NONLINEAR SEISMIC DESIGN OF MULTISTORY FRAMES

By

V. V. Bertero
Professor of Civil Engineering
University of California
Berkeley, California

and

H. Kamil
Project Engineer, EDAC
480 California Avenue
Palo Alto, California

Department of Civil Engineering
Division of Structural Engineering and
Structural Mechanics
University of California
Berkeley, California 94720

ABSTRACT

A five-step, computer-aided design procedure representing a significant change from current seismic design practices is proposed.

After establishing the "Design Earthquake" and the safety and serviceability criteria, appropriate values of a damping coefficient and displacement ductility factor are assumed. In the first step, an iterative Preliminary Analysis procedure is used to determine the design story shears using modal analysis, and centered around specified values of a seismic coefficient and drift index.

Then, a Preliminary Design is carried out using a simplified story-wise optimization procedure. This is followed by inelastic static and dynamic analyses of the Design. The maximum values of story shears and ductilities and their overall pattern, so obtained, are compared against the ones used initially. The procedure is repeated until a satisfactory agreement is obtained and final design story shears are determined. In the fourth step, these shears are used to attain the Final Optimum Design through a procedure similar to that used in the Preliminary Design, but using an improved story subassemblage and a more formal optimization technique.

Finally, the reliability of the Optimum Design is evaluated by determining its nonlinear response to earthquakes and by its serviceability. The design procedure is demonstrated on a 10-story, 3-bay frame.

1. INTRODUCTION

At present it is generally recognized that there is an urgent need for the development of seismic design procedures which are more rational and reliable than those that are commonly used in practice and which are based on static seismic code forces and linear-elastic procedures. The design and analysis proposed herein is an attempt to satisfy this need.

The ultimate objective of the designer is to have an economical, serviceable and safe building. To achieve this aim, an efficient preliminary design is necessary. Recent progress in computer technology has led to the development of sophisticated and efficient computer programs for the analysis of complex structures. However, use of these computer programs does not necessarily guarantee an efficient design, and this is especially true in the case of aseismic design. If the preliminary design of the structure is poor, repeated analyses of such a design, regardless of how sophisticated the computer programs are, will usually lead to an improved "poor design."

The importance of the design concept used and the need for an efficient preliminary design are things that cannot be overemphasized in aseismic design. For example, the lateral story shear (inertia forces) distribution, transmitted from the bottom to the top, depends on the distribution of the stiffnesses and strengths of the members through the whole structure, and this in turn depends on the preliminary sizes of the members and the design concept used. For example, if the lowest story is weak enough to yield first, the lateral shear transmitted through it upwards will be equal to the yield capacity of this story, regardless of the magnitude of the ground motion. If a preliminary design has these characteristics, so will the final design. Hence, it is important to start with a realistic preliminary design which should be as close as practically possible to the desired final design.

The aseismic design procedure suggested in this paper has been developed recognizing the importance of the overall design concept and the need for a sound preliminary design.

2. PROPOSED DESIGN PROCEDURE

In developing this proposed method, an attempt has been made to achieve the most economical (minimum weight) and practical design which is both serviceable and able to resist without collapse, a possible but not very likely major earthquake shaking. The method has been developed for the design of framed structures of buildings located in regions near active faults. Economic considerations require that for a major earthquake, the structure should be able to absorb and dissipate large amounts of energy through inelastic deformations without collapse, rather than remain elastic. In these cases, usually safety rather than serviceability requirements control the design, and the structure should therefore be designed using inelastic models. In other words, the design should be based on the limit state that actually controls it, and not on a fictitious state as is usually done when loading conditions prescribed in static-type seismic codes are used.

The proposed design method consists of a step-by-step computer-aided design procedure which is basically carried out in five steps: (1) Preliminary Analysis; (2) Preliminary Design: (3) Analysis of Preliminary Design; (4) Optimum Design; and (5) Analysis of the Reliability of Optimum Design.

In the first step, after careful analysis of the data, serviceability and safety requirements are established and the corresponding "Design Earthquakes" are selected. This selection takes the form of smoothed or average linear elastic response spectra for selected damping coefficients. The smoothed linear elastic response spectra for extreme earthquakes are then reduced to take into account the inelastic behavior corresponding to a properly selected pattern of values of ductility. Based on values of periods and modal shapes selected from tabulated values obtained from experimental and analytical investigations already carried out on similar frames, preliminary story shear forces are obtained from a modal superposition analysis. A step-by-step iterative procedure is used to achieve a proper combination of the values for the fundamental period, drift, damping, ductility, story shear forces, and seismic coefficient. When this is achieved, the values so obtained for the story shear forces are the ones considered for the subsequent preliminary design of the structural members.

The preliminary design consists of a story-wise strong column-weak girder limit design using optimization to obtain first the sizes of the girders and then the sections for the columns. This preliminary design can be carried out by hand computations or through the use of a computer program developed for this purpose, and is based on rigid plastic analysis using a single story subassemblage and including the P-Δ effect. Working load drift limitations can be imposed and an approximate cost minimization technique using linear programming can be applied[1].

The static response of the preliminary designed subassemblages and of the whole structure and then the dynamic response of the whole structure are obtained using two nonlinear computer programs[2,1] based on an elasto-plastic moment-curvature relationship with linear strain-hardening. The inelastic rotations, which are assumed to take place at localized plastic hinges, are computed to provide a measure of the plastic rotation demand on the critical regions of the structure. The P-Δ effect and the influence of axial force on the column yielding strength and flexural stiffness are also taken into account. Application of the nonlinear dynamic program permits the evaluation of the response of the preliminary designed structure to different earthquake motion time-histories.

From the outputs of the above two programs, maximum values as well as time-histories of the curvature, rotation, and displacement ductilities are obtained. If these ductility values and their variations agree closely with those preselected, and if the pattern of maximum shear forces obtained from the dynamic analysis as well as that of the shear capacities of each story obtained from the static analysis is close enough to the pattern of story shears used in the preliminary design, the values of this last set of shears are adjusted in accordance with those found from the two analyses, and they are then used for the final optimum design of the frame. If the agreement of one or more of the above parameters is poor, the results obtained in the preliminary analysis and design must be reviewed and modified until satisfactory agreement is achieved.

The final optimum design procedure is similar to that used for the preliminary design except that it uses more sophisticated subassemblages and a more formal linear programming technique.

Finally, the reliability of the optimum design is evaluated by analyzing its behavior under service and ultimate loading conditions. Analyses of the structure to several different earthquake time-histories are carried out using the nonlinear dynamic analysis computer program. Dynamic response analyses of the designed structure to a set of different ground motion time-histories, covering as many characteristics as possible which can be critical to the behavior of the structure, are necessary because of the uncertainties involved in predicting future earthquakes.

The different steps of this proposed procedure are discussed in more detail and its application is illustrated by designing a 10-story, 3-bay unbraced frame. This frame is shown in Fig. 1 and has been selected from Reference 3, where it is referred to as Frame B.

The discussion of the first two steps, viz., Preliminary Analysis and Preliminary Design, are presented under the following general heading of Preliminary Design Procedure.

3. PRELIMINARY DESIGN PROCEDURE

3.1 General Remarks. Any rational structural design procedure normally requires the use of analysis, design, re-analyses, and re-designs. Regardless of the kind of procedure used, after an analysis of the data available, the designer must start with some kind of preliminary or initial design. As pointed out in the Introduction, in the inelastic design of structures subjected to earthquake ground shaking, it is of utmost importance to start with a realistic initial design as close as practically possible to the final design. This design should satisfy the following criteria:

(1) It should have strong columns and weak girders. Furthermore, the columns should be designed with a larger factor of

safety than the girders. This should be done to cover the larger amounts of uncertainties that are involved in the design of columns as well as the probable effects of biaxial shear, bending, and increase in axial forces due to the other horizontal component as well as the vertical component of the ground shaking. Satisfaction of this requirement minimizes the possibility of soft stories (partial sidesway mechanisms).

(2) The weakness in the girders should be uniform at each floor as well as through the whole height of the building. This is required to avoid early yielding at one particular "critical region" of a girder and therefore to reduce the possibility of a considerably high rotation ductility demand in this region. Furthermore, in order for the application of modal superposition and the use of a reduced response spectrum to provide satisfactory results, it is desirable that critical regions of the entire structure yield simultaneously.

3.2 General Outline of Preliminary Design Procedure. The complete Preliminary Design Procedure can be divided into two main phases: (1) Preliminary Analysis, and (2) Preliminary Design. An outline of the different steps involved in these two phases follows:

(1) Preliminary Analysis

(a) Given: 1. Geometry of the Frame; 2. Dead, Live and Wind Loads; 3. Story Masses (obtained directly from dead loads and any live load attached mechanically or by high friction to the permanent mass); and 4. Design Earthquake.

<u>(b) Values Selected on the Basis of Available Information</u>: 1. Acceptable Seismic Coefficient (C); 2. Acceptable Drift Index (R) at service and collapse levels; 3. Standard Values of T_1/T_2, $T_1/T_3 \ldots T_1/T_i$ (from Standard Tables[4]) where T_i = Natural Period for the ith mode; and 4. Standard Values of the mode shapes (from Standard Tables[4]).

<u>(c) Values Assumed</u>: 1. Natural Period for the first mode, T_1; 2. Displacement ductility factor, μ; and 3. Damping coefficient, ξ.

<u>(d) To obtain</u>: The Lateral Story Shears for the given Design Earthquake considering the significant modes.

The Preliminary Analysis Procedure can be further subdivded into two phases:

<u>Phase 1</u>. Selection of proper or desirable values for the seismic coefficient, C, and the drift indices, R, (at service and collapse limit states). Step-by-step iterative procedure for the selection of the proper values for the natural period of the first mode, (T_1), the displacement ductility factor, (μ) and the damping coefficient, (ξ).

In each step of this iterative procedure, the values of C and R (at collapse) computed from the set of values selected for T_1, μ, and ξ are compared with the values that were originally selected as desirable for the seismic coefficient and the drift index. This step is repeated until close agreement between the computed and desirable values is obtained.

Phase 2. Computation of the design lateral story shears (using a modal analysis type of procedure).

Once acceptable values for T_1, μ, and ξ have been obtained in Phase 1, the design story shears are computed using the inelastic response spectrum for the given design earthquake.

(2) Preliminary Design - Basically a step-by-step trial and error procedure.

(a) Given: 1. Lateral Story Shears (obtained from above analysis); 2. Material Mechanical Characteristics (Stress-Strain Relationship); and 3. Load Factors.

(b) To find: 1. Sizes of Girders; and 2. Sizes of Columns.

The steps involved in the Preliminary Design Procedure are:

1. Story-wise strong column-weak girder design, using optimization to obtain the sizes of girders. The lateral story shears obtained in Phase 2 of the preliminary analysis are used for this purpose.

2. Design of columns to minimize the possibility of plastic hinges developing in these elements. The moment capacities of the selected sections for the columns (including the reduction introduced by the presence of axial forces) should be such

that at any joint $\sum(M_p)_{columns} > \sum(M_p)_{girders}$.

Furthermore, the probable elastic distribution of the girder moments at the joint [i.e. $\sum(M)_{girders}$] in each of the columns should be checked to see that they do not exceed their corresponding reduced yielding moment capacities.

3. Estimation of the story shear capacities for the preliminary design.

4. Check for serviceability under dead, live and wind design earthquakes.

5. Determination of the dynamic characteristics (i.e. periods T_i, and mode shapes ϕ_i) of the preliminary design.

6. With the values of T_i and ϕ_i determined in step 5, compute the lateral story shears from the design response spectra using modal analysis.

7. Compare shear values computed in step 6 with those used in the preliminary design and those found in step 3, and decide if it is necessary to carry out a new story-wise preliminary design.

Because of space limitations, rather than discussing in detail the different steps involved in the design procedure, its application will be briefly illustrated by a design example. For a more detailed discussion, see Reference 1.

3.3 Design Example. The frame shown in Fig. 1 is used as an example.

3.3.1 Preliminary Analysis.

(1) Given Data: The geometry of the frame, the design loads and the story masses are given in Fig. 1. The design earthquake is described quantitatively by the response spectrum shown in Fig. 2. This spectrum has been obtained assuming that reasonably expected earthquakes representing the most extreme seismic hazard at the building site will induce a peak horizontal acceleration of 0.5g, a peak velocity of 20.6 in/sec, and a dynamic or transient displacement of 12.5 in.

(2) Step-by-Step Procedure for Selecting Natural Period of the First Mode, Corresponding Displacement Ductility Factor and Damping Ratio:

(a) To Establish Realistically Acceptable Limit Values for Seismic Coefficient (C), Drift Index (R) and Damping Ratio (ξ). The limit values for C should be assigned according to present design and construction experience and economic considerations. An acceptable value of R should be selected on the basis of acceptable damage at service and incipient collage limit states. The acceptable damage level should result from economic considerations of this damage. The value of ξ is generally found to vary little with the natural frequency and seems to depend almost exclusively on the type of structure and the materials used. For this example it was decided that C values up to 0.20 were acceptable, and that R values should not exceed 0.004 at the first yielding or 0.02 at the ultimate (incipient collapse) state.

(b) To Select Values of the Ratios, T_1/T_i, and Values of the Mode Shapes, ϕ_{ij}. Based on results of the analysis of standard buildings of the type used here, and the experimental data available, the following values were selected[4]. It was assumed that only the first three modes would affect the response of the frame:

$$\frac{T_1}{T_2} = 2.7 \qquad \frac{T_1}{T_3} = 4.8$$

ϕ_{1j} = [1.0, 0.94, 0.86, 0.76, 0.64, 0.53, 0.41, 0.29, 0.18, 0.084]

ϕ_{2j} = [-1.00, -0.64, -0.14, 0.31, 0.65, 0.81, 0.82, 0.69, 0.48, 0.23]

ϕ_{3j} = [1.0, 0.096, -0.746, -0.986, -0.626, 0.0065, 0.606, 0.893, 0.813, 0.466].

(c) Assumptions Regarding Values of T_1, μ, and ξ. Based on previous experience with similar structures, the following values were assumed: T_1 = 2.5 sec; μ = 4; and ξ = 5%.

(d) Estimation of Maximum Response of Structure, Considering First Mode. The Smooth Response Spectrum for the linear elastic single-degree-of-freedom system, shown in Fig. 2 has been obtained by multiplying the peak values of the ground motion specified in (1) by the amplifications suggested by Newmark and Hall for 5% damping[5]. To introduce the effect of the accepted inelastic behavior developing a displacement ductility ratio of $\mu = 4$, a reduced spectrum is computed from the linear-elastic response spectrum by using the procedure suggested by Newmark and Hall[5]. The inelastic design spectrum for strength is also shown in Fig. 2. Using the first mode shape the following values are obtained:

Maximum Lateral Displacement

$$Y_1 = \frac{\sum_{i=1}^{10} M_i \phi_i}{\omega_1^2 \sum_{i=1}^{10} M_i \phi_i^2} (PS_a)_{inelastic} = 6.25 \text{ in.}$$

Base Shear

$$V_1 = \frac{\left(\sum_{i=1}^{10} M_i \phi_i\right)^2}{\sum_{i=1}^{10} M_i \phi_i^2} (PS_a)_{inelastic} = 950k$$

$$V_1 = C_1 \times W_{1\ effective} \qquad C_1 = \frac{V_1}{W_{1\ effective}}$$

where

$$W_{1\ effective} = \frac{\left(\sum_{i=1}^{10} M_i \phi_i\right)^2}{\sum_{i=1}^{10} M_i \phi_i^2} g = 1440k .$$

Therefore, $C_1 = \frac{950}{1440} = 0.066 < 0.20.$

According to the computed value for Y_1, and considering the inelastic behavior with a ductility ratio of 4, the maximum total drift is expected to be on the order of

$$Y_1 = \frac{6.25 \text{ in.}}{123 \text{ ft.} \times 12 \text{ in/ft.}} \times \mu = 0.0042 \times 4 = 0.0168 < 0.02.$$

Although the resulting value for C_1 is considerably lower than the acceptable limit of 0.20, if one considers that the effect of higher modes would increase the response, especially C_1, then the values $T_1 = 2.5$ sec., $\mu_\delta = 4$, and $\xi = 5\%$ that were selected initially can be accepted to carry out the preliminary design. As more data become available and experience develops, a better selection for the values of these coefficients can be made.

(e) Estimation of Lateral Story Shears for the Established Design Earthquake and Selected Preliminary Values for Dynamic Characteristics T_1, ξ, and μ Using Significant Modes. According to the values selected in (b), the displacement and base shear modal participation factors, λ_i^y and λ_i^v, respectively, can be estimated using the following formulas[1]:

$$\lambda_i^y = \frac{\sum_{i=1}^{10} M_i \phi_i}{\sum_{i=1}^{10} M_i \phi_i^2} = \frac{L_i^*}{M_i^*} \quad \text{and} \quad \lambda_i^v = \frac{(L_i^*)^2}{M_i^*} = \frac{(\sum M_i \phi_i)^2}{\sum M_i \phi_i^2}$$

From examination of the computed values for these modal participation factors and values of other factors involved in the formulas for estimating the contribution of each mode to the maximum displacement and base shear, i.e.

$$Y_i = \lambda_i^y \times (S_{d\ inelastic})_i = \lambda_i (PS_{a\ inelastic})_i \times \frac{(T_i)^2}{(2\pi)^2}$$

and

$$V_i = \lambda_i^v \times (PS_{a\ inelastic})_i ,$$

it is possible to recognize the number of modes which should be considered. In the design example, only the first three modes were significant.

Once the story shears for each of the significant modes are computed, it is possible to compute the Maximum Probable Story Shears. This computation is carried out by applying the square root of the sums of the square of the modal maximums (SRSSM). By applying this method to the design example, the following values, which are plotted in Fig. 3, have been obtained:

Story	S_1	S_2	S_3	S_4	S_5	S_6	S_7	S_8	S_9	S_{10}
Shear (Kips)	39	77	92	104	118	134	143	151	165	177

Use of Different Story Ductility Values, μ_j. Although in this design example it was assumed that μ_j is constant throughout the height of the building, in general it would be more rational to use different values for the ductility throughout the height. This is because the state of stress in the girders at the upper stories usually permits the development of large ductility and the consequence of large displacements at these upper stories is less detrimental than large story drifts in the bottom stories.

<u>P-Δ Effect</u>. It should be noted that this effect can be taken into consideration directly by estimating the additional story shear due to the P-Δ effect, i.e. $(\Delta S_{P-\Delta})_j = W_j \times \frac{\delta_j}{h_j}$ where

W_j = Effective Weight for the P-Δ Effect at Level j (i.e. the total dead + reduced live load of levels above story level j).

δ_j = Maximum Relative Story-to-Story Deflection at Story Level j. (This value should be estimated considering the expected inelastic displacement response spectra, which depends upon the assumed displacement ductility corresponding to that story level.)

h_j = Story Height at Level j.

Once the additional equivalent shear load $(\Delta S_{P-\Delta})_j$ at each story has been estimated, it can then be added to the corresponding story shear load computed previously as the SRSSMM. Therefore,

$$S'_{j\ max} = S_{j\ max} + (\Delta S_{P-\Delta})_j .$$

<u>3.3.2 Preliminary Design</u>: Story-Wise Optimum Strong Column - Weak Girder Design. The problem and its solution can be summarized as follows:

<u>(1) Given</u>: (a) Gravity and Wind Design Loads (see Fig. 1); (b) Seismic Lateral Story Shears obtained in the preliminary

analysis (see Fig. 3); (c) Safety or Overload Factors. [The values selected were 1.7 (DL + LL) and (DL + 1.4 LL + EQ). Reference 1 gives a detailed discussion on the selection of these values.]; and (d) Mechanical Characteristics of the Structural Material. (Structural Steel A36 was selected and the specified minimum guarantee yielding stress of 36 ksi was used in the design.)

(2) To Find: (a) Sizes of Girders, and (b) Sizes of Columns.

(3) Solution by Story-Wise Optimum Strong Column - Weak Girder Design.

(a) Selection of Story Subassemblage, Assumptions Involved. The story subassemblage used for this preliminary design is shown in Fig. 4. The selection of this type of subassemblage and the constraints imposed by strong column - weak girder design criteria simplify the preliminary design procedure by reducing the number of parameters to the number of girders in that story or less. The use of this subassemblage is justified by the presence of large earthquake lateral loads which force the points of inflection in columns to form very close to midheight except in the top and bottom stories. Also, the use of a strong column - weak girder design attempts to impose formation of plastic hinges in girders only, especially if the design of columns is done for moment, axial and shear force capacities slightly larger than those estimated from the highest strength capacities that the girders can develop. A safety factor of 1.2 is suggested to cover the uncertainties involved in the design of columns, as well as the possible nonequal distribution of girder moments at the joint ($M_{P_{girder}}$) between the columns.

For the design example used in this investigation, it has been assumed that the moment capacities of the girders in the second and third bays are equal. This simplification might be justified by the fact that the spans of the second and third bays are equal. Furthermore, this simplification reduces the design problem to a two-parameter problem which could be solved far more easily by hand using a two-parameter design space for linear programming, instead of using the standard linear programming "Simplex" subroutine.

It should be noted that the support conditions shown in Fig. 4 at the midheight of the columns do not represent the physical condition at these sections of the columns. They were selected to represent in a very conservative way the possible motion that could occur at the selected story level, if the whole structure had been converted in a mechanism as illustrated in Fig. 5.

After analyzing the independent mechanisms for each story subassemblage and formulating the possible effective combined mechanisms, the necessary computations to obtain the constraints for the governing mechanisms were carried out.

(b) Design of Girders - The Optimum Design Procedure. Linear programming optimization techniques are used here to obtain an optimum inelastic design. Linear programming can be used for obtaining an optimum plastic design because plastic design concerns itself only with equations of static equilibium, which are linear in member moment capacities.

The mechanism inequalities, resulting from an analysis of the governing mechanisms, are set up as linear constraints for the optimization problem.

A linear minimum weight objective function in terms of the moment capacities can be obtained as follows:

$$\text{Total weight of structure} = \sum_i \gamma_i \ \ell_i$$

where γ_i = weight/ft. of member i and ℓ_i = length of member i.

For economy W-Sections, tabulated in the AISC Steel Construction Manual, it can be shown that the weight per unit length γ for a certain range of sections can be expressed as a linear function of the moment capacities M_P.

Hence, the Minimum Weight Objective function can be expressed as follows:

$$\text{Total Weight of Structure } W = \sum_i M_{P_i} \ \ell_i = \text{Minimum},$$

where M_{P_i} = Moment Capacity of member i and ℓ_i = Length of member i.

Hence, the linear programming problem for the preliminary design of the story subassemblage shown in Fig. 4 can be set up as follows:

Find M_{P_A} and M_{P_B}

such that:

$$M_{P_A} \geq w_A L_A^2/16 \qquad \text{(i)}$$

$$M_{P_B} \geq w_B L_B^2/16 \qquad \text{(ii)}$$

$$M_{P_A} + 2M_{P_B} \geq (S_x H_x + S_y H_y) /2 \qquad \text{(iii)}$$

$$M_{P_A} + M_{P_B} \geq (S_x H_x + S_y H_y) /4 + w_A L_A^2/8 \qquad \text{(iv)}$$

$$M_{P_A} + 2M_{P_B} \geq (S_x H_x + S_y H_y) /4 + w_A L_A^2/8 + w_B L_B^2/4 \qquad \text{(v)}$$

and $$M_{P_A} L_A + M_{P_B} L_B = \text{Minimum}$$

Some constraint inequalities which are obviously dominated can be deleted.

This linear programming problem, being a two-parameter problem only, (M_{P_A} and M_{P_B}) lends itself to a direct geometric hand solution using a two-parameter design space.

By plotting the constraints (i) to (v) and the objective function in the form of a two-dimensional design space, as shown in Fig. 6, an optimum solution can be obtained immediately. In Fig. 6 the optimum design values, M_{P_A} and M_{P_B} are defined by point

P but in general it would depend on the particular design space and slope of the objective function.

This procedure of story-wise optimization is repeated for each story until an optimum design for all the girders in the frame is obtained.

Selection of W-Sections for Girders from AISC Steel Manual. For all the girders, W-Sections are selected from the AISC Steel Manual having moment capacities equal to or greater than those required in the previous optimization computations. It should be noted that if a chosen section is much larger than the one required, the whole design could be affected because of the relative difference in the sizes chosen for different spans. Hence, it is important to keep in mind that it is convenient to choose as small a size as possible for the larger span (the size could even be smaller than that required) and then to increase the size of the shorter span girder until it becomes equal to the section corresponding to the longer span. This applies to cases where the difference between spans is not very large. It does not apply to cases where one span is much shorter than the others, because in this case the criteria mentioned above may lead to very high shear.

(c) Design of Columns. Columns are now designed in such a way that at any joint in the frame, the plastic hinge(s) will form in the girders only, and not in the columns. This is achieved by choosing column sections in such a way that the sum of the moment capacities of the columns (reduced because of the axial forces in the columns) at a certain joint is greater than a certain safety factor times the sum of the actual moment capacities of the girders. In the computation of the actual moment capacity of a girder, it is better to consider the real yielding of steel and to include the additional strength which results from strain-hardening, i.e.

$$(M_p)_{columns} > F \times \sum (M_p)_{girders}$$

where F = Safety factor $\geq$ 1.0.

In the case of columns with different elastic stiffnesses, it is necessary to check that the distribution of moments at the joint does not lead to a moment greater than its yielding capacity.

Computation of Column Axial Forces. The moment capacities of the columns are reduced to take into consideration the presence of large axial forces. These forces are a result of gravity loads and overturning moments created by the earthquake. Also, it should be kept in mind that the design is being done for two loading conditions, namely (i) gravity loads, and (ii) gravity and earthquake loads.

Axial Forces Due to Gravity Loads Only. For the loading combination of 1.7 (DL + LL), the gravity load axial forces for columns are obtained by using the tributary areas supported by those columns. Starting at the roof level, the axial forces should be computed by taking the tributary area of roof loads supported by a particular column.

Axial Forces Due to Gravity Load plus Earthquake Load. For the gravity load of (DL + 1.4 LL) which is assumed to act simultaneously with the earthquake load, axial forces should be computed in the same way as described above for 1.7 (DL + LL).

Axial Forces Due to Overturning Moments Produced by the Earthquake Load. At each story the overturning moments for each mode are first obtained separately. The maximum probable overturning moment is then obtained by taking the square root of the sum of the square of the modal overturning moments for a particular story, as follows:

$$OTM_{max,i} = \sqrt{OTM_{i1}^2 + OTM_{i2}^2 + \dots OTM_{ij}^2 + \dots}$$

where j = indicator for modes

and i = indicator for stories.

Different approximate methods can be used to obtain the axial forces in columns at a certain story level i induced by the overturning moment. However, an easier and more realistic method might be to compute them by using the controlling mechanism for the corresponding floor.

Computation of Axial Forces in Columns for the Preliminary Design [Fig. 7(b)]. The contribution of each floor to the axial forces in column lines #1, #2, #3 and #4 are

$$\left[\frac{W_{Ai}\,L_A}{2} - 2\,\frac{(M_{P_A})_i}{L_A}\right], \left[\frac{W_{Ai}\,L_A}{2} + \frac{W_{Bi}\,L_B}{2} + 2\,\frac{(M_{P_A})_i}{L_A} - \frac{(M_{P_B})_i}{L_B}\right],$$

$$\left[\frac{W_{Bi}\,L_B}{2} + \frac{W_{Ci}\,L_C}{2} + 2\,\frac{(M_{P_B})_i}{L_B} - \frac{(M_{P_C})_i}{L_C}\right], \left[\frac{W_{Ci}\,L_C}{2} + \frac{(M_{P_C})_i}{L_C}\right]$$

respectively, as is obvious from Fig. 7(b). This is achieved by summing the girder shears at a particular joint. While the above values are the upper bounds for the exterior columns, they are not applicable for the interior columns.

By starting at the roof level, moving down story by story, and summing up the contribution of each floor to the column axial forces, an upper bound on the axial forces in the columns can be obtained. This upper bound can be reached only in the case of a simultaneous formation of all plastic hinges involved in the mechanism controlling the yielding of the whole structure, as shown in Fig. 5. The values obtained are close to the ideal design in which all the plastic hinges (in the whole structure) form simultaneously leading to the complete structural mechanism of Fig. 5.

Selection of Column Sizes. Using the appropriate values of the axial forces in the columns, the column sizes are picked up from Lehigh University Design Aids[3]. Reduced Plastic Moment

Tables are used in such a way that at a particular joint the sum of the reduced moment capacities of the columns should be greater than the sum of the moment capacities of the girders at that joint. The same column size should be used for at least two stories. The possibility of using the same column width for all the stories should also be taken into consideration.

(d) Complete Preliminary Design. The complete preliminary optimum inelastic strong column - weak girder design obtained by applying the above method to the frame of Fig. 1 is shown in Fig. 8.

3.3.3 Some Observations Regarding the Complete Preliminary Design. From the strength point of view, the type of subassemblage used in the above preliminary design procedure must yield a conservative design. The design has been based on the assumption that a sway mechanism forms as soon as sufficient plastic hinges develop in the girders. This is not true in the actual case, because to have a sway mechanism of this type, it is necessary that either plastic hinges form in all the columns of a particular subassemblage, as illustrated in Fig. 9, or a complete mechanism should form, as shown in Fig. 11. Because the column capacities selected are higher than those required by the plastic capacity of the adjoining girders, it is clear that the story shear required to obtain a sway mechanism would be higher than the value used. Also, because of the finite number of girder sizes available, the actual plastic capacity of the section selected for each girder is usually larger than that determined by using the minimum weight design procedure. Improvements can be achieved using 2- or 3-story subassemblages, but this becomes too complex for hand computations. Furthermore, strain-hardening in steel will also lead to higher strength. To have an idea of the actual strength of the designed structure, it is desirable to carry out an analysis for the determination of the static story shear capacities.

4. ANALYSIS OF PRELIMINARY DESIGN

<u>4.1 Computation of Static Shear Capacities for the Preliminary Design</u>. The static story shear capacities of the preliminary design obtained in the previous section were computed using an inelastic static analysis program, BADAS-1[2]. This was done in order to find out under what shear each of the stories would fail. These story shears would then be compared against both the values used in the design as well as the shears obtained by the inelastic dynamic analysis of the frame. The determination of these story capacities gives a general idea of the strengths of different stories and can be used to detect the presence of weaknesses in individual stories.

The static analysis was done by applying a pattern of story forces identical to those used in the preliminary design having a magnitude which was increased proportionately and monotonically until collapse occurred. This was done in two different ways:

first, by using the complete frame as a subassemblage and secondly, by using three-story subassemblages so that the story to be investigated had one story above and one below it. This provided the effect of frame continuity, except of course for the top story, which had only one story below it (two-story subassemblage). The following values were obtained:

STORY		1	2	3	4	5	6	7	8	9	10
Max. Shear (Kips)	Complete Frame	47	93	111	126	143	162	173	183	200	214
	3-story subass.	66	109	121	142	148	169	178	189	217	240

It should be noted that the three-story subassemblage offers higher values of story shears, as would be expected. This difference is particularly noticeable in the upper stories. A discussion of the comparison of these story shears against design story shears and those obtained by inelastic dynamic analysis is given later.

4.2 Check of Preliminary Design for Usefulness at Service States. Besides the usual checks of deflection against gravity loads, the other main check is for drift under service wind loads. Buildings could sway in a manner objectionable to human occupants if they do not have the required stiffness to withstand wind gusts. Also, large relative displacements between two consecutive floor may result in nonstructural damage.

To avoid these effects, it is required that at the working load level, the drift should not exceed the following limits:

$$\Delta \leq 0.0025h \text{ and } \Delta_t \leq 0.0025H$$

where h = Story Height, H = Frame Height,

Δ = Story Drift, and Δ_t = Total Drift

Therefore, an elastic analysis of the preliminary design is carried out to check the drift under wind loads. If it satisfies the above two criteria (two limits), the preliminary design is acceptable from a serviceability point of view.

<u>4.3 Determination of the Dynamic Characteristics of the Preliminary Design</u>. In this step, the periods and shapes of the significant modes of the preliminary design are computed to check their agreement or near agreement with those assumed and used for obtaining the lateral story shears in the preliminary design.

An analysis of the preliminary design, shown in Fig. 8, shows that the fundamental period for the bare frame is T = 2.05 secs. On the other hand, considering the reinforced concrete floor slabs as contributing to the stiffness of the girders the fundamental period is reduced to a value of T = 1.76 secs.

<u>4.4 Comparison of Assumed and Resulting Values of the Dynamic Characteristics</u>. A comparison of the above computed values against the period assumed to obtain the lateral story shears for the preliminary design of the bare frame, T = 2.50 secs., indicates that important differences exist between them; therefore,

additional examination of the significance of these differences is needed before accepting the preliminary design of Fig. 8.

The upper value of T = 2.05 secs. can be used to determine whether deflection at the ultimate state would be acceptable. In this particular case, because T = 2.05 and T = 2.50 secs. are in the range where the spectral displacement is constant (see Fig. 2), the displacement for T = 2.05 secs. is the same as that already estimated for T = 2.50 secs.

The value of T = 1.76 secs should be used to examine whether the expected increase in the inertia forces will not exceed considerably the values assumed for the design. Because the period of the structure is in the range where the spectra velocity (PS_v) or spectral displacement (PS_d) are constants, it is clear that the story shear corresponding to T = 1.76 secs. will be larger than those obtained using T = 2.50 secs.

The new story shears were obtained again using a spectral modal analysis based on the computed dynamic characteristics of the preliminary design including the contribution of the floor slabs, and were found to be as follows:

Story	1	2	3	4	5	6	7	8	9	10
Shear (Kips)	50	105	134	149	158	169	185	207	227	241

As was expected, the story shears were higher than the design story shears for T = 2.50 secs., particularly in the lower stories. For the lowest story, the shear for T = 1.76 secs. was about 27% higher than that for T = 2.50 secs. However, in comparison with the story shear capacities of the preliminary design, the above story shears for T = 1.76 secs., although somewhat higher, were not considered different enough to justify a new design. Therefore, it was decided to obtain the dynamic response of the design by using a time-history of ground motions.

4.5 Inelastic Dynamic Analysis. An inelastic dynamic analysis of the preliminary design (Fig. 8), considering the contribution of the concrete floor slabs to the stiffness of the girders, was performed by using the time-history record of the ground motion for the 1940 El Centro earthquake, N-S Component, with an acceleration level of 0.5g, and the following values of maximum (envelope) story shears were obtained.

Story	1	2	3	4	5	6	7	8	9	10
Shear (Kips)	127	158	158	177	197	219	235	251	270	279

The computer program, MULTY, used for this and other inelastic analyses in this study, was developed specifically for this study. A summary of the theoretical background and numerical algorithms used in this computer program is given in Appendix A. The P-Δ effect was included in the determination of these story shears.

4.6 Comparison of Assumed Design Story Shears, Static Shear Capacities, and Dynamic Story Shears Obtained by Inelastic Analysis.

To facilitate comparison of the different values of the story shears, they are plotted in Fig. 10. As has already been pointed out, the large difference observed between the assumed design shears and the new values computed as spectral mode shears using the values of the periods and modal shapes for the preliminary design ($T = 1.76$ secs.), would appear to indicate the need for a re-design. However, the following comparisons and discussion should show that this is not necessary.

Comparing the design story shears, static story shear capacities, and maximum dynamic story shears obtained by inelastic analysis, it can be observed that the design story shears are considerably lower than the story shears obtained using inelastic dynamic analysis. This was to be expected; in the computation of design shears, the fundamental period was assumed and was later found to be higher than the actual period of the structure while the sections selected were stronger than required. Furthermore, the computation of the design shears did not include the P-Δ and strain-hardening effects.

A comparison of the story shear capacities obtained from inelastic static analysis and story shears obtained using inelastic dynamic analysis, shows that the latter are slightly higher than the former except for the top story where the difference is considerably larger. The main reason for this difference is that for static inelastic analysis a certain fixed loading pattern was assumed, which corresponded to the deflected shape of the first mode of the structure. In the case of the time-history dynamic response, the superposition of several different modes usually results in the cancellation of one or more of the plastic hinges required to form the sway mechanism assumed in the static analysis. Thus a higher story shear, especially at the upper stories, should be obtained by dynamic analysis.

The fact that the static inelastic analysis did not include strain-hardening effects while the dynamic analysis did, further contributed to the observed difference. Also, the approximations involved in the use of subassemblages for the inelastic static analysis could account for part of this difference.

Because the spectral mode shears, computed on the basis of the dynamic characteristics of the significant modes of the preliminary design of Fig. 8, are very close to the static story shear capacities and smaller than the maximum dynamic shears, it seems that the preliminary design of Fig. 8 is reasonable and acceptable, as far as strength and maximum drift are concerned.

4.7 Behavior of the Preliminary Design Obtained by Inelastic Dynamic Analysis Using Time-History of Earthquake Ground Motion.

The maximum curvature girder ductility obtained by inelastic dynamic analysis, was found to be 5.27 with the P-Δ effect, and

4.3 without it. The difference clearly points out the importance of the P-Δ effect on the ductility. The maximum displacement ductility reached by dividing the maximum displacement of each story obtained from inelastic dynamic analysis, by the yield displacement of that story obtained from inelastic static analysis, was found to 3.1, and occurred in the lowest story. It should be recalled that the initial assumed value of the displacement ductility factor was 4.0, which is reasonably close to what was found here for the preliminary design. However, it is recommended that a suite of time-histories of ground motions corresponding to possible earthquakes having peak accelerations of 0.5g, but with a frequency content and pulses which contain sufficient energy over the entire range of critical periods to which the structure can deteriorate due to inelastic deformations, be used before accepting a preliminary design. Considerably different ductility demands usually result from the use of accelerograms having the same peak acceleration but with different pulse characteristics.

By using the inelastic dynamic analysis, the maximum girder plastic hinge rotation was found to be 0.014 rads., a value that can be developed by a compact cross section girder and by the careful detailing of connections. The maximum inelastic displacement was found to be 13.25 in. with the P-Δ effect. The maximum story drifts were also found to be in the allowable range which was set at approximately 0.025 in. Therefore, the preliminary design of Fig. 8 could be accepted. However, because a thorough analysis of the results obtained in the dynamic analysis shows that the preliminary design had certain weaknesses in the intermediate stories, it was decided to re-design the structure by increasing slightly the design story shears at these stories and using a more realistic story subassemblage for the final optimization procedure.

5. FINAL DESIGN AND OPTIMIZATION

5.1 General Remarks. Basically it is possible to apply linear programming to obtain the minimum weight design using the plastic design method for the whole frame, especially by using the constraints imposed by the adopted criterion or philosophy of strong column - weak girder design. However, in cases of multistory frame it would require a great deal of computer time and computer storage. Therefore, it was decided to divide the frame into smaller substructures or subassemblages and to apply the mechanism method of plastic design to each of these subassemblages, starting at the top of the structure.

The subassemblage selected for the final design is shown in Fig. 11. It has been used by El-Hafez and Powell[2], and has six design variables rather than the two for the subassemblage used in the preliminary design, Fig. 4. For each girder, half of its moment capacity is assumed to go to the story subassemblage above the girder and the other half is assumed to go to the story subassemblage below it; when those two subassemblages, above and below, are put back together, the total moment capacity for that girder is recuperated. Hence, the six variables used with this type of subassemblage are equal to half the moment capacity for each girder. Again, a strong column - weak girder type of design philosophy is used. To be consistent with the use of half the moment capacities for each girder, only half the gravity loads are used. The lateral story load applied at the top of each story subassemblage is equal to the shear in the story just above.

The main advantage of using this type of story subassemblage lies in the fact that more design parameters, namely, the moment capacities of all the girders (or rather half the moment capacities of the girders here) are being used now, as opposed to only two design parameters used for the story subassemblage of the preliminary design. This should provide a better distribution of moment capacities throughout a particular story.

Also, the assumption that the points of inflection occur at midpoints of each column used for the preliminary design subassemblage has been eliminated. In other words, this type of story subassemblage is, hopefully, more realistic and accurate than the one used for the preliminary design.

All the possible mechanisms for this type of story subassemblage are considered here, and a standard linear programming (Simplex) subroutine is used to obtain the final optimum design.

5.2 Estimation of Design Story Shears for Final Design. These shears are given below. They were estimated by multiplying the shear used for the preliminary design (based on T = 2.5 secs.) at each story, by the ratio of the corresponding shears obtained for the period of the resulting preliminary design (T = 1.76 secs) with respect to its static story shear capacity, and then normalizing the resulting values to have the same shear in the top story as that for T = 2.5 secs. This method was adopted after analyzing the results compared in Fig. 10. It is believed that the values, so computed, should lead to a more rational design.

Story	1	2	3	4	5	6	7	8	9	10
Shear (Kips)	47	86	107	118	126	135	147	164	180	193

5.3 Formulation of Linear Programming Problem for Final Design. Starting with basic independent mechanisms shown in Fig. 12, double, triple, quadruple, quintuple, and finally sextuple mechanisms are generated systematically. The mechanism inequalities are then written down and together with the following minimum weight objective fundtion derived from Fig. 11,

$$30\ M_{P_A} + 24\ M_{P_B} + 24\ M_{P_C} + 30\ M_{P_D} + 24\ M_{P_E} + 24\ M_{P_F} = \text{Minimum},$$

they constitute a linear programming problem having six design variables (M_{P_A}, M_{P_B}, M_{P_C}, M_{P_D}, M_{P_E}, and M_{P_F}) and additional slack variables, one for each equation. This problem is solved using a Simplex Subroutine (standard linear programming subroutine). The results provide the moment capacities for the girders which satsify the above constraints and at the same time provide minimum weight.

5.4 Selection of W-Sections for Girders from AISC Manual. All the stories, so designed above, are then assembled together to form the whole frame. W-Sections corresponding to the moment capacities of each girder are selected from the AISC Steel Manual in the same way they were for the preliminary design.

5.5 Design of Columns. The procedure for the design of columns is similar to that used for the preliminary design, and follows the method suggested in Reference 3.

5.6 Complete Final Optimum Inelastic Strong Column - Weak Girder Design. This final design is shown in Fig. 13.

6. ANALYSES OF THE RELIABILITY OF THE OPTIMUM DESIGN

Because of the uncertainties regarding the characteristics of future major earthquake ground shakings as well as the actual mechanical behavior of the structure itself, the nature of any aseismic design is nondeterministic. Therefore, it is necessary to subject the optimum design of Fig. 13 to a series of analyses to check its reliability under the possible bounds of the parameters controlling its behavior at service, and especially, at ultimate limit states.

6.1 Elastic Analysis. Because the preliminary design has already satisfied the serviceability requirements under service wind loads, the new design should also satisfy these requirements under slightly larger story shears. The analysis under wind load shows that the maximum story-to-story drift was 0.001h, which is considerably less than the usually allowable wind drift index of 0.0025h.

6.2 Determination of the Dynamic Characteristics. The periods and mode shapes were computed to check them against the assumed values. The fundamental period was found to be 1.67 secs. which compared very well with the value of 1.76 secs. used for evaluating the story shear forces for the final design. Since both the ratios of the periods of the higher modes to the periods of the first mode and the mode shapes were similar to the assumed values, the story shear forces used in the design of the frame shown in Fig. 13 were considered satisfactory.

6.3 Inelastic Dynamic Analysis of the Final Design. Because of the uncertainties regarding the characteristics of future severe ground shaking, the reliability of an aseismic design against a suite of ground motions should be checked. The suite of ground motion time-histories should be selected in such a way that it will test the inelastic response of the structure throughout the probable range of potentially critical periods to which it can respond due to the degradation of its stiffness. Results of the analysis carried out on the following two time-history accelerograms are summarized herein. (1) N.S. El Centro, 1940 and (2) N.21.E. Taft, 1952. The actual recorded accelerations of these two motions were increased so that the peak acceleration of each one becomes 0.5g.

As in the preliminary analysis and design, a damping coefficient of 5% was selected. The mechanical model used was a nonlinear elastic-plastic model with a strain-hardening coefficient of 0.5%. The concrete slab was considered in contributing to the stiffness of the girders.

6.3.1 Maximum Story Shears and Maximum Overturning Moments: The envelope of these maximum forces and moments are given in Figs. 14 and 15, respectively. The story shear values obtained from the dynamic inelastic analysis are compared with those specified by UBC, those used for the design, and those resulting from a dynamic linear-elastic analysis. Figure 14 also gives the values resulting from a nonlinear dynamic analysis using a model which does not include the P-Δ effects. From the analysis of the results presented in this figure, the following observations can be made:

(1) The story shears for inelastic analysis corresponding to El Centro and Taft are similar with a maximum difference of about 8% in lower stories. However, they are considerably higher than the values used in the design. This is believed to be a consequence of the fact that: (a) the limited size of the structural shapes available necessitated using larger sections than those required; (b) the strength of the columns was considered (1.2 to

1.3 being the required capacities); (c) the load patterns assumed in the design did not occur at one time; therefore, the mechanism assumed in the minimum weight design could not develop and the sway or beam-sway mechanism that actually developed at each story required considerably larger lateral shear force at that story. The recognition of this observed difference is not only important for the selection of design forces for the preliminary design but also for the final design and detailing of connections of beams and columns as well as for the design against shear of these members. This recognition is even more important for reinforced concrete than for steel structures.

(2) As expected, the story shears resulting from a dynamic elastic analysis are considerably higher (nearly twice at story 7) than those obtained from the inelastic analysis.

(3) The effect of P-Δ is about 10% in the lower stories.

(4) The design story shears according to UBC provisions are 3 to 4 times smaller than those used in the design of Fig. 13.

(5) The seismic coefficient C corresponding to the maximum base shear (335k) has a value of 0.18 which is economically acceptable.

The analyses of the plots for the overturning moments (Fig. 15) lead to observations similar to those formulated above for the story shears.

<u>6.3.2 Maximum Story Deflections and Maximum Story Drift</u>: The envelope of the maximum values for these parameters obtained from the inelastic analyses are given in Figs. 16 and 17, respectively, where they are compared with the corresponding values obtained from a linear elastic analysis using the Taft sample. From analyses of the results shown in these figures it can be seen that:

(1) The El Centro ground motion resulted in a lateral displacement that for some stories are about 50% higher than those resulting from Taft.

(2) The maximum displacement at the top reached 13.5 in. which gives a total drift ratio of about 0.009, i.e. considerably less than the acceptable upper limit of 0.02.

(3) The elastic analysis resulted in a lateral displacement which in the lower story was up to 30% higher than that obtained from the inelastic analysis under the same ground motion (Taft). However, the elastic displacement obtained for Taft was up to 20% smaller than the corresponding one for El Centro.

(4) The maximum values of drift for each story vary throughout the height of the building. This could not be expected upon examination of the maximum lateral displacement given in Fig. 16. The maximum story drift occurred at story 6, being 0.016 which is lower than the limit value of 0.02.

(5) The story drifts are sensitive to the ground motions.

The difference between values obtained for El Centro and Taft amounts to 25%.

(6) The results obtained from elastic analysis in some stories underestimate the drift values by as much as 25%, and for other stories overestimate them by as much as 37%.

6.3.3 Maximum Curvature Ductility Ratios: The curvature ductility ratio is an important parameter for the detailing of the critical regions of the members.

(1) Girders: The ductility demands for these members obtained from the inelastic analyses under both the 0.5g El Centro and the 0.5g Taft ground motions are compared in Fig. 18. The analysis of the results presented in this figure reveals the following:

(a) Both ground motions show considerable variations in curvature ductility demands throughout the height of the frame. The minimum value of 2.5 at the bottom floors (9 and 10) and the maximum value is 5.5 which is demanded by the girders at floors 1, 5, 6 and 7. Although this ductility demand can be easily supplied by any compact girder section, the wide variation in this ratio reveals that there is some weakness or sharp changes in the girder's plastic resistance along the height.

(b) The curvature ductility ratio appears to be highly sensitive to the type of ground motion used; for example, while the ductility ratio in some girders under El Centro is 50%

greater than that required by Taft (floors 9 and 10), the opposite is true for girders at other floors (3 and 4). This again ponits out the need for analyzing the design under different ground motions before accepting it.

(c) Curvature ductility ratios obtained using the results obtained from an elastic analysis are also shown in Fig. 18. These ratios, which were obtained approximately by dividing the computed maximum elastic girder moments by the corresponding moment capacities are much lower and more uniform throughout the height of the frame, than the values obtained from the inelastic analysis. This points out eh danger of using elastic analysis for estimating the ductility demands. Although the values obtained from the inelastic analyses do not really give the actual curvature ductility demand (because of the two component mechanical models used in this analysis[6]), properly interpreted, they can be used to detail the critical regions of the girder.

(2) Columns: Although the strong column - weak girder design philosophy was adopted, this does not guarantee that the columns will remain elastic during the seismic dynamic response The envelopes plotted in Fig. 19 indicate the following:

(a) Although the only column plastic hinges whose formation was allowed in the design were those at the foundation, some stories' plastic hinges develop at the ends of some columns.

(b) The largest ductility demand is for the exterior column at the top story which amounts to approximately 2.7. Although this is in general a high ductility ratio for columns, the fact that it occurred at the top column where the effect of axial forces on the flexural behavior of the column is very small makes it acceptable[7]. Design can be improved by strenghthening the exterior columns in the two upper stories.

6.3.4 Maximum Plastic Hinge Rotations: For the design of the critical regions of the members, the computed plastic hinge rotation is a better parameter than the computed curvature ductility ratios. The plastic hinge rotation demands as obtained from the inelastic analysis are plotted in the graphs of Figs. 20 and 21.

(1) Girders: From the results shown in Fig. 20, the following observations can be made: (a) Except for the roof exterior girder, the plastic hinge rotation demands in the other girders are equal to or smaller than 0.016 rads., which can be easily developed if compact sections are used; (b) Although the use of a compact section will also permit the top exterior girder to develop a plastic hinge rotation of 0.026 rads., which is the demand shown in Fig. 20, the design can be improved by selecting a stiffer and stronger girder.

(2) Columns: From Fig. 21, it is clear that except for the two upper stories the amount of plastic hinge rotation

developed under both ground motions is very small and therefore acceptable. The plastic hinge rotation demand at the top exterior column is about 0.026 rads. Although this is a high value for a column, the amount of axial force in this particular column is relatively small and as a result, the designed column can provide such a rotation without a decrease in its plastic resistance[7].

6.3.5 Time-Histories of Plastic Hinge Rotations: To be able to determine the kind of inelastic rotation that a plastic hinge undergoes, it is convenient to plot the time-history of its rotation. An example of such a time-history is given in Fig. 22. An analysis of time time-history reveals that in general there was a lack of any significant rotation reversal, which is highly desirable.

From the evaluation of the results obtained for the time-history analyses carried out and presented and discussed above, it can be concluded that the designed frame is acceptable for the two ground motions considered in such analyses. Regarding the sensitivity of the story drift and especially the curvature ductility and plastic hinge rotation for the ground motions used, it is clear that before accepting the design achieved by the method suggested here, it is still necessary to analyze it for a complete suite of expected accelerograms.

7. CONCLUSIONS AND RECOMMENDATIONS

(1) In the seismic design procedure developed in this study all the possible excitations, as well as a large number of possible factors for determining the selection of the design criteria are considered. These include: (a) safety (against collapse); (b) serviceability; (c) ductility requirements according to the maximum lateral deformation (damage control); and (d) economic considerations (minimum weight). In addition, most of the parameters controlling the development of inertia forces at different stages of the design procedure are considered, e.g. fundamental period, damping ratio, acceptable displacement ductility factor, seismic coefficient, allowable and maximum story drift, and geometric nonlinearity effects (P-Δ and beam-column). Taking these factors into account, the suggested seismic design procedure exerts far greater control over the seismic design and is therefore a more rational method than others presently being used.

(2) Application of the proposed procedure to the design example presented here seems to indicate that in general the method works well and can be applied in practice. The preliminary design procedure is complete, self-contained and at the same time, simple. It can be used by consulting engineers in the absence of sophisticated computer hard and soft wares and lacking manpower and time, if used with care and good judgment.

(3) The application of the method also reveals that there is room for improvement. For example, at the start of the preliminary design, values of the fundamental period, damping ratio, acceptable displacement ductility factor, seismic coefficient C_1, and story drift must be selected. At present there is not enough data to make a rational selection of these coefficients. Efforts should be devoted to collecting the required data and to studying the relationship among these factors and parameters. A better evaluation of what constitutes proper load factors is also needed.

(4) The importance of a realistic estimation of the fundamental period of the structure should be emphasized. Because of the uncertainties involved regarding the contribution of the floor system and the distortion of the beam-column joint (panel zone) to the lateral stiffness of the frame, reasonably estimated bounds should be used in analyses.

(5) A comparison of the story shear used for the preliminary design, the static shear capacity of each story and the maximum story shear found from a inelastic dynamic time-history analysis, reveals the following:

(a) The static shear capacity of each story is considerably higher than the corresponding design forces. The main reasons for this are, first, the story subassemblage used assumes that the columns will not offer lateral restraint against the sway beyond that offered by the girders and that their plastic moments are conservatively selected; secondly, because of the finite number of sections available, sections with moment capacities greater than those required are used.

(b) The maximum story shears obtained from nonlinear dynamic time-history analysis are greater than the static story shear capacities. The main reason for this is the fact that the overall sway mechanism assumed in the static analysis either does not occur, or if it does, one or more plastic hinges immediately start unloading and as a result, each story can resist considerably higher shears.

These two conclusions are significant because they indicate: first, that in the trial and error procedure, if a new preliminary design is attempted, the designer should not directly apply the forces that have been obtained from analyses (static or dynamic) of the previous preliminary design. These forces should be reduced. Secondly, the final detailing of the members as well as the design and detailing of their connections, should be done according to the actual capacity of each member and not according to the values resulting from the use of the design forces.

(5) The nonlinear dynamic time-history analyses of two different ground motions reveal that the story drift and especially the curvature ductility and plastic hinge rotations are very sensitive to the ground motion input. This points out the need for analyzing the final design under several possible critical ground motions. Because of the uncertainties regarding the

characteristics of possible future ground motions, there is a need to define what constitutes the critical parameters of a ground shaking for the nonlinear response of the structure, and what are the reasonably expected upper bound values for these parameters. Meanwhile, the structure should be designed with sufficient initial strength and high ductility having stable strength to cover up the uncertainties regarding the critical parameters of ground motions.

(7) Although dynamic linear elastic analyses of the design structure can give an idea of the expected maximum lateral displacement and therefore of the damage in general, this type of analysis is not reliable especially for designing and detailing the critical inelastic regions.

(8) Present knowledge of real mechanical behavior (excitation and mechanical models) should be supplemented by providing the building with high ductility. This can be accomplished with a more careful detailing of the critical regions which control the inelastic response and a more thorough on-site inspection during construction than has been done in the past or is used for standard (non-earthquake resistant) structure. Therefore, one of the main problems confronting the designer is recognizing the critical regions and obtaining guide values for controlling their detailing. While the location of critical regions is not very difficult, determination of the parameters controlling their possible behavior and therefore their detailing is more complex. At present it is

accepted that one of the most important factors in detailing is the so-called required rotation ductility. The writers would like to go further and state that more important than ductility requirements is the need to understand the complete moment-rotation and hysteretic shear force-shear distortion behavior of these regions during probable extreme earthquakes (extreme in intensity, frequency, content and duration).

(9) The computer programs currently used in the nonlinear dynamic analysis of tall buildings consider only stable, bilinear-type hysteretic loops, neglecting the observed degradations in stiffness and in strength with an increase in the number of cycles of reversal deformation beyond yielding. It is recognized that this stiffness and strength degradation can be highly significant, particularly in the response of a reinforced concrete structure to an extreme and long duration earthquake. However, realistic models that account for this degradation are too complex to incorporate into practical computer programs for the analysis of tall buildings. Nevertheless, it should be possible to develop effective models which are both simple enough for practical computations and close enough to reality to permit the detection of the essential features of the actual degradation effects. A computer program which includes the effect of the inelastic shear distortion of the panel zone of the joint has been developed[8]. Because the inelastic behavior of structures (especially concrete

structures) is sensitive to the sequence of excitations, developing the efficient models mentioned above will require the integration of carefully planned experimental and analytical investigations. An iterative approach should be used in which a critical loading sequence is first determined from a dynamic time-history analysis based on a simplified mechanical model. Using this loading sequence in experiments will permit the improvement of the mechanical model to be used in a subsequent dynamic analysis. The new analysis will lead to the selection of a new loading sequence for the experiments, and so on, until close agreement between two consecutive analytical and experimental results is obtained.

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8. APPENDIX A

Methods of Analyses

Several different analytical procedures were used in this investigation, as described below:

(1) Linear Elastic Static and Dynamic Analysis. Linear elastic static analyses were performed mainly to check the serviceability under wind loads. Linear elastic dynamic analyses

were performed to compute the fundamental period of the structure at different stages of the design procedure, and also to compute the elastic dynamic response of the final design to compare its elastic behavior against its inelastic behavior. Standard computer programs, available at the University of California, Berkeley, were used for this purpose.

(2) Nonlinear Inelastic Static Analyses. Nonlinear inelastic static analyses were performed, mainly at the preliminary design stage, to compute the story shear capacities of the frame. A computer program, BADAS-1, was used for this purpose. This computer program is described in detail in Reference 2 and need not be discussed here.

(3) Nonlinear Inelastic Dynamic Analyses. These analyses were performed at the preliminary design stage as well as the final design stage, and formed an integral part of this investigation. A special purpose, efficient, and refined computer program, MULTY, was developed for this purpose[1]. The salient features of this computer program are briefly described below.

(a) The whole method of analysis is based on the concept of a yielding surface, especially in checking the formation, loading, and unloading of plastic hinges. The effect of axial forces on the moment capacities of the columns is included. Yield surfaces for steel as well as for reinforced concrete can be used.

(b) Overshoot corrections are made whenever the moment at the end of a member exceeds the yield moment.

(c) Geometric nonlinearity is included in the program in a simplified way.

(d) The structure stiffness matrix is assembled only when the status of a member anywhere in the structure changes from elastic to inelastic or vice versa.

(e) The plastic hinge rotations as well as curvature ductilities are computed at the two ends of each member for each time step.

(f) A mass and stiffness dependent damping matrix is used.

(g) An improved and efficient procedure for the solution of dynamic equilibrium equations is used.

(h) The ends of all the members are assumed to follow a two-component bilinear hysteretic loop.

(i) The program uses some very efficient numerical algorithms and takes far less computer time and core storage than several other programs.

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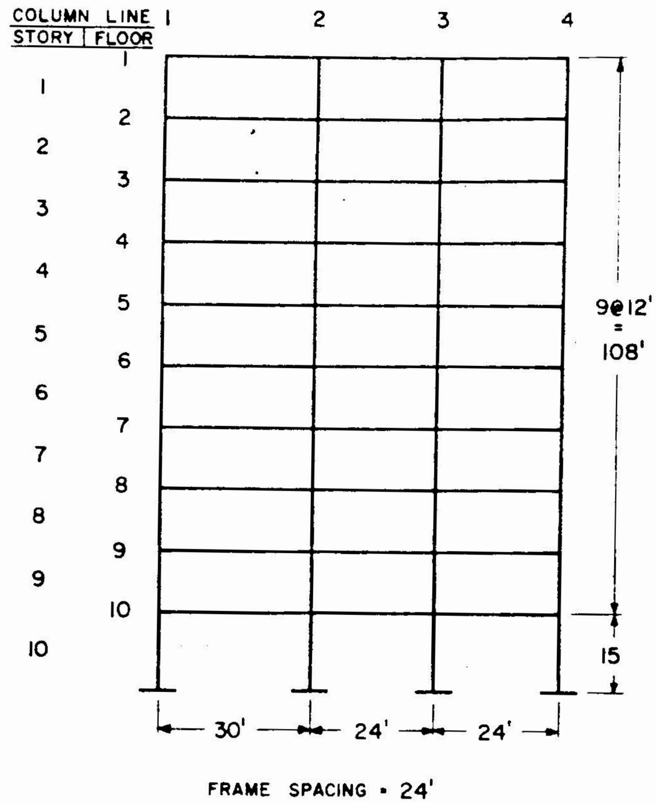


DESIGN LOADS AND STORY MASSES

WORKING LOADS

DEAD
- Roof..............60psf
- Other Floors........80psf
- Exterior Walls......45psf

LIVE
- Roof..............30psf
- Other Floors........80psf

WIND 20psf

STORY MASSES

Roof...............4.0 $\frac{\text{K-sec}^2}{\text{ft.}}$

Other Floors........6.0 $\frac{\text{K-sec}^2}{\text{ft.}}$

FIG. 1 – DESIGN EXAMPLE [3]

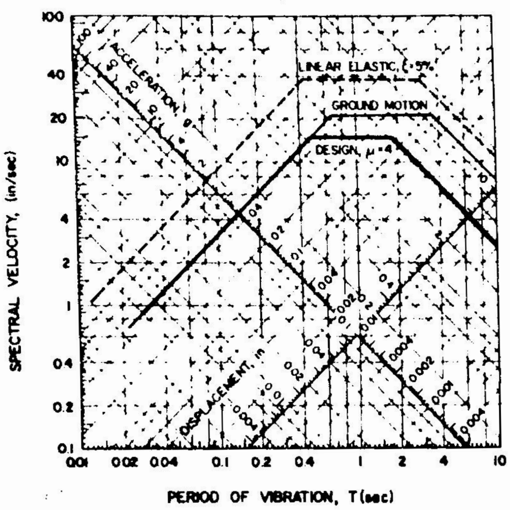


FIG 2 DESIGN RESPONSE SPECTRUM

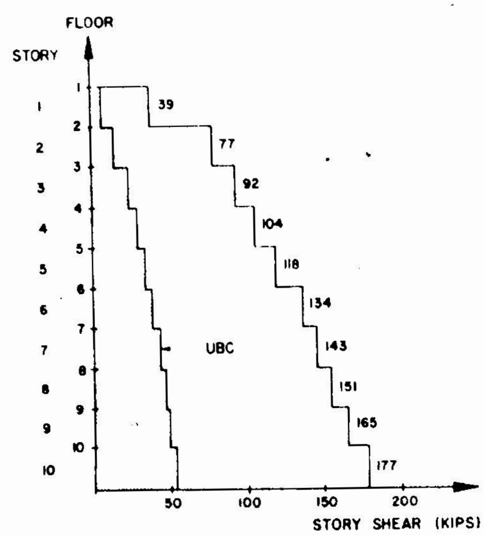


FIG 3 STORY SHEAR FOR PRELIMINARY DESIGN

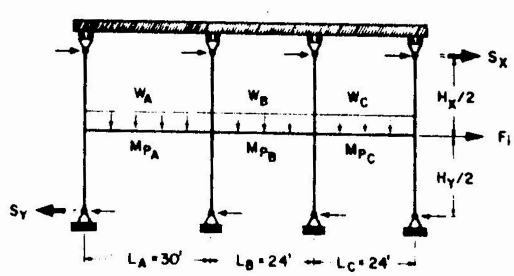


FIG. 4 SUBASSEMBLAGE FOR PRELIMINARY DESIGN

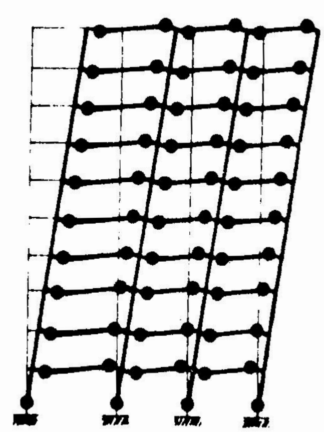


FIG. 5 COMPLETE SWAY MECHANISM

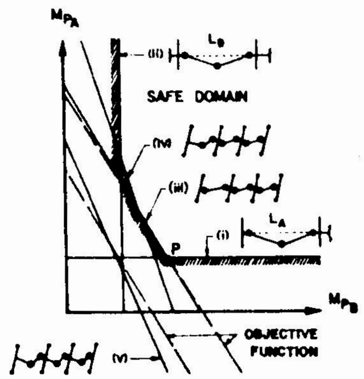


FIG. 6 TWO DIMENSIONAL DESIGN SPACE FOR PRELIMINARY DESIGN (FOR A SAMPLE INTERMEDIATE STORY)

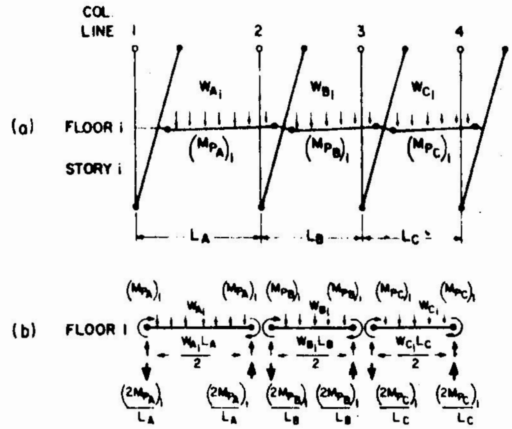


FIG. 7 COMPUTATION OF THE CONTRIBUTION OF FLOOR i TO THE AXIAL FORCES IN COLUMNS FOR THE PRELIMINARY DESIGN

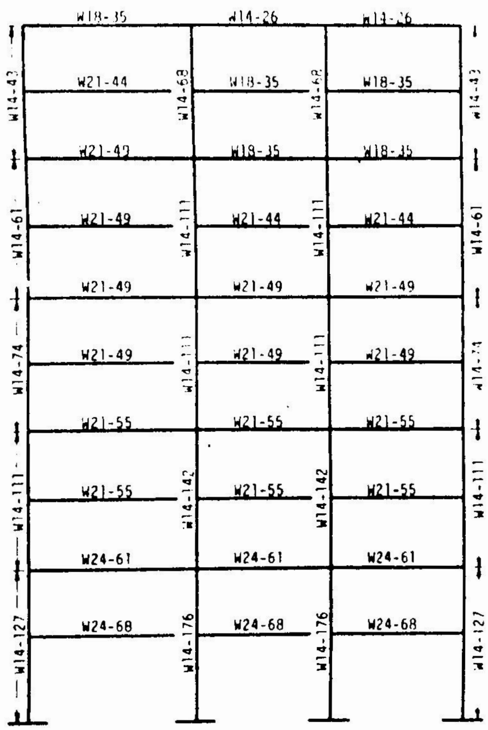


FIG. 8 FINAL OPTIMUM STRONG COLUMN--WEAK GIRDER PRELIMINARY DESIGN

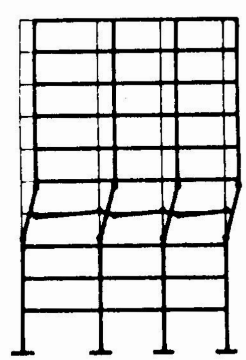


FIG. 9 PARTIAL SWAY MECHANISM

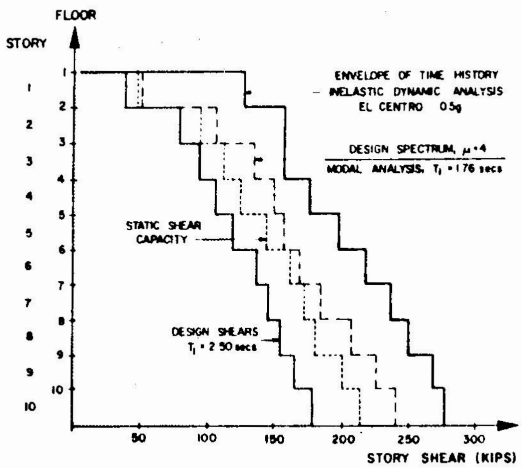


FIG 10 COMPARISON OF STORY SHEARS CORRESPONDING TO PRELIMINARY DESIGN

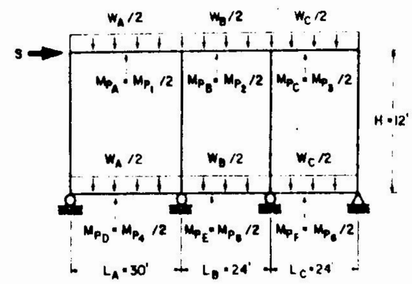
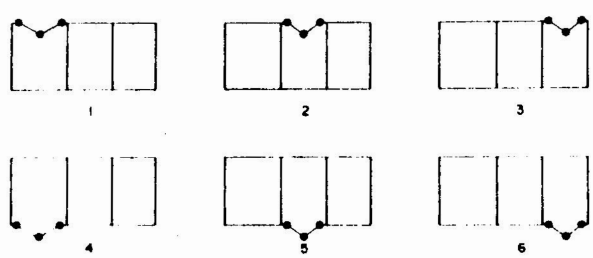


FIG. 11 STORY SUBASSEMBLAGE FOR FINAL DESIGN



NO. OF INDEPENDENT MECHANISMS = $^7C_1$ = 7

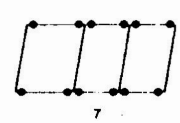


FIG 12 SINGLE OR INDEPENDENT MECHANISMS

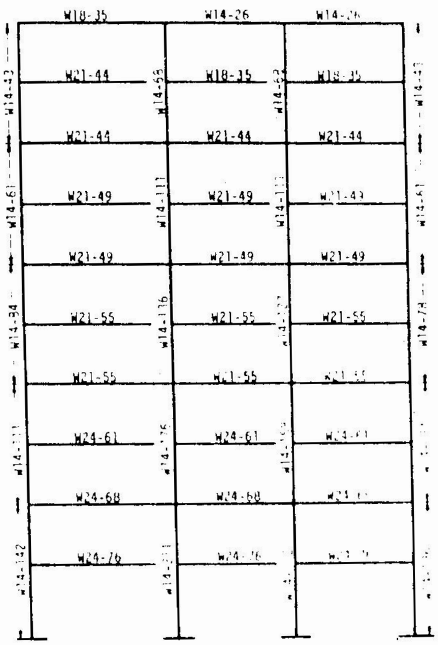


FIG. 13 FINAL OPTIMUM DESIGN

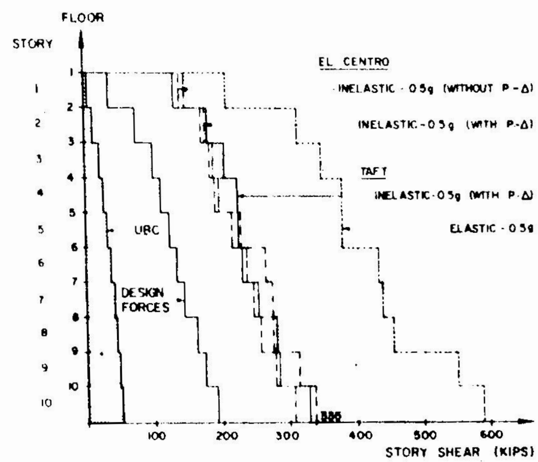


FIG 14 MAXIMUM STORY SHEARS

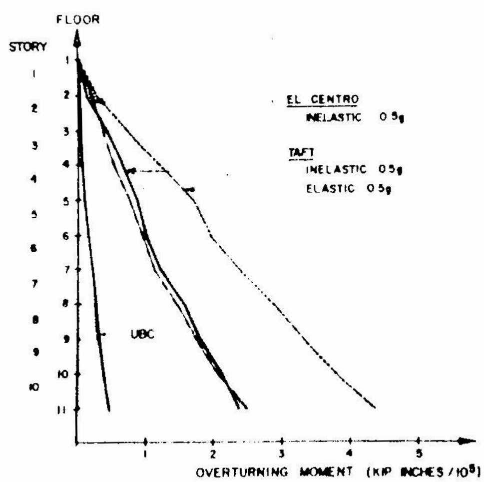


FIG 15 MAXIMUM OVERTURNING MOMENTS

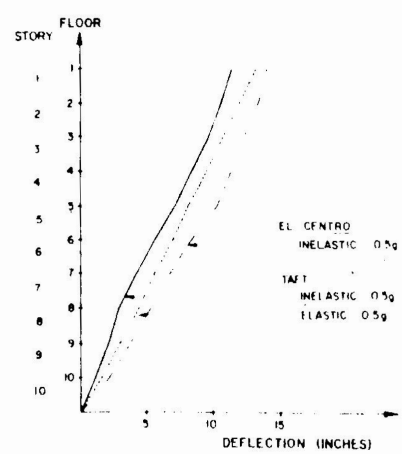


FIG 16 MAXIMUM LATERAL DISPLACEMENTS

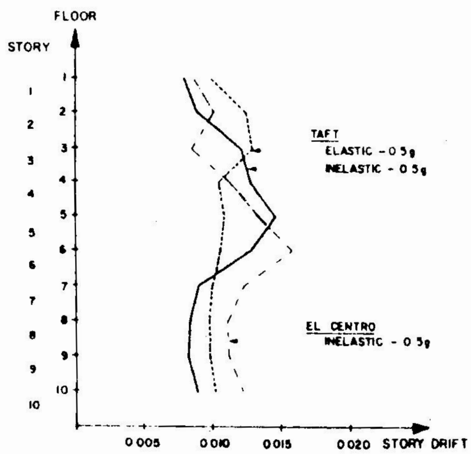


FIG. 17 MAXIMUM STORY DRIFTS

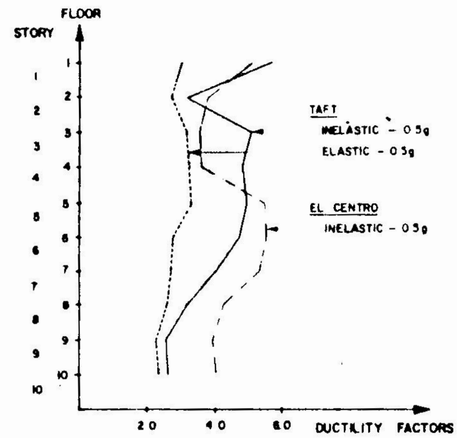


FIG. 18 MAXIMUM CURVATURE DUCTILITY FACTOR-GIRDERS

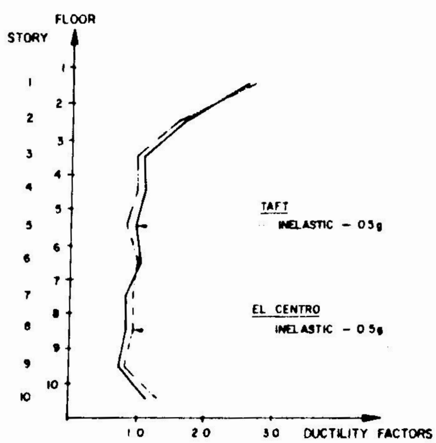


FIG. 19 MAXIMUM CURVATURE DUCTILITY FACTORS - COLUMNS

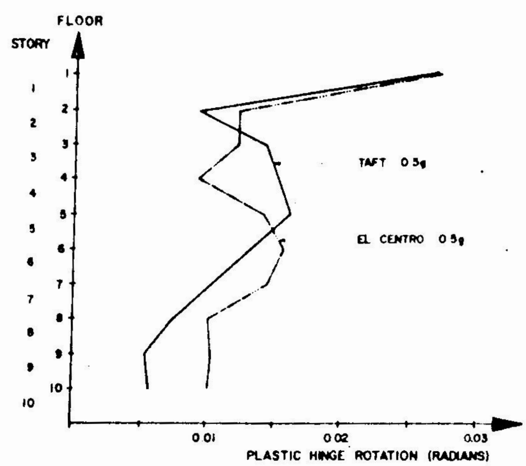


FIG. 20 MAXIMUM PLASTIC HINGE ROTATIONS - GIRDERS

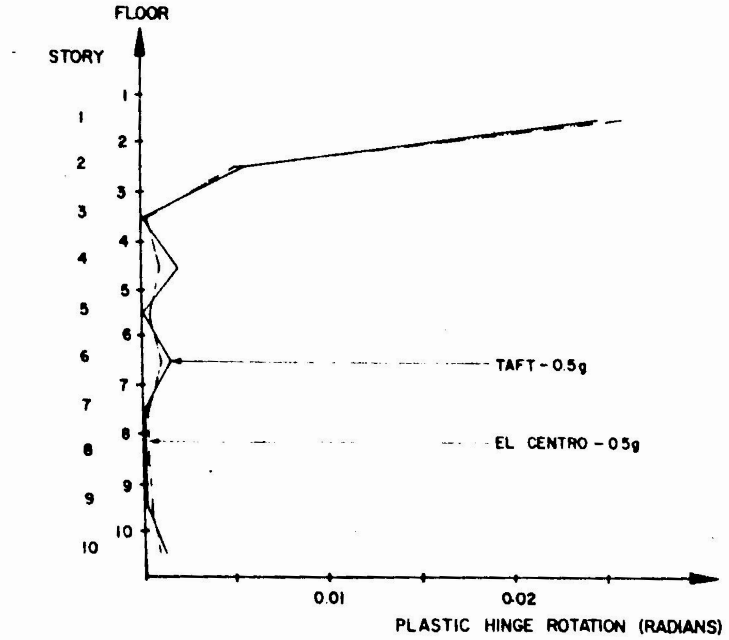


FIG. 21 MAXIMUM PLASTIC HINGE ROTATIONS – COLUMNS

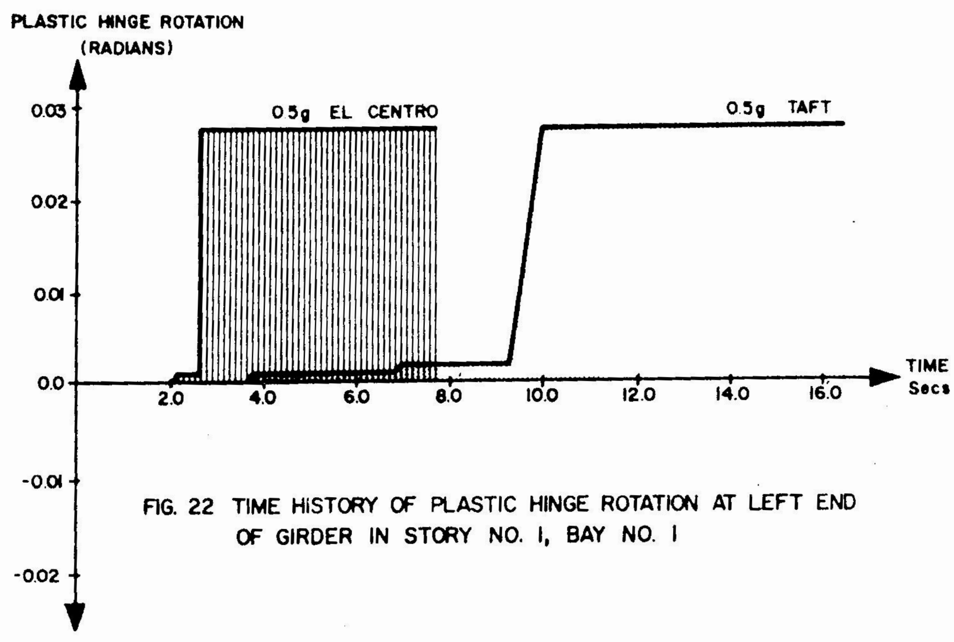


FIG. 22 TIME HISTORY OF PLASTIC HINGE ROTATION AT LEFT END OF GIRDER IN STORY NO. 1, BAY NO. 1