for seismic forces should be based on an inelastic analysis of the frame for the given ground motion. Such an analysis is, however, very complex and time consuming. Also, because of the difficulty in modelling material behaviour in the inelastic range and the uncertainty inherent in specifying a design ground motion, the use of an overly sophisticated analysis as a design tool is rarely justified. It is therefore customary to obtain the design forces by scaling down the forces obtained from an elastic analysis. In NBC 1977, the reduction factor used in scaling down the elastic forces is taken as equal to μ, the ductility factor, except when the fundamental period of the structure is less than 0.5 s. This is based on the reasoning that the total elastic displacement of the system is nearly equal to the total elastic-plastic displacement. Blume and others (2) have presented the results of elastic and elasto-plastic analyses of a single degree of freedom system subjected to El-Centro ground motion. They found that the maximum relative displacement of an elastic system was not too different from the maximum relative displacement of an elasto-plastic system with the same period of vibration based on initial stiffness. The displacements never differed by more than a factor of 2; for zero damping, they were generally smaller for the elasto-plastic system, but the trend was reversed for 10% damping.

For additional investigation of the general nature of this relationship between the elastic and elasto-plastic single degree of freedom systems, the single storey frame shown in Fig. 2a was analyzed for 15 s of the standard ground motion scaled to represent an earthquake with a maximum ground acceleration of 50% of gravity. The mass of the frame was adjusted to give 18 different values of the period, giving, in effect, 18 different frames. For each frame, an elastic analysis was carried out first. This was followed by an elasto-plastic analysis in which the strength of each member was set at a value equal to one fourth the maximum moment obtained in it during the elastic analysis. With the member strengths set as above, plastic hinges formed at the column bases and at the two ends of the beam simultaneously, giving a single degree of freedom system with a truly elasto-plastic force-displacement relationship as shown in Fig. 2b.

Figure 3a presents a comparison of the maximum elasto-plastic displacement and the maximum elastic displacement for each period of vibration. The results show that the elasto-plastic displacement is generally higher than the elastic displacement except in the long period range. Figure 3b shows the displacement ductility of the frame and the rotation ductility of the girder. The displacement ductility, μ_Δ, is defined as the ratio of total elasto-plastic displacement to the displacement at yield, and is, in this case, equal to four times the ratio of the maximum elasto-plastic displacement to the maximum elastic displacement. The rotation ductility, μ_Θ, is defined as the maximum elasto-plastic rotation at an end hinge to the yield rotation at the same location, and is given by

$$\mu_\Theta = 1 + [\Theta_p + (M_p L/6EI)] \qquad (1)$$

where Θ_p = the plastic hinge rotation,

M_p = plastic moment capacity of the section,

L = length of the member, and

I = moment of inertia of the member.

The results show that the displacement ductility is higher than the force reduction factor, λ. The rotation ductility factor is still higher, at times reaching a value that may be difficult to provide for in design.

To investigate the effect of gravity load on response, elasto-plastic analyses were repeated for all the eighteen frames with a uniformly distributed gravity load added to the girder. In each case, the gravity load was adjusted to give a girder end moment equal to 75% of the maximum earthquake design moment, the latter being equal to one fourth the moment obtained in elastic analysis. The strength of each member was then set as equal to the sum of the design earthquake moment and the corresponding gravity moment. Because of the presence of gravity load, plastic hinges do not form simultaneously at the two ends of the girder, and the force displacement relationship is modified as shown in Fig. 2b. The rotation and displacement ductility factors for frames carrying gravity loads are shown in Fig. 3b. The ductility demand is considerably reduced and is closer to the force reduction factor of 4. In fact, the ratio of gravity moment to the design earthquake moment is likely to be higher than 0.75 in a practical building frame, and an increase in this ratio should result in a further reduction in the ductility demand.

The gravity load moments affect the ductility demand in two ways. First the order in which the plastic hinges form, and as a result the total response, is significantly altered. This is evident from Fig. 2b. Second, the rotation ductility demand, which is a ratio of the total rotation at a plastic hinge to the yield rotation at the same location, changes because the yield rotation increases due to the increased girder strength while the total rotation, in general, decreases.

Multi-Degree of Freedom System

To investigate the relationship between elastic and elasto-plastic response of multi-degree of freedom systems, the three storey frame shown in Fig. 4a was studied. The mass at each floor was assumed to be the same and equal to a reference mass m. The reference mass was varied to give six different values of the fundamental period, ranging from 0.3 s to 0.8 s., thus in effect giving six different frames. An elastic analysis was carried out for each frame for the standard ground motion scaled to 50% g and a damping of 5% of critical in the first mode.

For a single storey frame, plastic hinges normally form at the two girder ends, as well as at the column bases. For a multistorey frame, provided the columns are properly designed, it is possible to avoid the formation of hinges in the columns. Plastic hinges may still have to be allowed at the bases of the first storey columns. In the elasto-plastic analysis of each three storey frame the girder strengths were set at one fourth the maximum moment obtained from elastic analysis. To avoid plastic hinges in the columns, column strengths were kept higher than the maximum elastic moments, except in the first storey, where the strength of each column was set equal to half the maximum moment obtained for that column during the elastic analysis.

The ductility ratios for girders, obtained from the elasto-plastic analyses, are shown in Fig. 4b. It will be observed that, even though a uniform reduction factor of 4 was used in obtaining the girder strengths, the girder ductility is not uniform, and is, in fact, much larger than 4 in some cases.

The three storey frames were next analyzed for the combined effect of gravity loads and seismic vibrations. The gravity load consisted of a uniformly distributed load on each girder. The magnitude of this load was adjusted so that it produced in the first storey girder an end moment equal to the corresponding design earthquake moment. The strength of each girder was then increased so that it was equal to the sum of the earthquake moment and the gravity load moment. The girder ductility ratios obtained from the analyses are shown in Fig. 4c. The ductility demand in all the girders is now observed to be closer to the reduction factor of 4.

The reduction factors obtained for the column moments and column axial loads are shown in Figs 5 and 6 respectively, a reduction factor being defined as the ratio of the maximum elastic response value to the corresponding elasto-plastic response value. Because the first-storey columns were designed to have a strength equal to half the elastic moment plus the full gravity moment, if any, the reduction factors for these columns should obviously be close to 2, and, in any case, not less than 2. Fig. 5 shows that this is true. In some cases, the reduction factor is in fact higher than 2, indicating that the maximum column moments obtained in the elasto-plastic response were less than half their elastic values and that therefore no plastic hinges formed in the first storey columns. The top moment in an uppermost column is constrained to be equal to the top storey girder end moment to satisfy equilibrium; the reduction factor for such a column should thus be equal to 4. The middle storey columns are designed to remain elastic, and it is of interest to note that the reduction factors for these columns is equal to about 2, that is only about half of the reduction factor used for the girders.

The reduction factors obtained for axial loads need some explanation. Figure 7 shows the distribution of moments throughout the length of a girder under various conditions of loading. If the frame is elastic, the maximum girder end moments resulting from earthquake excitation are each equal to M_E, and the supporting columns each receive an axial load equal to $2M_E/L$. If no gravity loads are present and the girder is designed to have a plastic moment capacity equal to M_E/λ, λ being the force reduction factor, the moment distribution will be as shown in Fig. 7b. In this case, the girder will contribute an axial load equal to $2M_E/\lambda L$ to each of the supporting columns. If all girders supported by the column under consideration attain the maximum end moments simultaneously, the design earthquake load in the column will obviously be $1/\lambda$ times the corresponding elastic force. The dashed line in Fig.6 shows that this is true, that is, when no gravity loads are present, the reduction factor applicable to column axial loads is the same as that applicable to the girders.

If the girder end moments due to gravity loads are equal to M_G, the girder must be designed to have a plastic moment capacity of $M_G + (M_E/\lambda)$. Assuming that under the combined effect of gravity and earthquake forces plastic hinges still form at both ends of the girder, the distribution of moments due to earthquake forces should be as shown in Fig. 7c. This distribution will contribute an axial force equal to $2/L[M_G + (M_E/\lambda)]$ to each of the supporting columns. The reduction factor for axial loads, λ_1, is then given by

$$\frac{\lambda_1}{\lambda} = \frac{1}{1 + MG/(M_E/\lambda)} \qquad (2)$$

Thus λ_1/λ depends upon the ratio of the design gravity moment (M_G) to the design earthquake moment (M_E/λ) for the girder. If the two moments are equal, λ_1 is only half of λ. If M_G is twice (M_E/λ), λ_1 is only one third of λ. For the three storey frames, the ratio $M/(M_E/\lambda)$ is one for the first-storey girder and greater than 1 for the upper girders. One would therefore expect the reduction factors for the column axial loads to be less than $1/2\ \lambda$, that is, less than 2. The results presented in Fig. 6 show that this is true.

The evidence presented so far strongly indicates that if columns have to remain elastic while the girders yield, the reduction factors to be applied to the column moments and axial loads should be considerably less than that applied to the girder moments. This is supported by studies carried out by the author on several tall frames. The results of one such study are presented here. The ten storey frame shown in Fig. 8 was analyzed for the standard ground motion, scaled for a peak ground acceleration of 0.15g, and with 5% damping in the first mode. For elasto-plastic analysis, the strength of each girder was set to be equal to the sum of the end moment due to gravity and one fourth the elastic moment due to earthquake. The ratio of the gravity moment, M_G, to the design earthquake moment, $M_E/4$, was 2.25 for the first storey girder and 4.73 for the top storey girder.

The results of elastic and elasto-plastic analyses are shown in Figs. 9, 10 and 11. The girder ductility ratios lie between 2 and 2.5, which is consistent with the results obtained for the three storey frame, considering that the gravity moments are relatively larger here. The reduction factors for column moments range from 1.7 to about 2.7, while the reduction factors for column axial loads vary from 1.6 to 1.9. Figure 11 shows that the elasto-plastic displacements are smaller than the elastic displacements. The fundamental period for the frame worked out to 2.45 s, and it seems usual for the elasto-plastic deflections to be smaller than the elastic deflections for buildings with periods of this order.

DESIGN BASED ON TIME SERIES RESPONSE ANALYSIS

From the results presented in this paper, it is evident that the correlation between the results of an elastic and an elasto-plastic seismic analysis is only approximate. However, one can be reasonably certain that if the seismic design forces in the members of a frame are obtained by applying appropriate reduction factors to the forces obtained through an elastic analysis, the ductility demand in girders will be of the same order as the girder reduction factor. It is also evident that if inelasticity is to be confined to the girders, the reduction factors to be applied to the column forces should be smaller than the one applied to girder moments. The above factors suggest that, where the dynamic analysis approach to seismic design of buildings is appropriate, the following procedure may be used. The procedure is primarily applicable to rigid jointed steel frames. A limit states design format is followed, and the load factors specified in Section 4.1.4.2 of the National Building Code (NBC 1977) are used.

1. Select a suitable design ground motion. Typically, it would be a motion compatible with the design response spectrum applicable to the site under consideration.

2. Determine approximate member sizes by means of a preliminary design for gravity loads and the lateral forces prescribed in the code for wind and earthquake.

3. Carry out an elastic dynamic analysis of the structure for the design ground motion. The floor mass to be used in the analysis may be equal to 1.25 times the design dead load.

4. Scale down the earthquake forces in the member obtained from step 3 above by applying appropriate reduction factors. The magnitude of reduction factors will depend upon the ductility capacity of the members. At the present stage of knowledge, it seems appropriate to use a reduction factor of 4 for the girder end moments and a reduction factor of 2 for the column moments and column axial loads. The reduced values are referred to as the design earthquake forces.

5. Design the girders so that the ultimate strength of the section multiplied by the understrength factor of 0.9 is equal to the sum of: design earthquake force, 1.25 times the design dead load force, and 1.05 times the design live load force. Design the columns for a combination of the factored dead load forces, the factored live load forces, and the design earthquake forces. If the redesigned building structure is substantially different from the preliminary design, Steps 3 through 5 may have to be repeated.

6. Check the frame, so designed, for gravity and wind loads in accordance with the code. Include a check on the lateral displacements due to wind.

7. For tall buildings, elastic analysis for earthquake ground motion gives rather conservative estimates of the maximum displacements and maximum interstorey displacements. The maximum values of such displacements in the elasto-plastic frame may, in fact, be smaller than those in the elastic frame. Check that the maximum displacements as given by the elastic analysis are within desired limits.

DESIGN EXAMPLE USING TIME-SERIES RESPONSE ANALYSIS

Figure 12 shows a 10-storey setback frame to be designed by the time-series response analysis method for an earthquake motion in the east-west direction. The figure also shows the floor dead and live loads used in the design. Assuming that the building is located in seismic zone No. 3 of NBC 1977, the equivalent static load method gives a design base shear of 117 kips (519.5 kN). A preliminary design of the building is carried out for this base shear, in accordance with the code. Using steel with a yield strength of 36 ksi (250 MPa) member sizes shown in Fig. 12 b and c are obtained. It is assumed that the wind loads are small and do not govern the design.

The building selected for design has a rather large setback. It belongs to one of the several types of structures where a dynamic analysis is preferable to the equivalent static load method. In fact, from studies carried out by the author and reported elsewhere (4), it would be expected that in the event of a major earthquake, the tower portion of the building would be in distress if the equivalent lateral load method of the code were used for the seismic design. A dynamic analysis of the setback building, as designed above, for the standard ground motion and 5% damping in the first mode gives a maximum base shear of 3822 kips (17,000 kN). Assuming, for the purpose of this design, that the base shear value given by the equivalent static load method is reasonable, the scale factor to be applied to the standard ground motion is derived as follows:

$$\text{Scale factor} = (117/3822) \times \text{ductility factor} \times \text{load factor}.$$

Using a ductility factor of 4 and a load factor of 1.05, the scale factor works out to 0.128. In the present example, the design ground motion was taken as 0.15 times the standard ground motion.

The setback building, as designed by the code, was analyzed for the design ground motion referred to above and with 5% damping in the first mode. The results of both an elastic and an elasto-plastic analysis are shown in Fig. 13. The girder and column ductility factors are calculated in accordance with Eq. 1, except in those cases when no plastic hinge forms in a member, when the ductility factor is defined as the ratio of the maximum moment to the plastic moment capacity of the member. The accentuated displacement and interstorey drift responses of the tower are evident from the results. The rotation ductility ratios that develop in the tower beams are rather high. Plastic hinges also form in some of the base columns, particularly in those that are directly below the tower. The advantage of a dynamic analysis for the seismic design of an irregular structure is quite evident. With the help of such an analysis, it is possible to identify sections of the structure that are likely to be in distress. One can then strengthen these sections or, if feasible, alter the structural layout. A simplified method of analysis will not in general be able to predict the exact locations where the structure may need strengthening.

The setback building was redesigned, using the elastic time-series analysis and the force reduction factors described earlier in this paper. The member forces obtained from the elastic analysis with 5% damping were scaled down by applying a reduction factor of 4 to the beam moments and 2 to the column axial loads and column moments. Individual frame members were then redesigned for the combination of factored gravity loads and the design earthquake forces. It was found that members in the tower portion and some columns in the base needed strengthening. Most of these members were redesigned in steel with a yield strength of 44 ksi (300 MPa). Because of the use of higher strength steel the stiffnesses of the members were practically unchanged and the process of analysis and design did not need to be repeated. Details of the redesigned frame are shown in Fig. 14. Members shown by bold lines are those designed in 44 ksi (300 MPa) steel.

Figure 15 shows the results of an elastic and an elasto-plastic analysis of the redesigned building structure for the design ground motion. The ductility ratios of the girders in the tower have been reduced to about three fourths of their earlier values. Also, plastic hinges occur in the columns only at two locations. Hinges are formed at the bases of some of the first storey columns. Plastic hinges also occur at the top of two exterior columns in the base. At these locations, a beam frames into only one column, and because the strength of the framing beam is greater than the strength of the column, the hinge migrates from the beam to the column. If it is considered desirable to avoid these hinges, the column size must be increased so that its strength is greater than that of the framing beam.

SUMMARY AND CONCLUSIONS

It is recognized that a dynamic analysis is essential for the seismic design of buildings that are of irregular layout or have a non uniform distribution of stiffness and mass. Because the building structure is expected to undergo substantial inelastic deformation during a design earthquake, the dynamic analysis should ideally take inelastic material behaviour into account. However, because an inelastic analysis is very complex and expensive, it is rarely employed in design. An elastic analysis is used instead, and the forces obtained from such an analysis are suitably adjusted to account for the effect of ductility and inelastic deformation.

This paper examines the correlation between elastic and elasto-plastic response of single and multi-degree of freedom systems and suggests a method of design that uses an elastic dynamic analysis of the structure. A single storey frame is analyzed first. It is shown that the maximum elasto-plastic deflection can be very different from the maximum elastic deflection. This implies that estimates of ductility demand based on the assumption of equal elastic and elasto-plastic displacements can be inaccurate, sometimes on the unsafe side. The effect of gravity load on the response of the single storey frame is examined next. It is noted that when frame members are designed so that each has a strength equal to the sum of the maximum end moment due to gravity and a fraction of the maximum elastic earthquake moment, the ductility demand is considerably reduced and is closer to the reduction factor used in obtaining the earthquake forces.

Studies of multistorey building frames are presented next. It is shown that, even though the girders are designed for forces obtained by applying a uniform force reduction factor to the corresponding elastic earthquake forces, the ductility demand is not uniform, and is, in some cases, much higher than the reduction factor used. The effect of designing the members for a combination of gravity and earthquake forces is similar to that in the single storey frame: the ductility demands are reduced and are more uniform.

It is further shown that in the multistorey frames, if the columns are designed to remain elastic, the elastic to inelastic force ratio for girders is larger than that for columns. It is concluded that if the design objective is to confine the inelasticity to the girders leaving the columns elastic, the reduction factor to be applied to the elastic forces should be smaller for the columns and larger for the girders. A multistorey steel-frame building with a large setback is then designed on the basis of a time-series elastic analysis. The member design forces are obtained by the application of two different plasticity force reduction factors, a larger one for the girders and a smaller one for the columns. The results of an elasto-plastic analysis of the building as designed are presented to show that the dynamic analysis design procedure used in conjunction with a variable force reduction factor leads to a satisfactory design.

Acknowledgement

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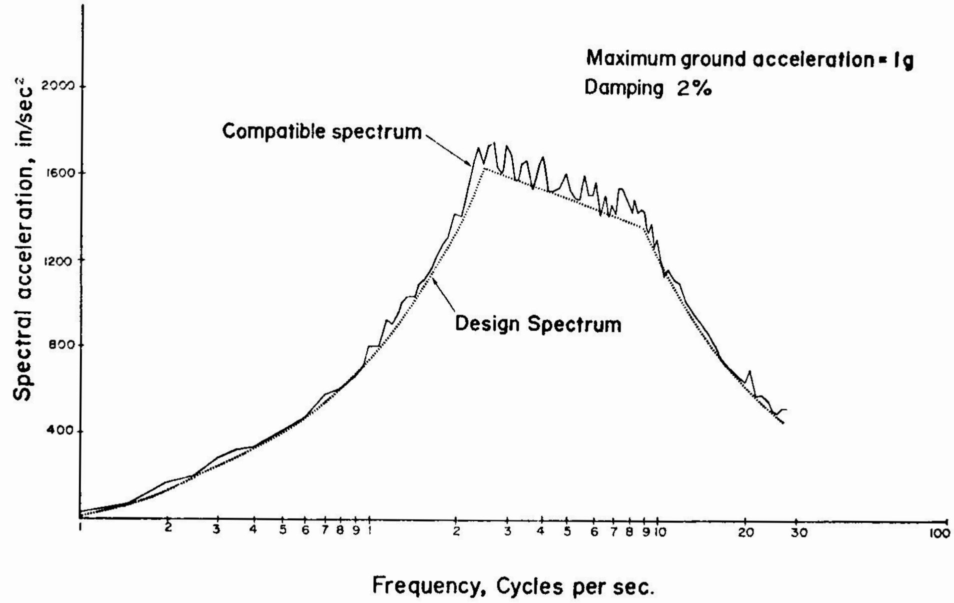


Fig. 1(a)

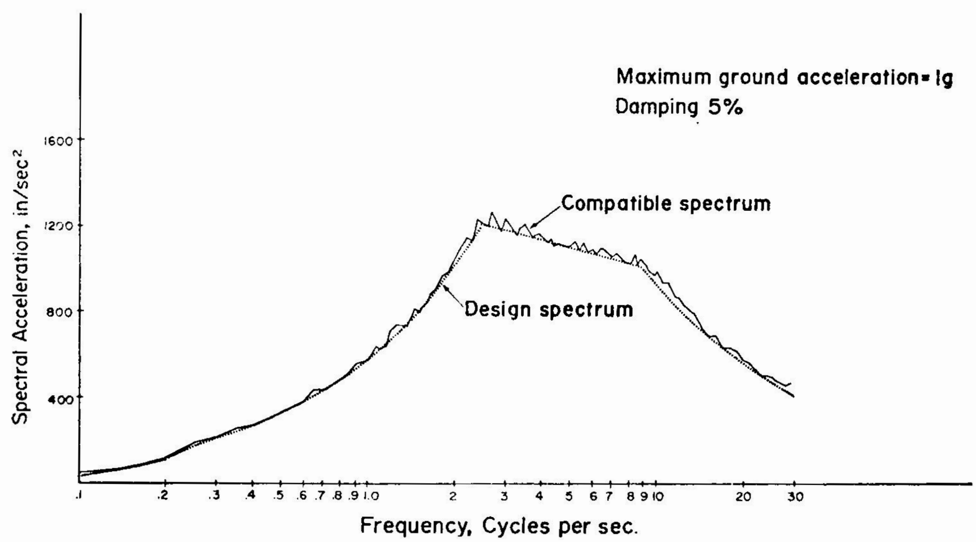
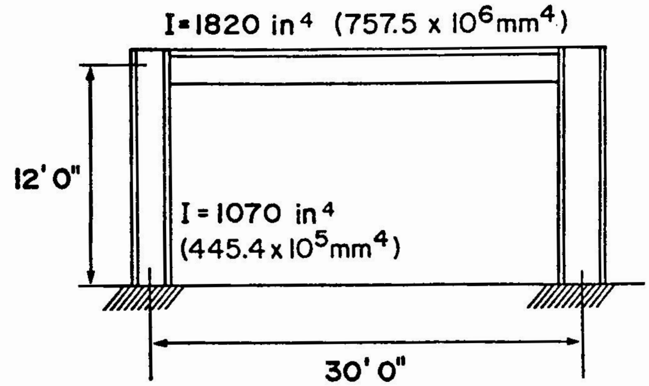
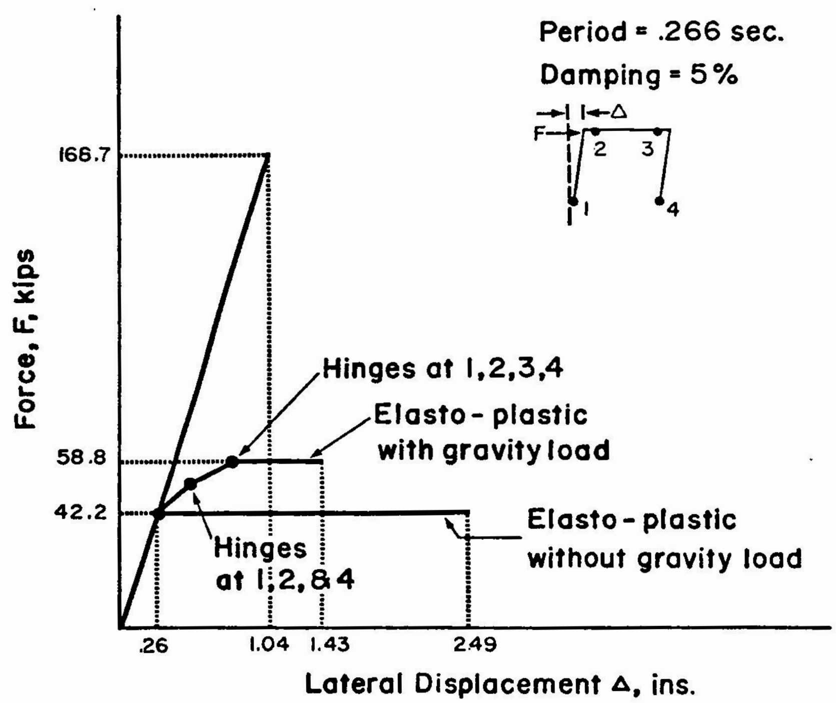


FIG. 1(b) DESIGN SPECTRUM AND ACCELERATION SPECTRUM OF COMPATIBLE MOTION

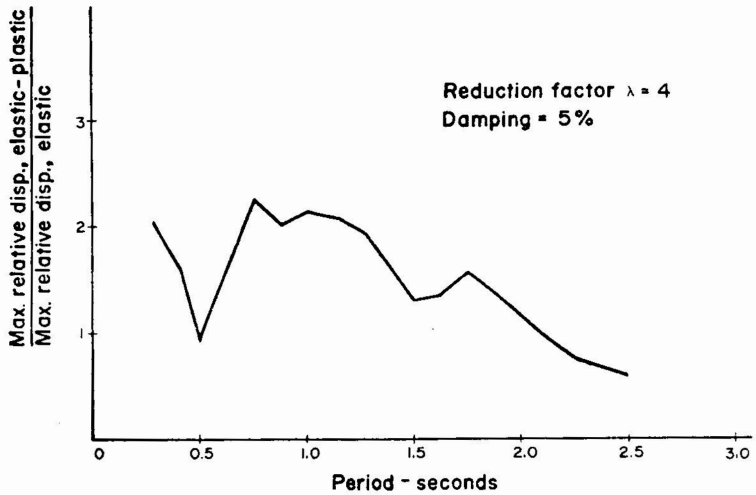


(a) Single Storey Frame

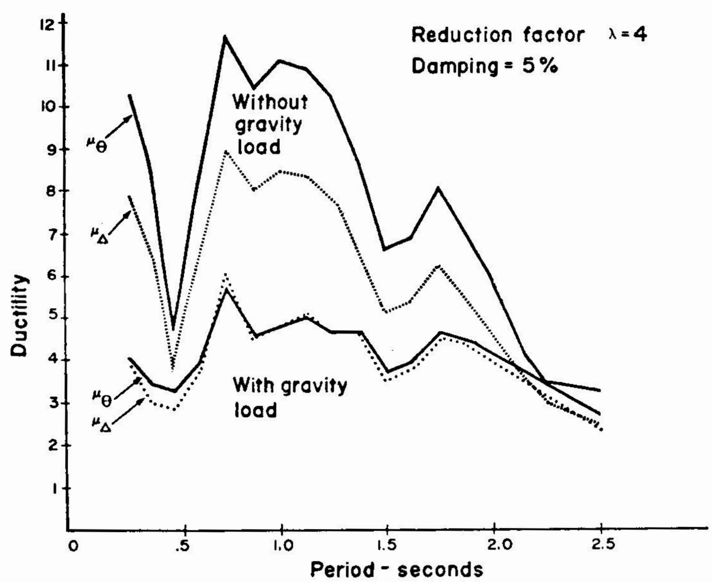


(b) Response of a typical single storey frame

FIG. 2 SEISMIC RESPONSE OF A SINGLE DEGREE OF FREEDOM SYSTEM

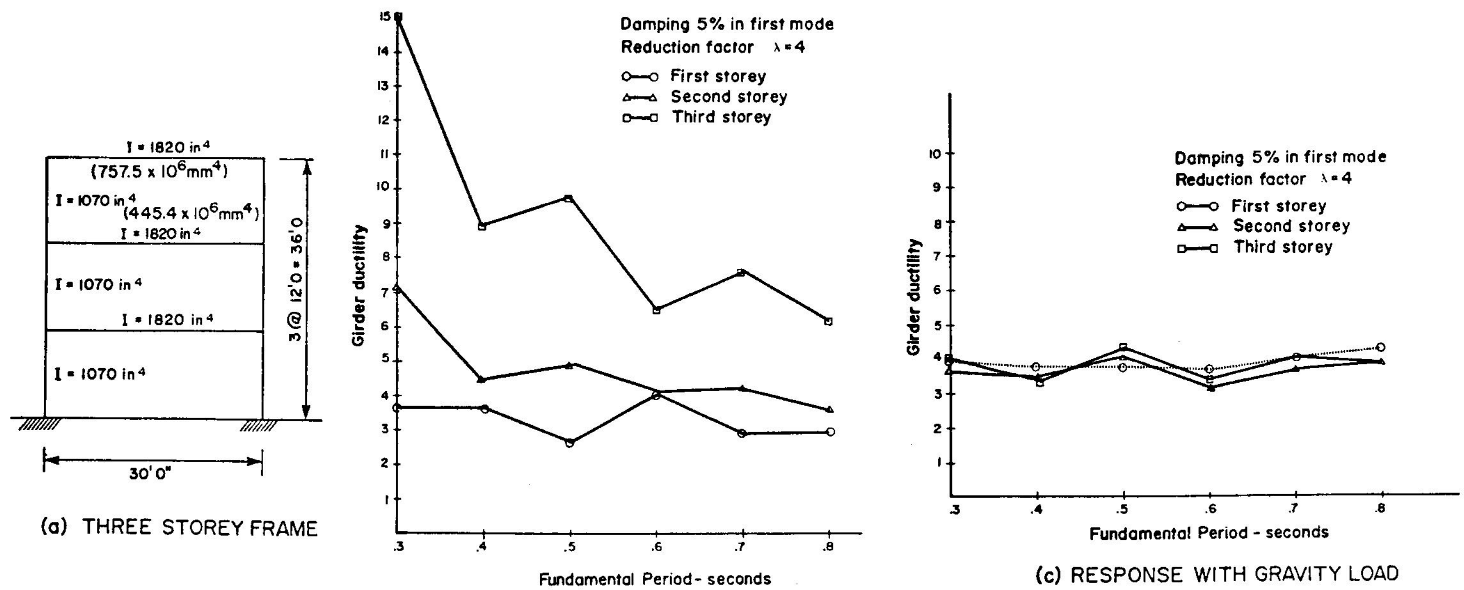


3 (a)



3(b)

FIG. 3 RESPONSE OF SINGLE STOREY FRAME

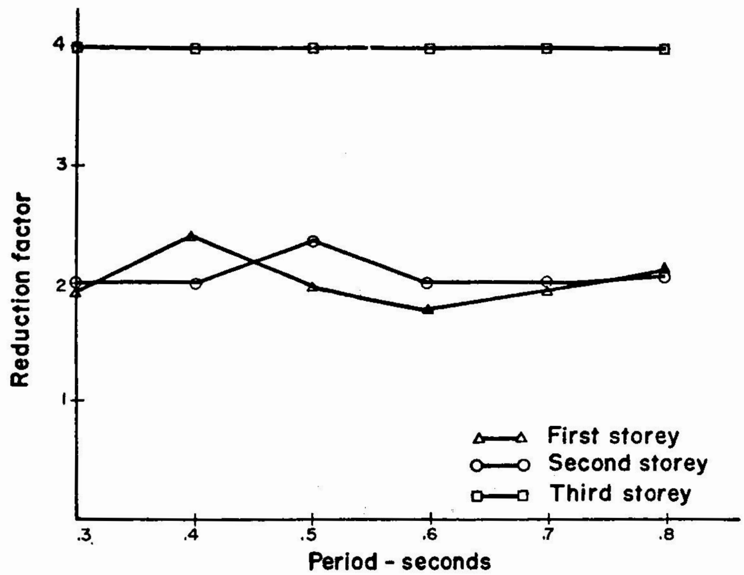


(a) THREE STOREY FRAME

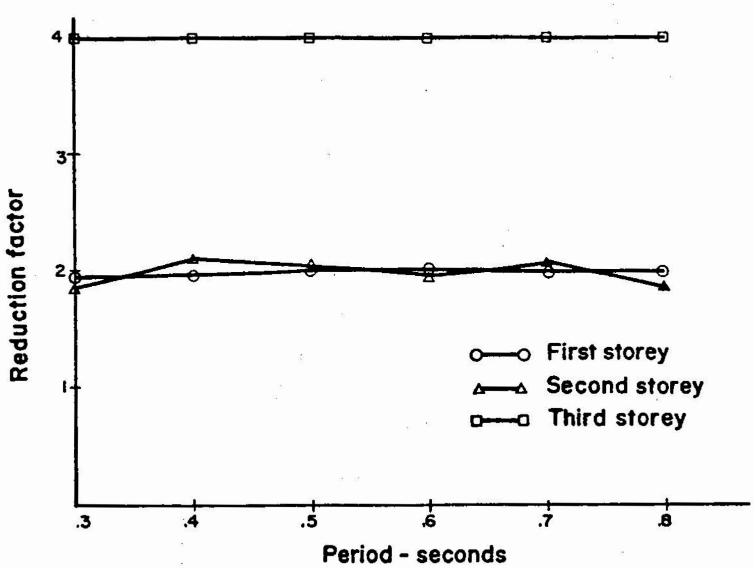
(b) RESPONSE WITHOUT GRAVITY LOAD

(c) RESPONSE WITH GRAVITY LOAD

FIG. 4 RESPONSE OF THREE STOREY FRAME



(a) Response without Gravity Load



(b) Response with Gravity Load

FIG. 5 REDUCTION FACTORS FOR COLUMN MOMENTS IN THREE STOREY FRAME

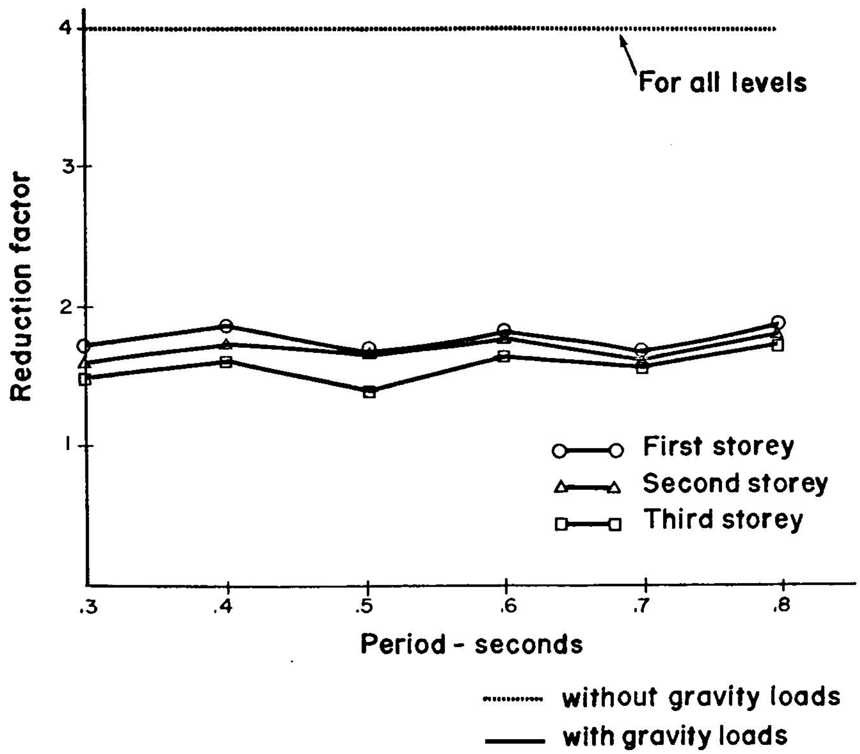
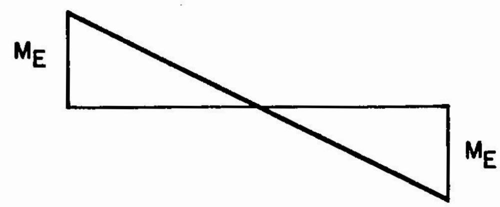
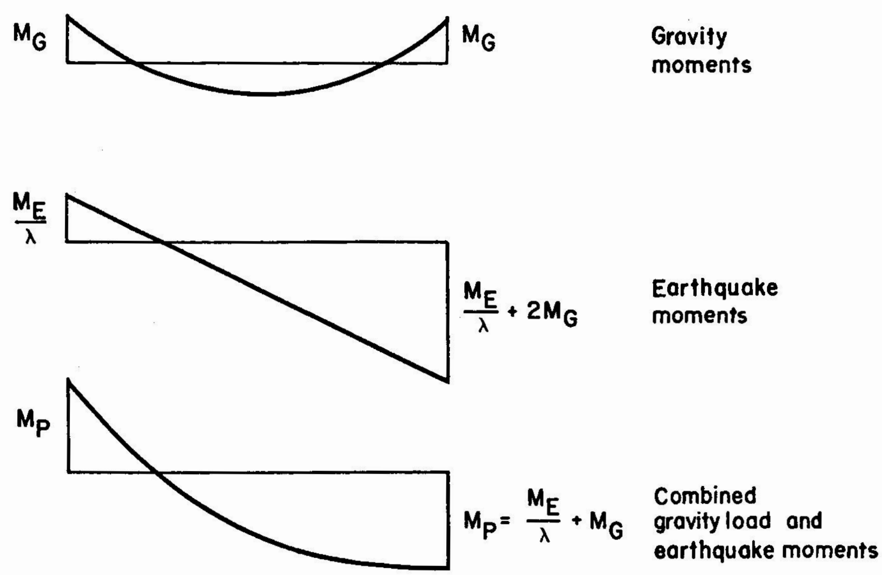


FIG. 6 REDUCTION FACTORS FOR COLUMN AXIAL LOADS IN THREE STOREY FRAME



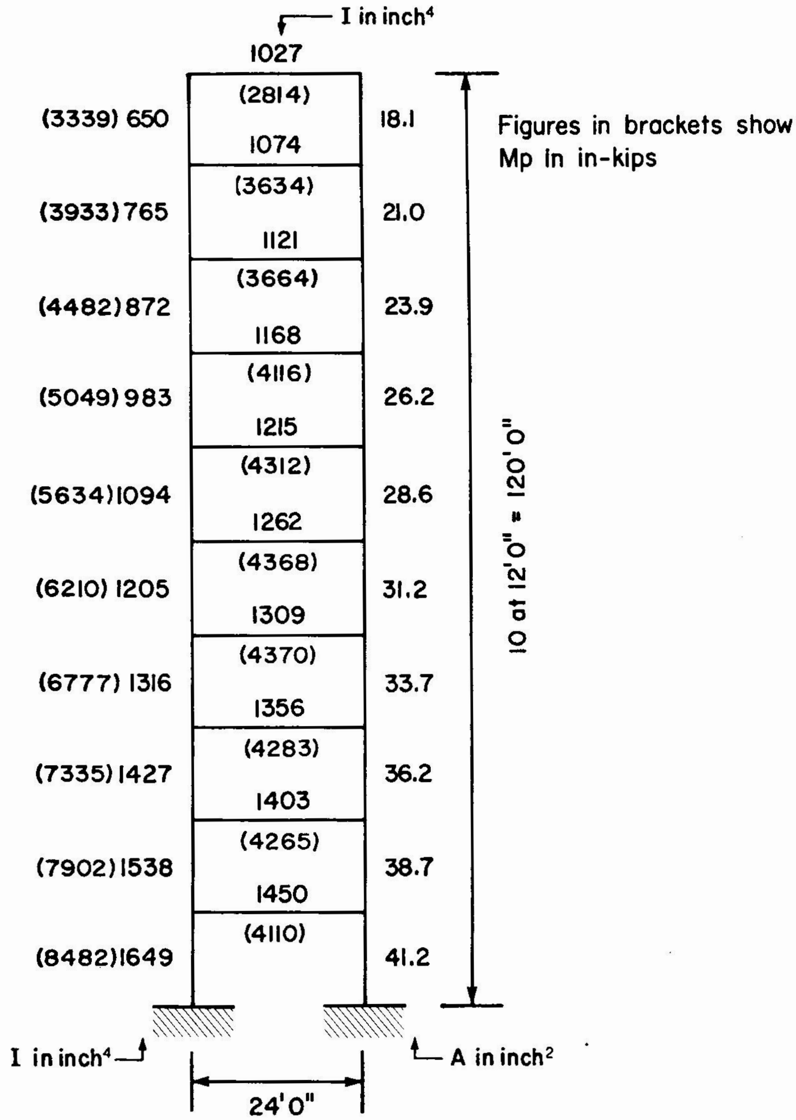
(a) Elastic Frame, Earthquake Moments

(b) Elasto-plastic Frame, Earthquake Moments



(c) Elasto - Plastic Frame, Gravity and Earthquake Moments

FIG. 7 DISTRIBUTION OF MOMENT ALONG THE LENGTH OF GIRDER



Floor mass = .224 KIP Sec.^{2}/in. (39.26 x 10^3 kg.)

Fundamental period = 2.45 Sec.

FIG. 8 TEN STOREY ONE BAY FRAME

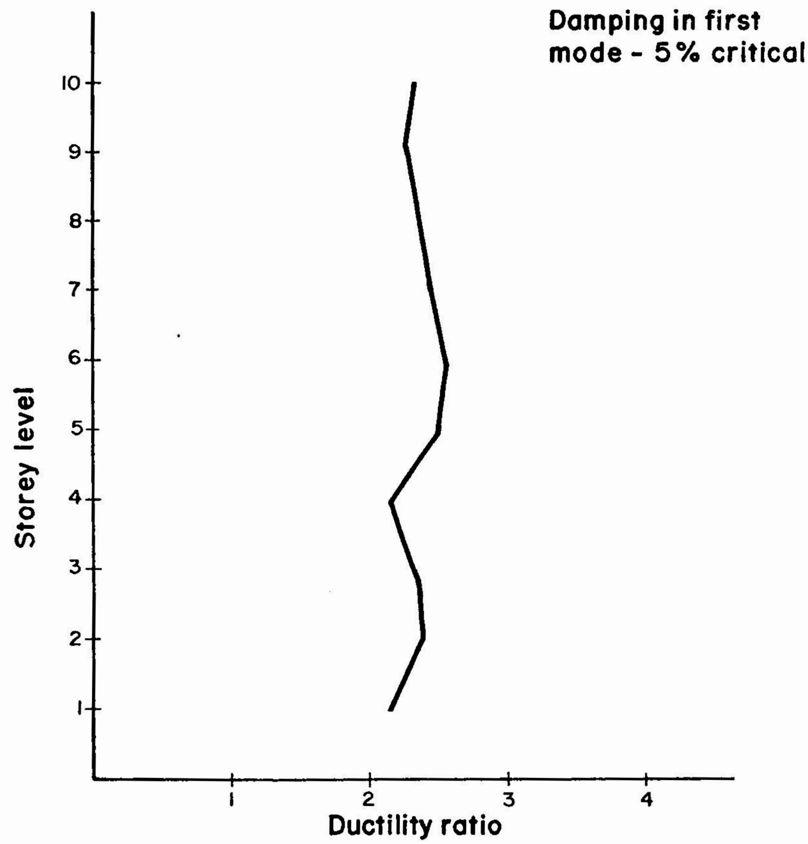


FIG. 9 GIRDER DUCTILITY DEMAND
IN TEN STOREY FRAME

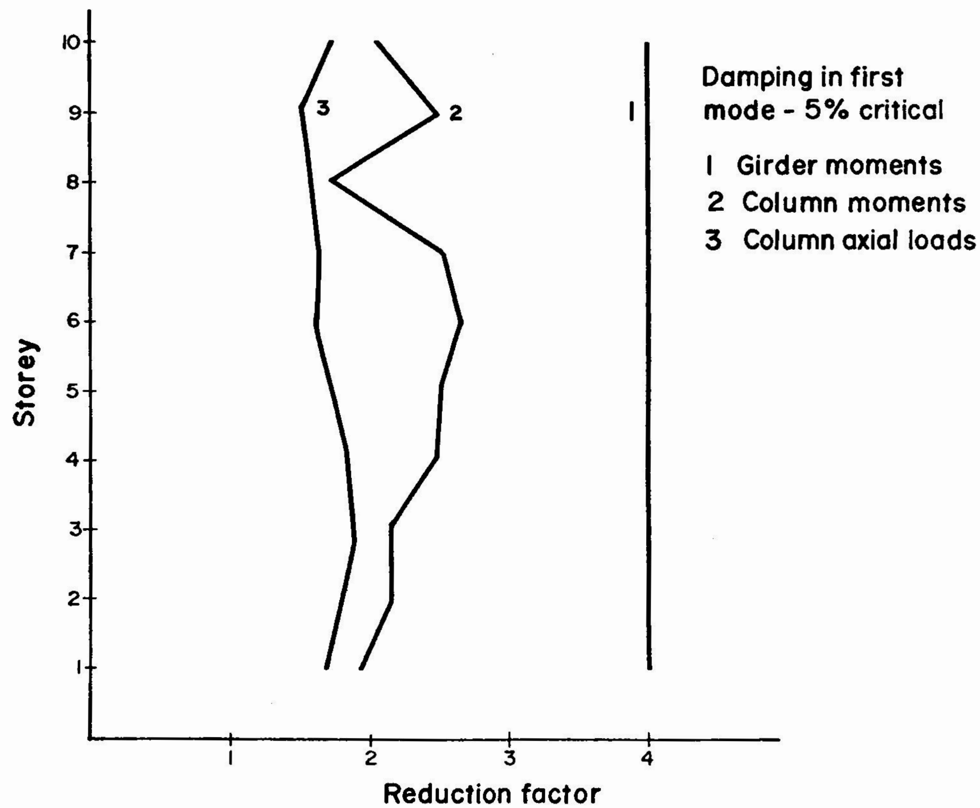


FIG. 10 REDUCTION FACTORS IN TEN STOREY FRAME

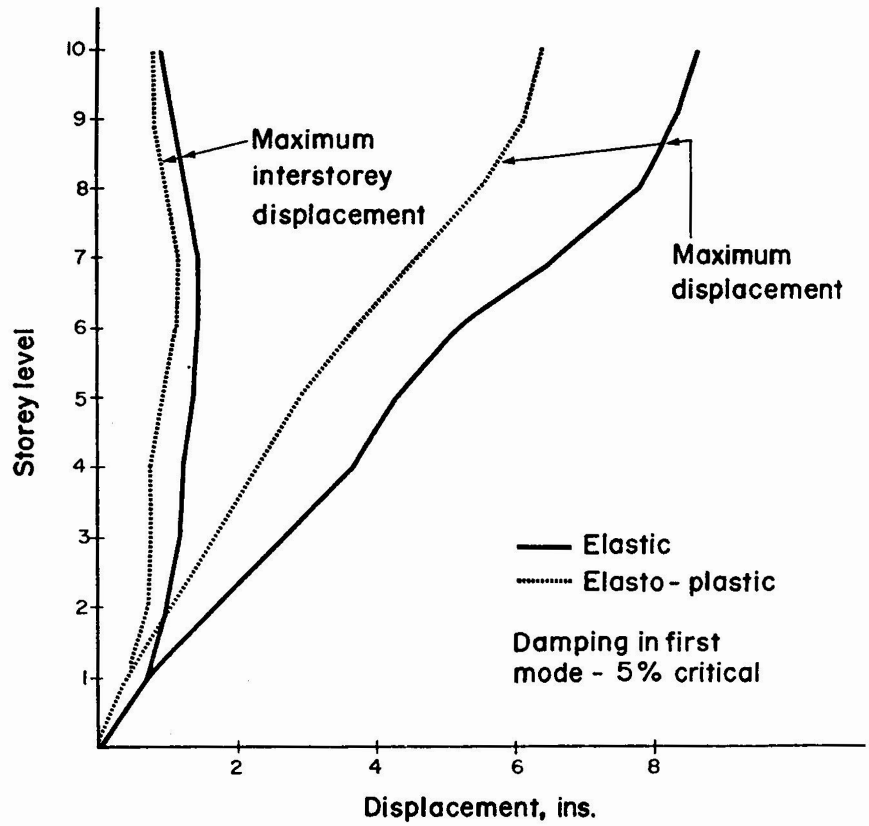
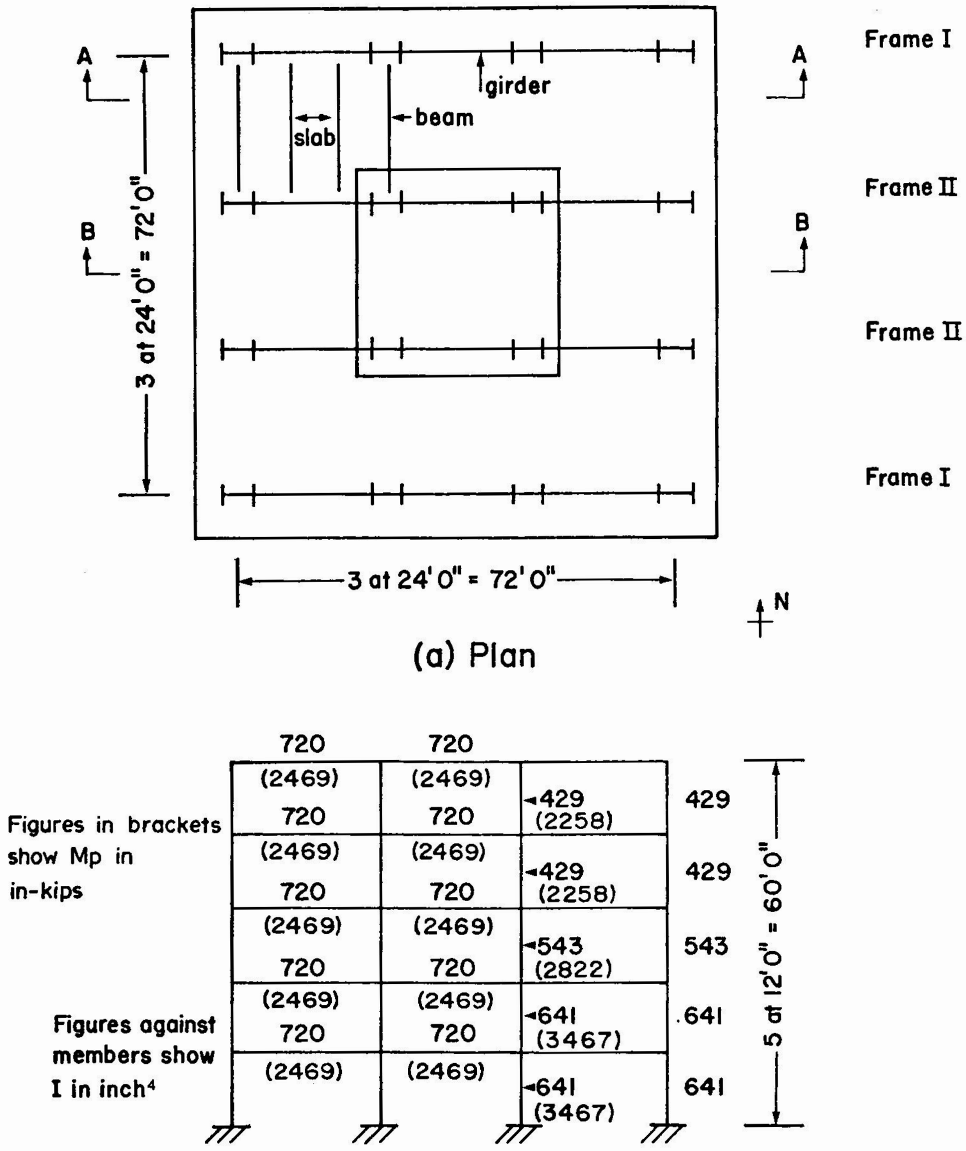


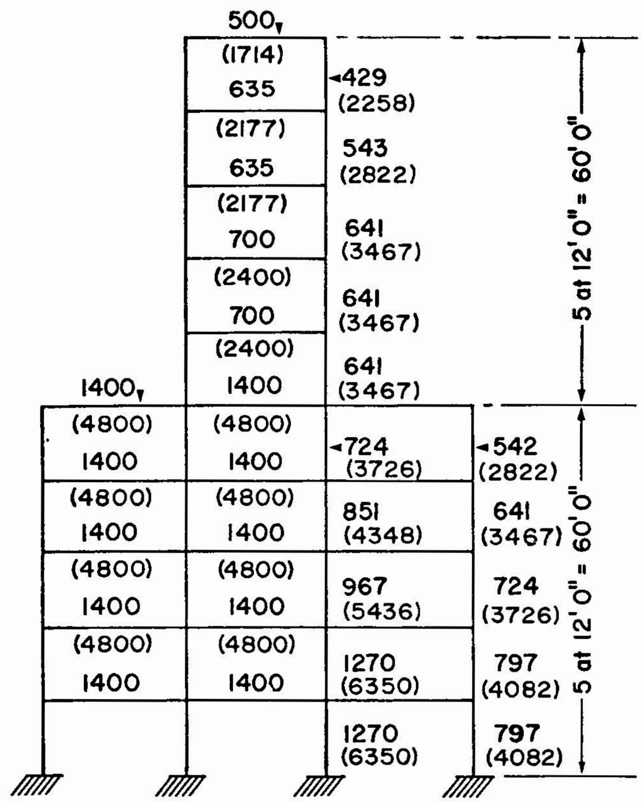
FIG. 11 DISPLACEMENTS IN TEN STOREY FRAME



(b) Section A-A and Preliminary Design of Frame I

FIG. 12 TEN STOREY SET-BACK BUILDING

Figures in brackets show Mp in in-kips
Figures against members show I in inch[4]



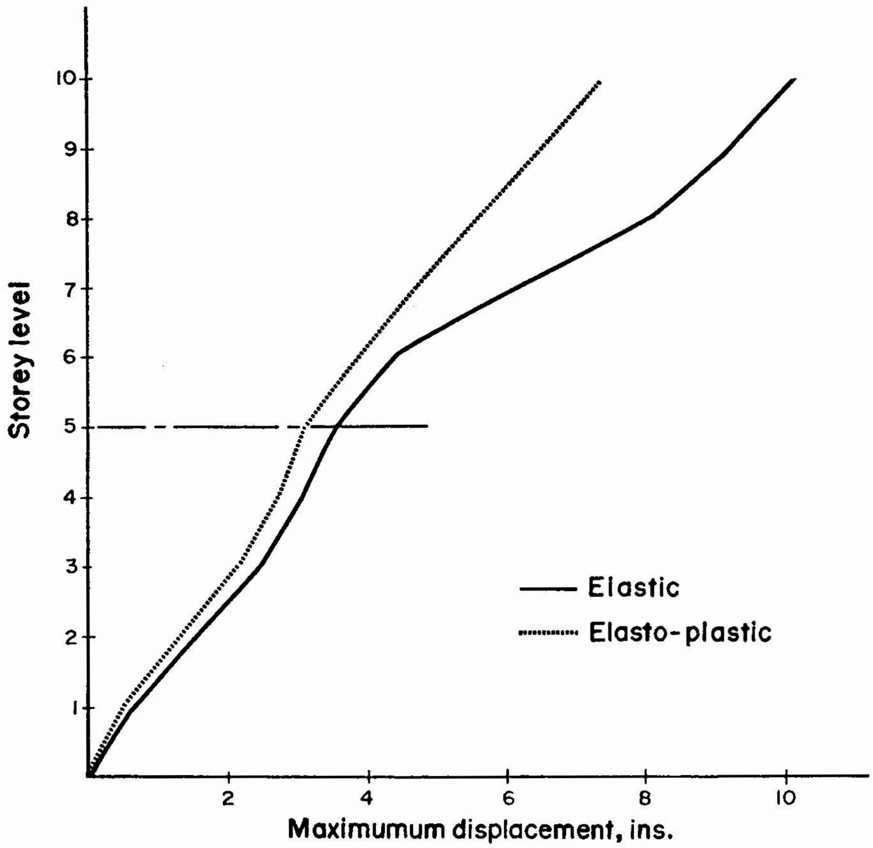
(c) Section B-B and Preliminary Design of Frame II

Design Data -

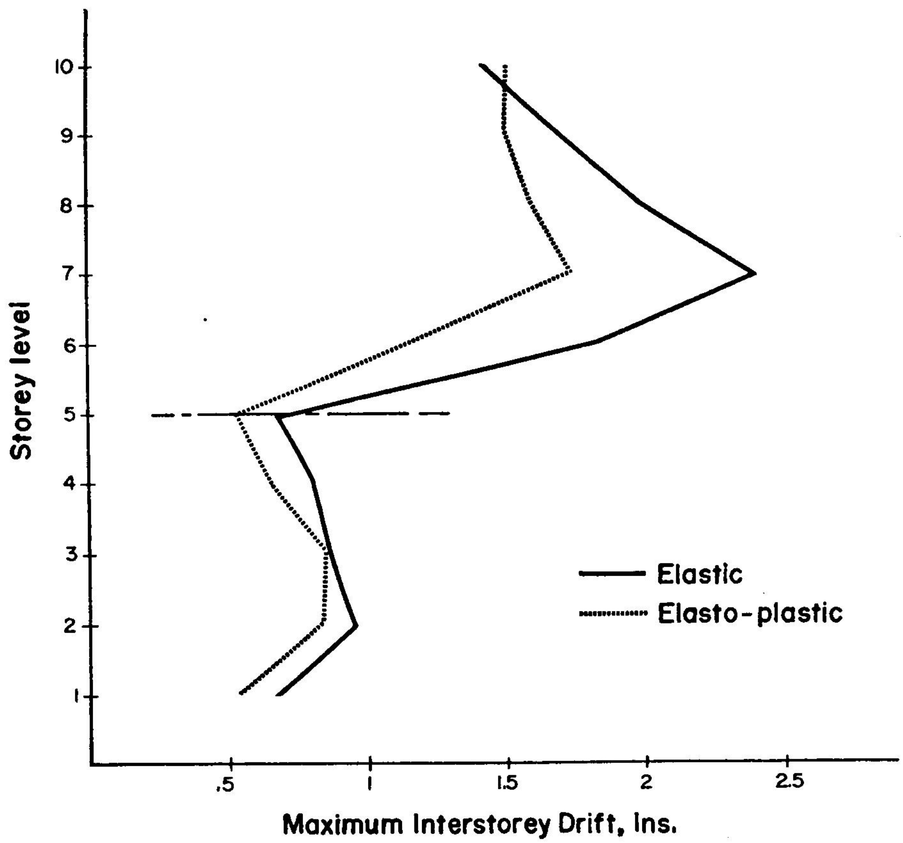
Dead load including allowance for member weights and fixed partitions — 120 psf, all floors

Live load including allowance for movable partitions — 100 psf, all floors

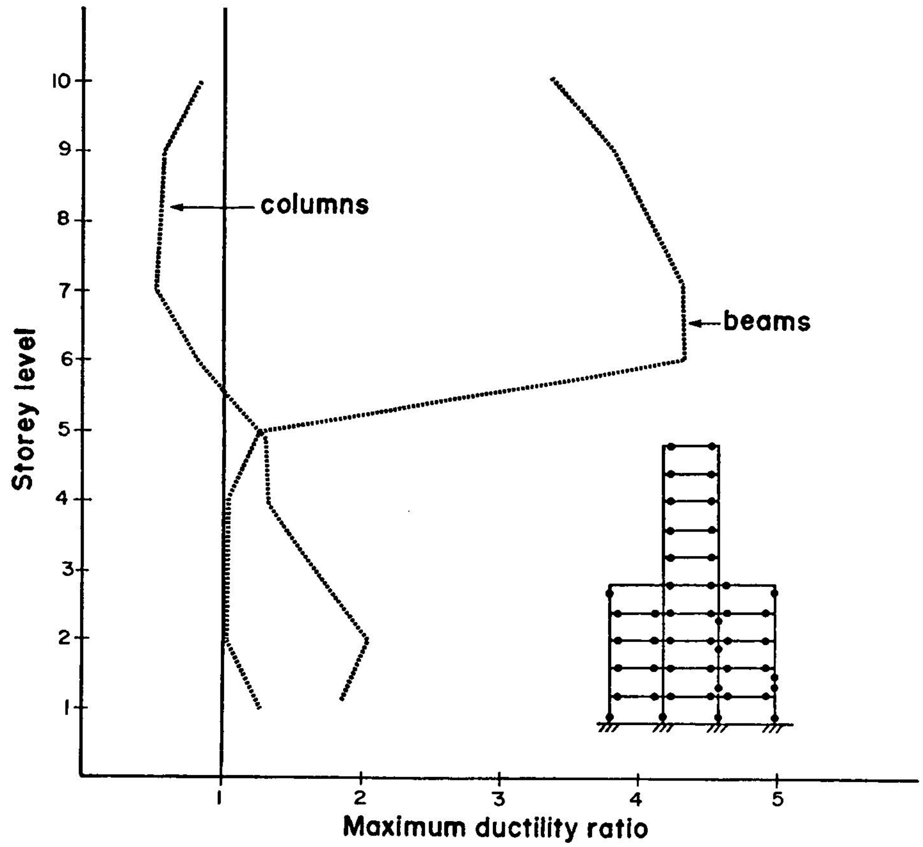
FIG. 12 CONTD.



13(a) MAXIMUM STOREY DISPLACEMENTS



13(b) MAXIMUM INTERSTOREY DRIFTS



(c) Girder and Column Ductility Ratios

FIG. 13 RESPONSE OF TEN STOREY FRAME
SET-BACK BUILDING

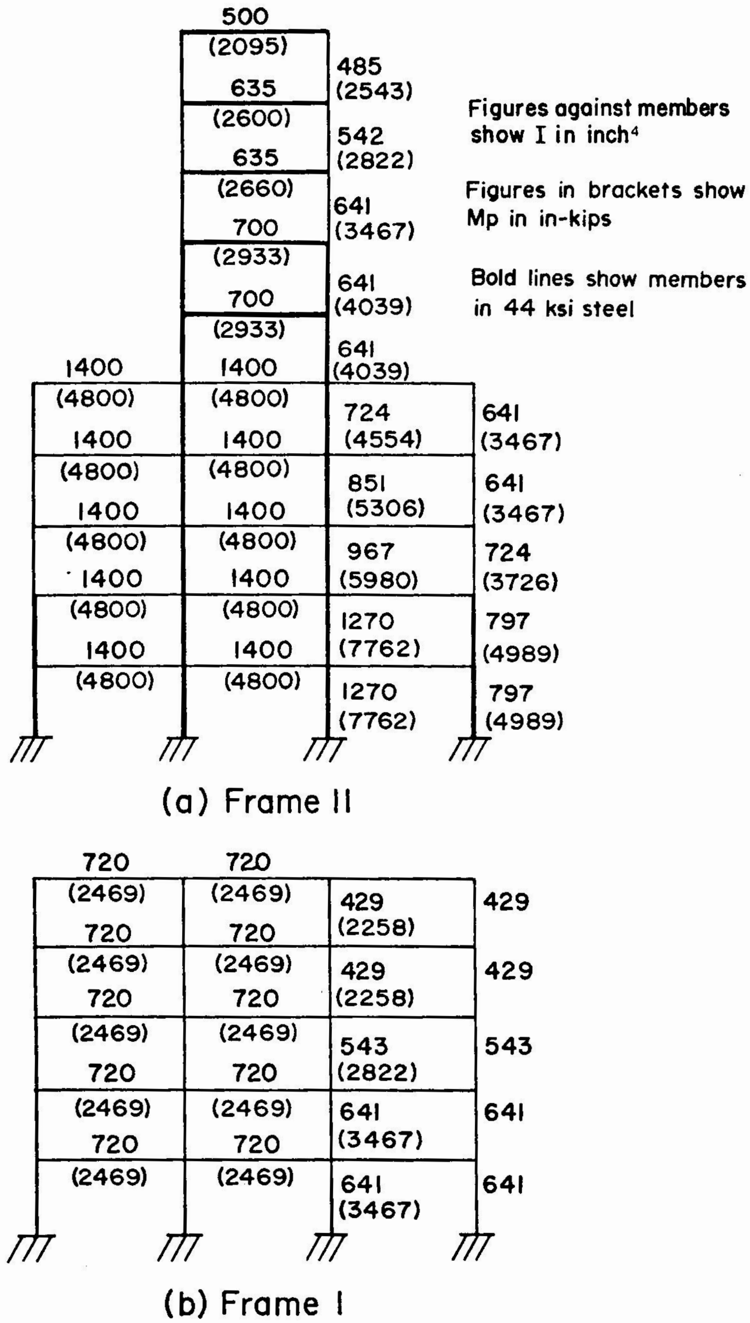
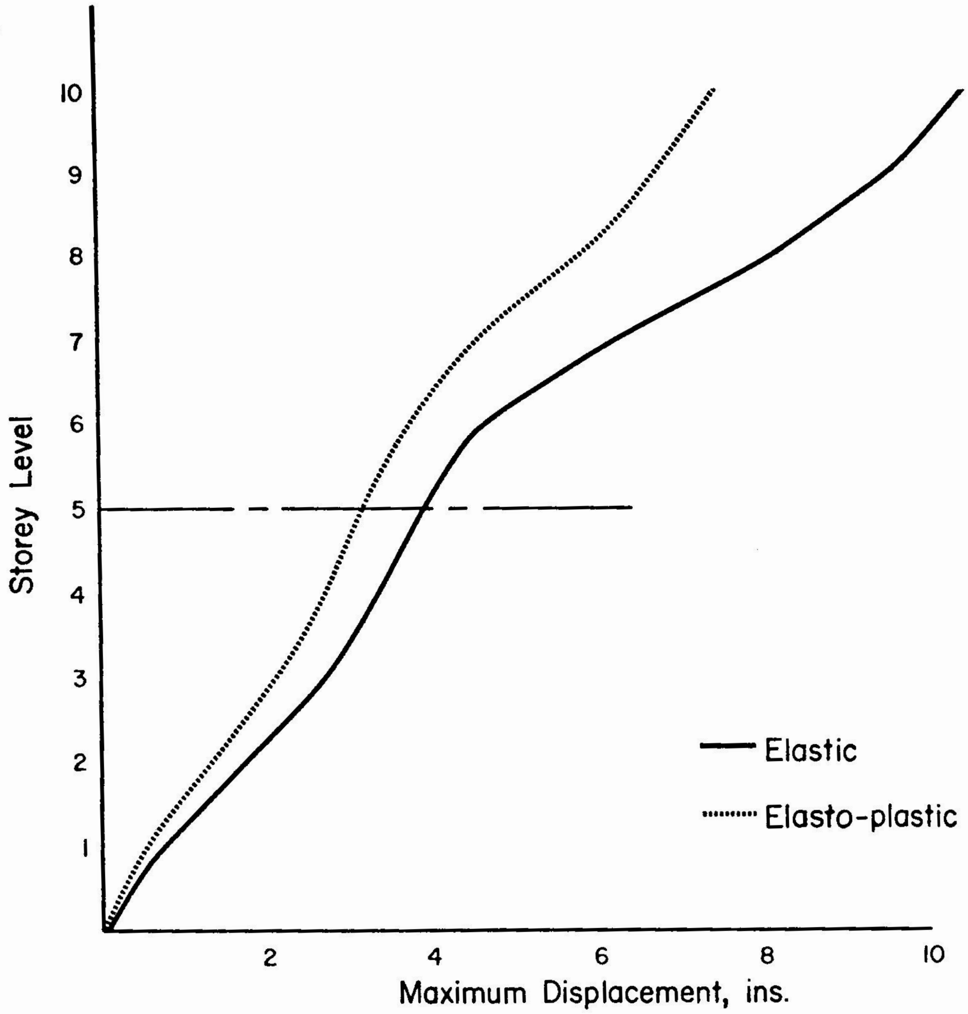
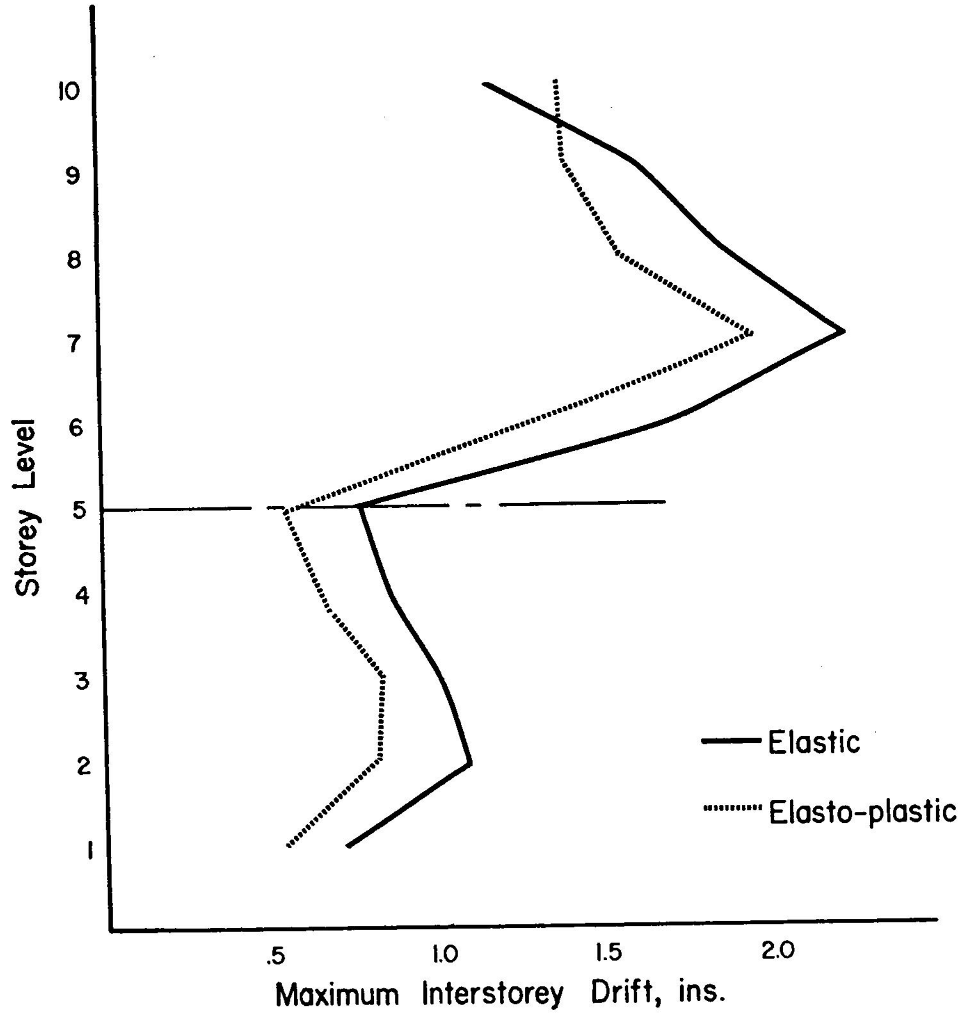


FIG. 14 FINAL DESIGN OF THE TEN STOREY SET-BACK BUILDING



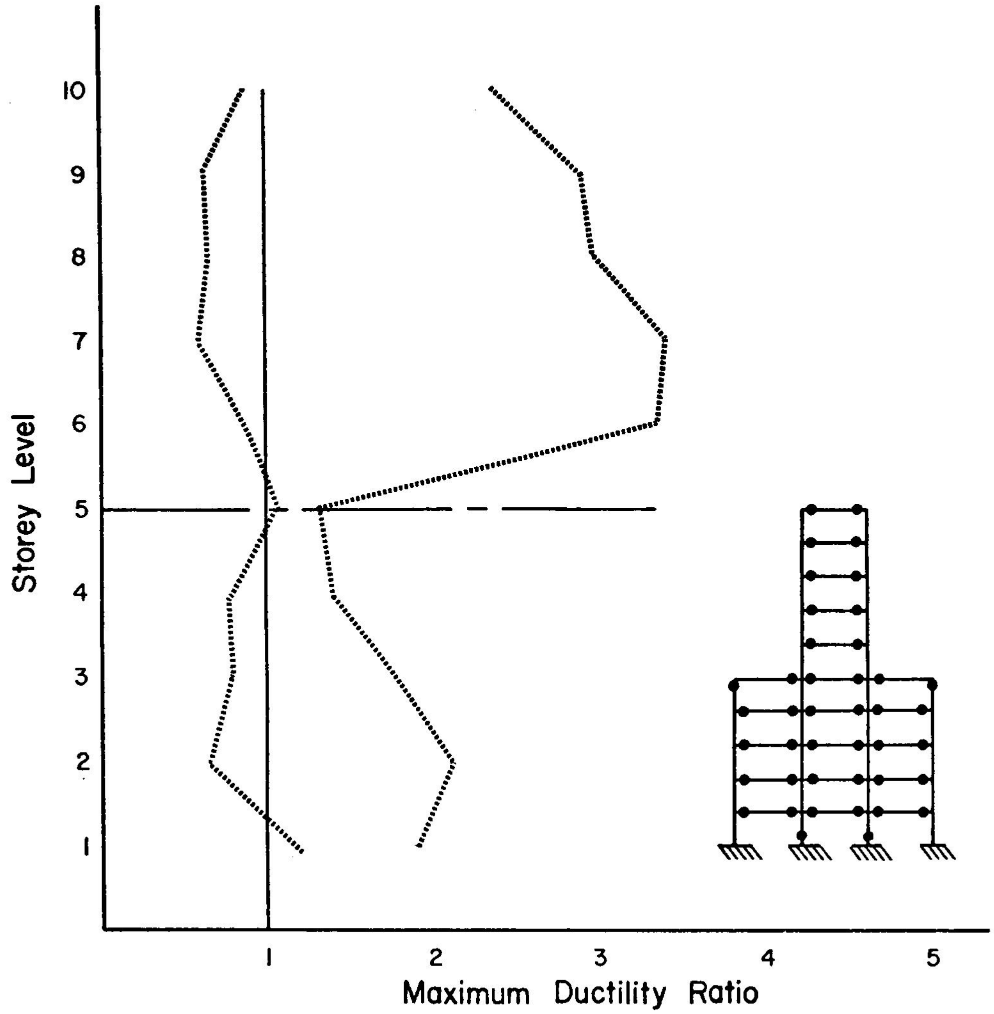
(a) Maximum Storey Displacements

FIG. 15 RESPONSE OF REDESIGNED TEN STOREY SET-BACK FRAME



(b) Maximum Interstorey Drift

FIG. 15 RESPONSE OF REDESIGNED TEN STOREY SET-BACK FRAME



(c) Girder and Column Ductility Ratios

FIG.15 RESPONSE OF REDESIGNED TEN STOREY SET-BACK FRAME