SEISMIC ANALYSIS OF REINFORCED CONCRETE BUILDINGS

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PREFACE

General. This paper describes a method of analysis of single story and multi-storied reinforced concrete buildings. In these buildings, the various vertical framing members are framed into and connected to reinforced concrete floor slab structures. The resisting framing elements of the building are divided into the following categories:

a. Open frames (see Fig. 1 a).
b. Walled frames (see Fig. 7).
c. Seismic shear walls (see Fig. 1 a).

In actual buildings, various complicated types of framing may be encountered which are sometimes difficult to classify wholly into any one of the above groups.

Open rectangular continuous frames are the most convenient construction element insofar as usable space is concerned. However, because of the practical limitation of dimensions, an effective resistance against lateral force cannot be expected integrally especially in multi-storied buildings. Walled type frames usually used in exterior walls have the benefit of a walled girder (spandrel) and a walled column and have several times the resisting capacity of open frames. Seismic walls are the most effective lateral force resisting elements. In the case of high, many storied buildings, they have a reduced resisting capacity in the upper stories as the effective rigidities of these walls decreases owing to the accumulation of the bending deformation of the lower walls. It should be noted here that the structural elements that frame into the seismic walls on either side help to restrain the bending deformation of the wall. It is important that they restrain effectively in order to keep the relative stiffness of the upper story walls as high as is possible practically.

The basic principle of the stress analysis in this paper is to distribute the lateral shear at any one story to the resisting elements of the story. This distribution is made in proportion to the D-Values, distribution coefficients, of these elements.

The D-Value principle was first presented by Dr Tachu Naito in 1922 (1). This principle gives a tool for solving the high statical indeterminacy of the structural framing of a building. This author's years of research on the analysis of structural framing subjected to lateral forces (2) has resulted in a method of systematically applying the D-Value which is practical to use and theoretically quite exact. This method is described briefly in this paper.

The original method consisted of an application of D-Values to the analysis of open frames. It was published in 1929 (3) and since 1935

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has been the basis of the Standard used by the Architectural Institute of Japan (4). The method was extended for application to walled frames for the use of engineers in the Office of the Engineer, GHQ, Far East Command, United States Army by this writer when engaged as a consultant to that Office and was published later in the Journal of the Architectural Institute of Japan. Later the method was expanded to Seismic Walls in collaboration with Mr D.W. Butler and assisted by the personnel of the author's University Laboratory. The method was presented in a co-authored English language text (5). Some of the Tables prepared for the text are included in this paper. The method has been applied to the design of many actual buildings and as presented here contains the latest application techniques.

GENERAL METHOD OF ANALYSIS

Chapter 1. Principles. The analysis of a building for seismic forces is carried out generally by application of the following principles:

1. Direction of earthquake forces. In the analysis, the horizontal component of earthquake force only is considered. The vertical component is disregarded,normally. If particular consideration of the vertical force component is necessary, it requires special application. The horizontal force is assumed to act separately in the longitudinal and transverse directions. Consideration of the simultaneous action of the two force components is not necessary usually. Each case requires a check as to the validity of this assumption.

2. Action of earthquake force. The earthquake force is assumed to act at the floor slab level. If the framing and mass distribution is such that a large amount of force is reactive at the mid-height of any one story, the local stress due to this force must be considered.

3. Displacement of the floor slabs. The assumption is made that the floor structures are rigid in the horizontal direction. Accordingly, all resisting framing elements in any one story are assumed to have the same relative horizontal displacement. When eccentricity exists between the center of shear and the center of rigidity (D-Values), the resulting torsion must be taken into account. When the floor structure is not considered to be rigid sufficiently to validate this assumption, as in the case of a prefabricated floor slab or if the distance between adjacent seismic walls is large, special consideration is necessary (6).

4. Plastic deformation. The shear distribution and the stress analysis of the resisting framing elements is to be made according to the elastic theory. For portions of the structure where the accumulation of stress is overly large, such as the adjacent and boundary framing of a wall, the stress can be decreased due to localized plastic deformation.

5. D-Values, the shearing force distribution coefficients. The D-Value of a resisting elements at any one story is defined by the amount of shear reactive to the element when the relative horizontal displacement at the story under consideration has a unit value. To calculate the D-Value, assume a shearing force distribution and solve for

the stress and deformation of the framing elements. Using the shearing force, V , and the determined relative displacement, δ , of each framing element in the story, the D-Values are then determined by equation 1. See Fig. 1.

$$D = \frac{V}{\delta} \qquad \text{.......} \; (1)$$

The values calculated by equation 1 are the absolute rigidity values. With unit displacement, the relative rigidity values determine the shear distribution.

To simplify the D-Value, into an abstract number, the writer uses the common unit $(\frac{12EK_o}{h_n^2})$ (kg/cm) for n-th story as shown in the equation 2. In connection with the D-Value calculations, the author uses a nominal value expressed in the invert unit of the D-Value, i.e., $(\frac{h_n^2}{12EK_o})$ (cm/kg)

Nominal values

$$\delta \quad \text{in } (\frac{h_n^2}{12EK_o}) \qquad \text{.......} \; (2)$$

$$D = \frac{V}{\delta} \quad \text{in } (\frac{12EK_o}{h_n^2}) \qquad \text{.......} \; (3)$$

6. Condition of the foundation. In the normal building, it is unusual for the foundation to neither settle, displace laterally or rotate. The normal building does not have sufficient foundation rigidity to prevent these occurrences. It is recommended, where necessary, that allowances be made for such occurrences. In the method presented in this paper, the foundation rotation incurred by the large foundation reaction under seismic walls is considered.

Chapter 2. Method of Seismic Analysis.

1. Evaluation of the D-Value.

a. General: Using the best judgment possible, assume an initial shearing force distribution to a vertical frame, full building height; and make the stress analysis of the frame under the action of this assumed shearing force reactive to each resisting element. Use of formula 3 then gives the D-Values of the elements. The accuracy of the D-Values determined can be checked by using them to distribute the shear and then comparing the similarity of the results with the initial assumption.

b. Continuous frame: In the case of ordinary continuous frames, the simple shear distribution assumption is good enough for practical use as the effect of the variation of shearing distribution along the frame height on the D-Values is small, except when the relative stiffness of the framing beams is small, i.e., $\bar{k} < 0.5$. If the analysis is made by the method of moment distribution, the assumption of straight line deflection (7) may be applicable for large values of relative stiffness, i.e., $\bar{k} > 1.0$.

c. Walled frame: The shear distribution assumption for the walled frame is similar to that described above for the continuous frame. In the walled frame analysis however, ordinary frame theory is not valid as the width of the member is large compared with its length. This necessitates consideration of shearing deformation combined with bending deformation. The analysis of the walled frames can be made by a special method of slope deflection or moment distribution by the use of charts and constants prepared by the author and others (2), (5) and (8).

d. Seismic wall: In the calculation of the D-Value for a seismic wall, it is practical to calculate the deflection regarding the wall as a cantilever free standing on the ground and taking into account the bending and shearing deformation under the action of the assumed shearing force (see Fig. 9). Total deflection can be determined combining the deflection due to foundation rotation, δ_R, with those of bending and shear, δ_B and δ_S. Use equation 4:

$$\delta = \delta_S + \delta_B + \delta_R \qquad \ldots\ldots\ (4)$$

In the case of multi-storied walls, the reactions of the beams or girders framed into the walls give an important effect on the bending moments and the bending deflections (See Fig. 1 a). This is the boundary effect and it should be considered in the analysis.

e. Seismic wall with openings: A similar calculation may be applied to walls with openings in which the frame deflection, δ_F, instead of the shear deflection, δ_S, is used. Because of the openings, local bending stresses come into existence similar to the case of the walled frame. Use equation 5 and see Fig. 8.

$$\delta = \delta_F + \delta_B + \delta_R \qquad \ldots\ldots\ (5)$$

2. Distribution of the story shear to the framing elements. The total shear, V_T, at any one story is distributed to the framing elements of the story proportional to the D_n-Value of each element, n, by use of equation 6.

$$V_n = \frac{V_T}{\Sigma D_n} \cdot D_n \qquad \ldots\ldots\ (6)$$

Where n = any framing element, column, wall, walled frame or bent.
ΣD_n = total of floor D-Values

Fig. 2 is an example of shear distribution in the x-direction at any one story. The D-Values are shown beside the framing elements. The D-Values on the right side of the figure are the sum of the story elements. Calculating $\frac{V_T}{\Sigma D_n}$ and multiplying by each D-Value, the distributed shear is obtained as shown Fig. 2 b.

3. Torsional correction of the distributed shear. If the shear centerline (center of mass) at any one story does not coincide with the D-Value centerline for that story in the same direction, the story shear causes not only story translation but also story rotation about the D-Value center (see Fig. 3). For any large eccentricity, it is necessary to correct the value of distributed shear by multiplying by the factor obtained by use of equation 7. As shown, equation 7 gives the value for

x-direction shear correction. For shear correction in the y-direction, equation 7 should have y-direction values interchanged (See Fig. 2 and 3).

$$\alpha = 1 + \frac{\Sigma D x \cdot E}{I x + I y} \cdot y \qquad \ldots\ldots\ (7)$$

Where x, y = coordinates passing through the D-Value center. An example of the shear correction is shown in Fig's. 2 and 3.

4. Calculation of stresses. The stresses of the bent framing, or the elements thereof, are determined for the distributed shear. A sketch of the distributed shears for one of the framing bents is shown in Fig. 1. If one prefers an approximate analysis, the distributed shear to the framing elements may be used to determine the stresses of the elements.

5. Recheck of the analysis. If it is found that the resulting shear distribution is not similar to that of the preliminary assumption, the calculation must be re-run with a new assumption of shear distribution. This is very important in the analysis of seismic walls.

PRACTICAL METHOD OF ANALYSIS

In this part, a practical method of analysis of open frames, walled frames and seismic walls is proposed. The calculations can be made only after the framing dimensions, and tentative section sizes with the stiffness ratios, $\bar{k}_s$, relative to the standard stiffness, K_o, have been determined.

Chapter 1. Open frames.

The analysis of open frames is to be performed in the following steps: 1st, determine the D-Values of the columns. 2nd, determine the column inflection points and calculate the bending moments in the columns. 3rd, determine the beam or girder stresses and the column axial forces. These calculations are based on the mean relative stiffness ratios, $\bar{k}_s$, of all top and bottom beams framed into the columns, as shown below. It should be noted that as $\bar{k}$ becomes smaller the error increases. If $\bar{k}$ is less than 0.2 the method is not accurate enough for reliable use. If there is a large change in column stiffness between consecutive stories as well as large differences between column top and bottom beam stiffnesses, an exact method of calculation must be employed.

1. Column D-Values.

a. Uniform story height: The D-Value of columns is given by equation 8.

$$D = a\, k_c \qquad \ldots\ldots\ (8)$$

Where: a = Constant dependent on $\bar{k}$ (see Fig. 4).
k_c = k of the column being considered.

For the 3 cases shown in Fig. 4, a is obtained as follows:

Case 1. General:

$$a = \frac{\bar{k}}{2 + \bar{k}} \qquad \text{....... (9)}$$

When the sum of the beam stiffnesses at one end, i.e., $k_3 + k_4$, is much larger than that of the other end, a should be kept as not larger than that to be obtained by Case 2 applying equation 10 as if that end were fixed.

Case 2. One end fixed (see Fig. 4):

$$a = \frac{0.5 + \bar{k}}{2 + \bar{k}} \qquad \text{........(10)}$$

Case 3. One end pinned (see Fig. 4):

$$a = \frac{0.5\,\bar{k}}{1 + \bar{k}} \qquad \text{....... (11)}$$

b. Non-uniform height:

Case 4. A column with a height, h', differing from the standard height, h, (see Fig. 4):

$$D' = a' k_c' \qquad \text{....... (12)}$$

where: $$a' = a \times \left(\frac{h}{h'}\right)^2 \qquad \text{....... (13)}$$

Case 5. A column actually composed of two short columns of different heights, h_1 and h_2, which totalled equals the standard height, h, (see Fig. 4):

$$D' = \frac{1}{\frac{1}{D_1}\left(\frac{h_1}{h}\right)^2 + \frac{1}{D_2}\left(\frac{h_2}{h}\right)^2} \qquad \text{....... (14)}$$

if $h_1 = h_2$ $$D' = \frac{4}{\frac{1}{D_1} + \frac{1}{D_2}} \qquad \text{....... (15)}$$

if $D_1 \fallingdotseq D_2$ $$D' = D_1 + D_2 \qquad \text{....... (16)}$$

2. Inflection point and bending moment in columns (fixed at the base).

a. Equation 17 is given for calculating the column height percentage of the inflection point, y (see Fig. 5).

$$y = y_0 + y_1 + y_2 + y_3 \qquad \text{....... (17)}$$

Where:

y_0 = The initial standard percentage height which is determined by the mean beam stiffness ratio, $\bar{k}$, and the location of the story, n, in an m-storied building and is given in Table 1.

y_1 = The correction term, due to variation in value between the upper beam stiffness and lower beam stiffness, is given in Table 2 as the function of the ratio shown in equation 18.

$$\alpha_1 = \frac{k_1 + k_2}{k_3 + k_4} \qquad \text{....... (18)}$$

y_2 = The correction term, due to variation of the story height of the upper adjacent story, is given in Table 3 as the function of the ratio shown in equation 19.

$$\alpha_2 = \frac{h_u}{h} \qquad \text{....... (19)}$$

y_3 = The correction term, due to variation of the story height of the lower adjacent story, is given in Table 3 as the function of the ratio shown in equation 20.

$$\alpha_3 = \frac{h_\ell}{h} \qquad \text{....... (20)}$$

b. Bending moments of columns: As the mode of bending moment of a column over its height is linear, the bottom and top moments are obtained simply by multiplying the column shear by the distance from the point of inflection. This gives:

$$M_B = V \cdot yh \qquad = Vh \cdot y \qquad \text{....... (21)}$$

$$M_T = V \cdot (1-y)h = Vh \cdot (1-y) \qquad \text{....... (22)}$$

When all of the column heights in any one story are equal, it is advisable to use the story column moment, Vh (eqs. 21 and 22). In such cases, it is preferable to distribute the total story moment $V_T h$ to each element in the regular manner of shear distribution.

3. The stresses of beams and the axial force in columns. The end moments of beams are determined by distributing at a joint the sum of the end moments of the upper and lower columns to the ends of the beams in a force direction proportional to the stiffness ratios. The shearing force of a beam is calculated by dividing the sum of the end moments by its span length.

The axial force in a column is determined by summing the shearing force of the beams from the top down to the story being considered.

4. Correction necessary due to the degree of fixation of the column base. When the column base can not be considered as fixed, the following method of correction is recommended: The degree of actual fixation of the column base depends upon the stiffness ratios of column footing tiebeams and the elasticity of the soil. In the actual evaluation of the degree of fixation, it is recommended that the following be considered: The third stiffness, k_{F0}, corresponding to the actual soil resistance is adjudged by the designer, as shown in Fig. 5 c, and the total resisting stiffness is calculated by equation 23.

$$k_{FT} = k_{F0} + k_{F1} + k_{F2} \qquad \text{....... (23)}$$

The principle of the calculation is as follows: 1st, regarding the column base as fixed, find the moment at the base, FEM (see Fig. 5 b).

2nd, release the FEM and propagate the effect upward by the method given below (see Fig. 5 c). 3rd, add the above moments to obtain the corrected moment of the column (see Fig. 5 d). The procedure in determining the stresses in beams are similar to that described above for columns. The procedure of propagation is derived by the assumption that the column moment is propagated uniformly along its height and decreases in value when it passes through the column joint; and additionally that the resisting moment of the beams have their inflection points at the centers.

The moment propagation is made multiplying the moment to be propagated by the carryover factor, C_F, obtained by the use of equation 24, or by the use of Fig. 6.

$$C_F = (1+3\bar{k}) - \sqrt{(1+3\bar{k})^2 - 1} \qquad \ldots\ldots\ (24)$$

where: $\bar{k}$ is given in Fig. 6.

Chapter 2. Walled frames.

1. Representative frame. The analysis of the walled frame is made by use of a representative frame as described herein (see Fig. 7): 1st, determine the rectangular frame line. 2nd, consider the bending and the rigid zone at the joint. 3rd, the rigid zone is determined to a distance of 0.25 d from the joint face as shown in Fig. 7 a.

2. Method of analysis. The analysis is made on the representative frame as shown in Fig. 7 b. The exact and alternate methods of analysis are provided in the texts (2) and (5).

Chapter 3. Seismic walls.

1. The free-standing seismic wall. A practical method is explained herein (see Fig. 9).

a. Shearing deformation: The deflection at the n-th story is given by equation 25.

$$\delta_{sn} = \frac{\kappa V_n h_n}{G A_{wn}} \qquad \ldots\ldots\ (25)$$

where: κ = the coefficient of the shearing angle (Select values between 1.0 and 1.2)

By use of the common unit, $\frac{h_n^2}{12EK_o}$, equation 26 is obtained.

Nominal values $\quad \delta_{sn} = \Delta_{sn} \times \frac{27.6K_o}{h_n} \quad$ in $\left(\frac{h_n^2}{12EK_o}\right)$ .... (26)

where: E/G = 2.3

$$\Delta_{sn} = \frac{\kappa V_n}{A_{wn}}$$

A coefficient for plastic deformation can be induced into equation 26 but for the sake of simplicity it is omitted in the paper.

b. Bending deformation. The bending moment diagrams of the wall are triangular or trapezoidal. Assuming a rectangular diagram of equivalent area, the equation of deflection at the n-th story is obtained by the moment-area method, (M/I), by the use of equation 27.

$$\delta_{Bn} = \sum_{i=1}^{n-1} \frac{M_i h_i}{E I_i} \cdot h_n + \frac{1}{2} \cdot \frac{M_n h_n^2}{E I_n} \quad \text{.......} \quad (27)$$

Where: M_i, M_n = mean bending moment at stories i and n.

Arranging by the common unit, $\frac{h_n^2}{12EK_0}$, equation 28 is obtained.

Nominal value $\delta_{Bn} = 4\Delta_{Bn}\frac{3}{h_n}$ in $\left(\frac{h_n^2}{12EK_0}\right)$ .... (28)

Where: $\Delta_{Bn} = \sum_{i=1}^{n-1} \frac{M_i}{k_{wi}} + \frac{1}{2}\frac{M_n}{k_{wn}}$ .... (29)

$$k_{wn} = \frac{K_{wn}}{K_0} \quad \text{.......} \quad (30)$$

c. Foundation rotation: Regarding the wall rotating as a whole, the deflection of the n-th story is given by equation 31. See Fig. 9.

$$\delta_{Rn} = \theta h_n \quad \text{.......} \quad (31)$$

Transpose, using the common unit, and obtain equation 32.

Nominal value $\delta_{Rn} = \frac{12EK_0\theta}{h_n}$ in $\left(\frac{h_n^2}{12EK_0}\right)$ .... (32)

d. Tabulated calculation of δ_B For ease in calculation use the tabular form shown by Fig. 10. Filling up the table in the order directed, the deflection is obtained.

e. Total deflection: The final deflections and the D-Values are determined by use of equation 33 and 34.

$$\delta_{wn} = \delta_{sn} + \delta_{Bn} + \delta_{Rn} \quad \text{.......} \quad (33)$$

$$D_{wn} = \frac{V_n}{\delta_{wn}} \quad \text{.......} \quad (34)$$

2. Boundary effect on open frames. Generally in the building frame, seismic walls are connected with adjacent open frames. To take into account the boundary effect between these two types of framing, the following method is recommended (see Fig. 11): 1st, assuming the distributed shearing force, calculate the deformation of the wall as a self-standing wall. 2nd, assume that the boundary members keep continuous deformation with the wall as shown is Figs. 11 b and 11 c. 3rd, calculate the FEM due to the above deformation and, applying the method of moment distribution, find the stresses in the boundary frame. On a

portion of the frame adjacent to the wall use one cycle of moment distribution only as this is an approximate solution. The D-Values of the adjacent columns in the plane of the wall can be determined in this same procedure. 4th, determine the beam reaction at the wall and calculate the additional moment caused by the boundary effect. The corrected moment distribution is then obtained as shown in Fig 12. 5th, make the correction necessary to the deflections and the D-Values of the wall. The additional deflections, due to the additional moments, can be calculated using the tabulation method previously described. Add these results to the initial deflection values and the corrected deflections and D-Values can be determined. 6th, if a large difference is found between the initial and the corrected values of deflection and D-Value, repeat steps 3 to 5.

NOTE: In the case of a tall multi-storied seismic wall, the boundary correction of step 4 causes over-correction. It is recommended, therefore, that a part of the additional moments so determined be used in the further calculation of additional deflections, revised D-Values and boundary stresses.

3. Seismic walls with openings.

a. Walls with small openings.

Shearing deflection: The deflection is larger than that of a solid wall. The existence of an opening disturbs the uniform distribution of the shearing stress in the wall, creates local stresses around the opening and, accordingly, increases deformation. The larger the opening, the larger the deflection; and as the size of the opening increases the wall deformation tends towards that of the wall with large openings. Accordingly, the shear deflection of a wall with an opening is named frame deformation, δ_F, and it is calculated by use of equation 35.

$$\delta_F = \frac{1}{r}\delta_S \qquad \text{.......} \quad (35)$$

Where δ_S = the shear deflection of the wall with no opening

$r = (1.0 - 1.25\ p)$

p = periphery ratio of the opening $= \sqrt{\dfrac{\text{opening area}}{\text{gross wall area}}}$

Formula 35 is applicable for a p-value of 0.0 to 0.4

Bending deflection: The moment of inertia for use in the calculation of the deflection, due to the bending moment of the wall as a whole, is to be assumed as between that of the section through the opening, A-A', and that of the section without opening, B-B', as shown in Fig. 8. The calculation of the deflections can be made by use of the method shown for solid walls.

Total deflections and the D-Values: The total deflection at the n-th story is obtained by use of equation 36 as the sum of the above deflections and that due to foundation rotation. The D-Value is obtained by use of equation 37.

$$\delta_{wn} = \delta_{Fn} + \delta_{Bn} + \delta_{Rn} \qquad \cdots\cdots\cdot (36)$$

$$D_{wn} = \frac{V_{wn}}{\delta_{wn}} \qquad \cdots\cdots\cdot (37)$$

Design of the section: At the n-th story of the wall with openings, the necessary design stresses are the axial force, the bending moment and the shearing force. The section size and reinforcement can be calculated then by conventional methods for reinforced concrete. The peripheral stress of the opening, especially that amount of tension requiring added reinforcement, can be determined by the theory of reinforced concrete. Assuming zero tension at the center of one face of the opening, equilibrium between the tension, T, and the shearing stress, τ, in the wall is given in equation 38 (see Fig. 8 a).

$$T = \frac{1}{2}\tau \cdot t \cdot h \qquad \cdots\cdots\cdot (39)$$

b. Walls with large opening (p = over 0.4).

Deflection and the D-Value: The determination of the deflection can be made in a similar manner as that for walls with small openings except the frame deflection is to be calculated as a walled frame (see Fig. 8 b).

Design of the section: The section and reinforcement of the wall at the n-th story can be determined for the stresses given in Fig. 8 b, i.e., the axial force, P, the bending moment, M, of the entire wall; and the walled frame stresses due to the shearing force, V.

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NOMENCLATURE

			Unit
A	=	Area	cm^2
A_w	=	Sectional area of wall	cm^2
D	=	Shear distribution coefficient (D-Value), absolute	kg/cm
	=	Ditto, nominal value expressed in the common unit $\frac{12EK_o}{h_n^2}$ at the n-th story	None
δ	=	Relative horizontal displacement of a column top and bottom, absolute	cm
	=	Ditto, nominal value expressed in the common unit $\frac{h_n^2}{12EK_o}$ at the n-th story	Kg
E	=	Modulus of elasticity of concrete	kg/cm^2
G	=	Modulus of rigidity of concrete	kg/cm^2
h	=	Story height	cm
I	=	Moment of inertia of Area	cm^4
	=	Moment of inertia of D-Values at any one story	cm^2
K	=	Absolute stiffness of a member, I/ℓ	cm^3
K_o	=	Standard stiffness	cm^3
$\varkappa$	=	Kappa, Shape factor relating to shearing stress distribution	None
k	=	Relative stiffness ratio $k = \frac{K}{K_o}$	None
$\bar{k}$	=	Mean value of k of beams relative to column stiffness	None
ℓ	=	Length of member	cm
M	=	Bending moment	kg-cm
n, i	=	Story number	None
V	=	Shear	Kg
W	=	Weight	Kg
x, y	=	Coordinate	cm
y	=	Percentage of total column height to inflection point	None

FIGURE CAPTIONS

Fig. 1: Earthquake force, lateral shear, and deflection in a building frame.

Fig. 2: Shear distribution to the framing with torsional correction.

Fig. 3: Calculation of the torsional correction factor.

Fig. 4: The D-Value calculation for an open frame.

Fig. 5: Calculation of bending moment in colums of an open frame.

Fig. 6: Moment propagation through a joint.

Fig. 7: Walled-frame and its representative joint-rigid frame.

Fig. 8: Design stresses in walls with openings.

a. Small openings.
b. Large openings (W.F.) adjacent to open frame (O.F.).

Fig. 9: The seismic wall and its deflection.

Fig.10: Calculation of bending deflection of the seismic wall.

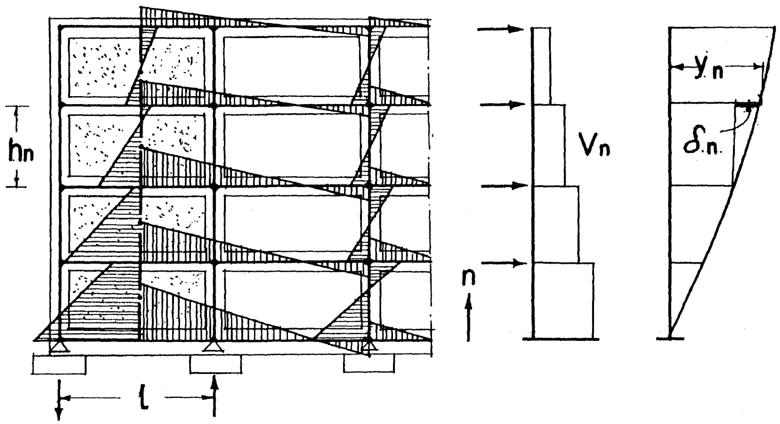
Fig.11: The seismic wall and the boundary frame.

Fig.12: Reaction of adjacent frame on the seismic wall.

Table 1: Values of initial standard percentage height, y_0, for use in equation 17.

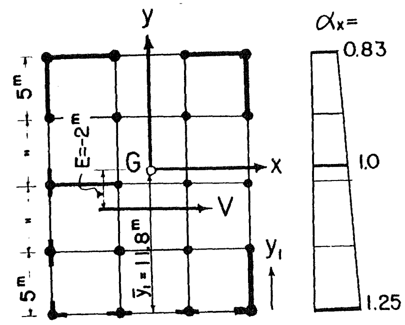
Table 2: Values of correction term, y_1, for use in equation 17.

Table 3: Values of correction terms, y_2, and y_3, for use in equation 17.



(a) (b) (c)

FIG. 1



	D_x	y_1	$D_x y_1$	y_1^2	$D_x y_1^2$
	20	20	400	400	8000
	6	15	90	225	1350
	13	10	130	100	1300
	6	5	30	25	150
	10	0	0	0	0
Σ	55		650		10800

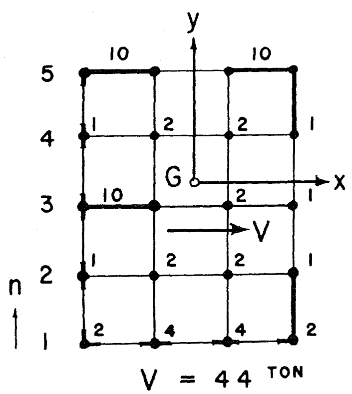
$$\alpha = 1 + \frac{\Sigma D_x \times E}{I_x + I_y}\, y = 1 - \frac{55 \times 2}{5100}\, y$$

$$\bar{y}_1 = \frac{\Sigma D_x \cdot y_1}{\Sigma D_x} = \frac{650}{55} = 11.8^{m}$$

$$I_x = \Sigma D_x \cdot y_1^2 - \Sigma D_x \cdot \bar{y}_1^2 = 3{,}100$$

$$I_x + I_y = 5{,}100$$

FIG. 3



(a)

	D_n	V_n	αV_n
	20	16.0 TON	13.2 TON
	6	4.8	4.4
	13	10.4	10.8
	6	4.8	5.6
	10	8.0	10.0
Σ	55	44.0	44.0

(b)

FIG. 2

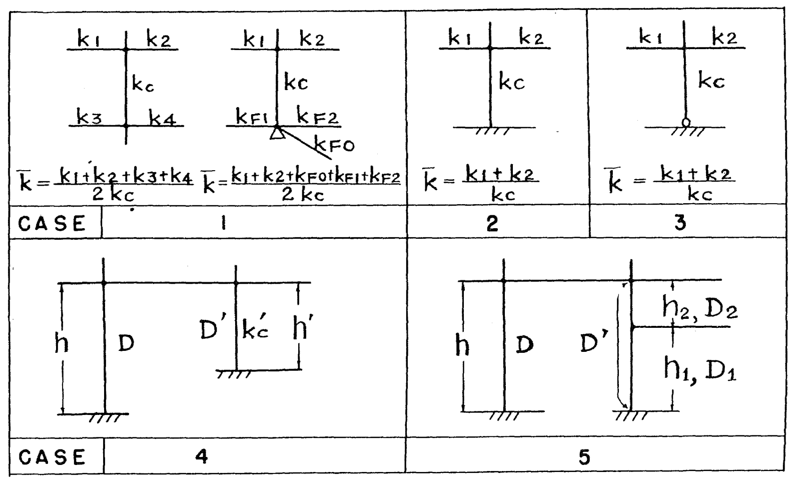


FIG. 4

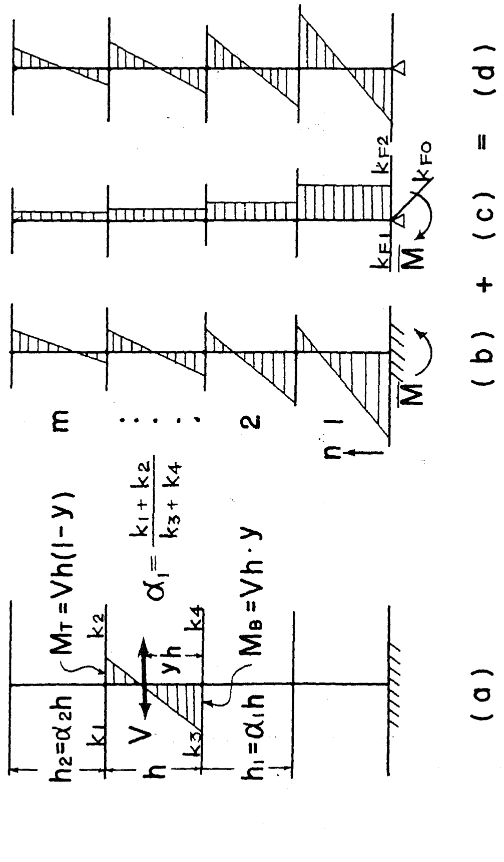


FIG. 5

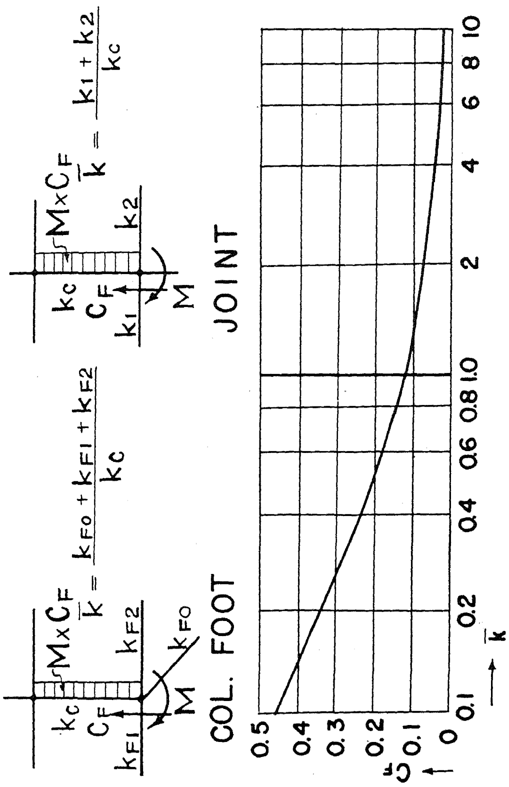


FIG. 6

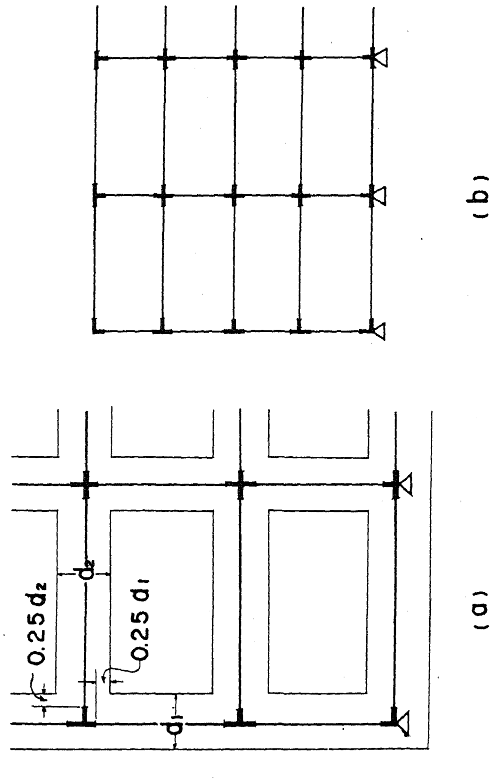


FIG. 7

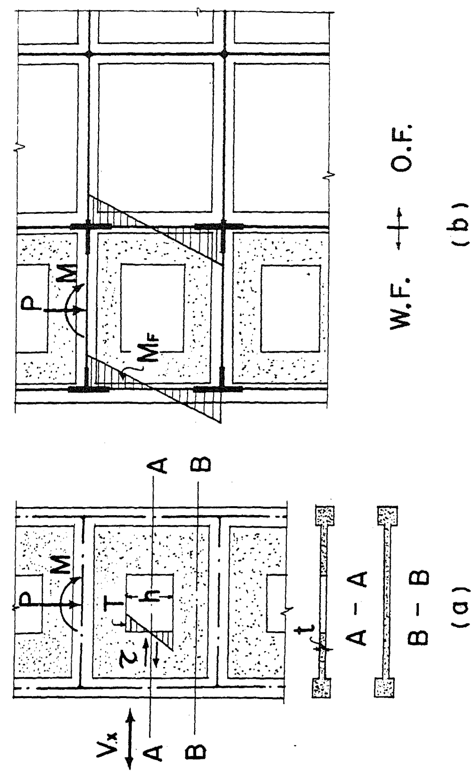


FIG. 8

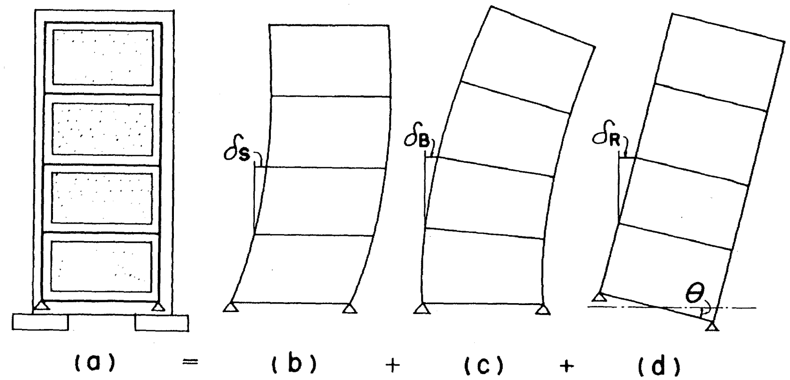
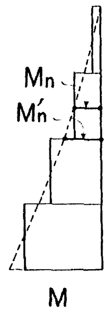


FIG. 9



	δ_{Bn}							
0	1	2	3	4	5	6	7	8
n	$V_n h_n$	M'_n	$2M_n$	k_w	$\frac{2M_n}{k_w}$	$4\Delta_B$	$3/h_w$	δ_{Bn} (6)x(7)
4	A_1	A_2 $(=A_1)$	A_3 $(=A_2)$		A_4	A_5 $(B_4+B_3+A_4)$		
3	B_1	B_2 (A_2+B_1)	B_3 (A_2+B_2)		B_4	B_5 $(C_4+C_3+B_4)$		
2	C_1	C_2 (B_2+C_1)	C_3 (B_2+C_2)		C_4	C_5 $(D_4+D_3+C_4)$		
1	D'_1	D_2 (C_2+D_1)	D_3 (C_2+D_2)		D_4	D_5 $(=D_4)$		

FIG. 10

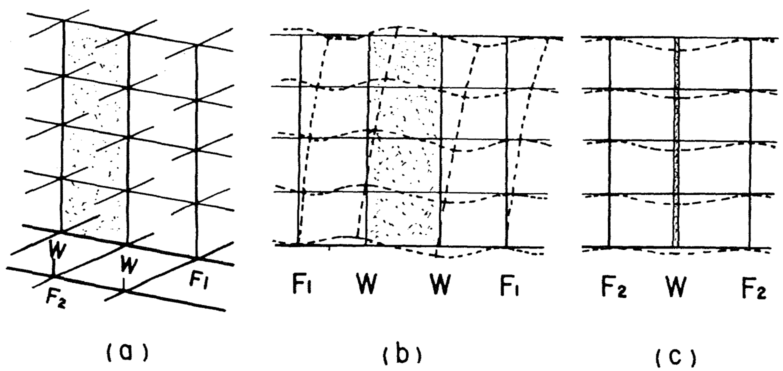


FIG. 11

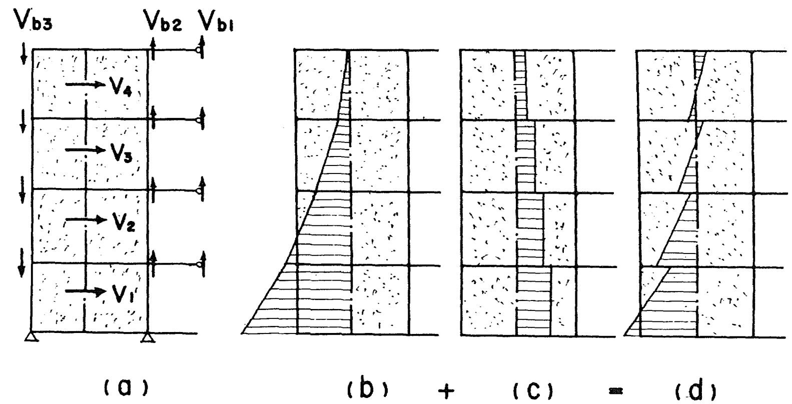


FIG. 12

TABLE I

y_o : STANDARD HEIGHT OF INFLECTION POINT

no. of stories	location of the story	$\bar{k}$ 0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	2.0	3.0	4.0	5.0
1	1	.80	.75	.70	.65	.65	.60	.60	.60	.60	.55	.55	.55	.55	.55
2	2	.45	.40	.35	.35	.35	.35	.40	.40	.40	.40	.45	.45	.45	.45
	1	.95	.80	.75	.70	.65	.65	.65	.60	.60	.60	.55	.55	.55	.50
3	3	.15	.20	.20	.25	.30	.30	.30	.35	.35	.35	.40	.45	.45	.45
	2	.55	.50	.45	.45	.45	.45	.45	.45	.45	.45	.45	.50	.50	.50
	1	1.00	.85	.80	.75	.70	.70	.65	.65	.65	.60	.55	.55	.55	.55
4	4	−.05	.05	.15	.20	.25	.30	.30	.35	.35	.35	.40	.45	.45	.45
	3	.25	.30	.30	.35	.35	.40	.40	.40	.40	.45	.45	.50	.50	.50
	2	.60	.55	.50	.50	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50
	1	1.10	.90	.80	.75	.70	.70	.65	.65	.65	.60	.55	.55	.55	.55
5	5	−.20	.0	.15	.20	.25	.30	.30	.30	.35	.35	.40	.45	.45	.45
	4	.10	.20	.25	.30	.35	.35	.40	.40	.40	.40	.45	.45	.50	.50
	3	.40	.40	.40	.40	.40	.45	.45	.45	.45	.45	.50	.50	.50	.50
	2	.65	.55	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50
	1	1.20	.95	.80	.75	.75	.70	.70	.65	.65	.65	.55	.55	.55	.55
6	6	−.30	.0	.10	.20	.25	.25	.30	.30	.35	.35	.40	.45	.45	.45
	5	.0	.20	.25	.30	.35	.35	.40	.40	.40	.40	.45	.45	.50	.50
	4	.20	.30	.35	.35	.40	.40	.40	.45	.45	.45	.45	.50	.50	.50
	3	.40	.40	.40	.45	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50
	2	.70	.60	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50
	1	1.20	.95	.85	.80	.75	.70	.70	.65	.65	.65	.55	.55	.55	.55
7	7	−.35	−.05	.10	.20	.20	.25	.30	.30	.35	.35	.40	.45	.45	.45
	6	−.10	.15	.25	.30	.35	.35	.35	.40	.40	.40	.45	.45	.50	.50
	5	.10	.25	.30	.35	.40	.40	.40	.45	.45	.45	.45	.50	.50	.50
	4	.30	.35	.40	.40	.40	.45	.45	.45	.45	.45	.50	.50	.50	.50
	3	.50	.45	.45	.45	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50
	2	.75	.60	.55	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50
	1	1.20	.95	.85	.80	.75	.70	.70	.65	.65	.65	.55	.55	.55	.55
8	8	−.35	−.15	.10	.15	.25	.25	.30	.30	.35	.35	.40	.45	.45	.45
	7	−.10	.15	.25	.30	.35	.35	.40	.40	.40	.40	.45	.50	.50	.50
	6	.05	.25	.30	.35	.40	.40	.40	.45	.45	.45	.45	.50	.50	.50
	5	.20	.30	.35	.40	.40	.45	.45	.45	.45	.45	.50	.50	.50	.50
	4	.35	.40	.40	.45	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50
	3	.50	.45	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50	.50	.50
	2	.75	.60	.55	.55	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50
	1	1.20	1.00	.85	.80	.75	.70	.70	.65	.65	.65	.55	.55	.55	.55
9	9	−.40	−.05	.10	.20	.25	.25	.30	.30	.35	.35	.45	.45	.45	.45
	8	−.15	.15	.25	.30	.35	.35	.35	.40	.40	.40	.45	.45	.50	.50
	7	.05	.25	.30	.35	.40	.40	.40	.45	.45	.45	.45	.50	.50	.50
	6	.15	.30	.35	.40	.40	.45	.45	.45	.45	.45	.50	.50	.50	.50
	5	.25	.35	.40	.40	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50
	4	.40	.40	.40	.45	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50
	3	.55	.45	.45	.45	.45	.45	.45	.45	.50	.50	.50	.50	.50	.50
	2	.80	.65	.55	.55	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50
	1	1.20	1.00	.85	.80	.75	.70	.70	.65	.65	.65	.55	.55	.55	.55

y_1 : BEAM CORRECTION TERM

α_1	$\bar{k}$ 0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	2.0	3.0	4.0	5.0
0.4	.55	.40	.30	.25	.20	.20	.20	.15	.15	.15	.05	.05	.05	.05
0.5	.45	.30	.20	.20	.15	.15	.15	.10	.10	.10	.05	.05	.05	.05
0.6	.30	.20	.15	.15	.10	.10	.10	.10	.05	.05	.05	.05	0	0
0.7	.20	.15	.10	.10	.10	.05	.05	.05	.05	.05	.05	0	0	0
0.8	.15	.10	.05	.05	.05	.05	.05	.05	.05	0	0	0	0	0
0.9	.05	.05	.05	.05	0	0	0	0	0	0	0	0	0	0

k_{b1} k_{b2} k_{b3} k_{b4}

$k_{bu} = k_{b1} + k_{b2}$

$k_{bl} = k_{b3} + k_{b4}$

$$\alpha_1 = \frac{k_{bu}}{k_{bl}}$$

α_1 IS NOT CONSIDERED FOR BOTTOM STORY. WHEN k_{bu} HAS A LARGER VALUE THAN k_{bl} USE THE INVERSE RATIO, $\alpha_1 = \frac{k_{bl}}{k_{bu}}$. TAKE NEGATIVE SIGN OF y_1 VALUE THUS OBTAINED.

TABLE 2

y_2 : UPPER STORY CORRECTION TERM

y_3 : LOWER STORY CORRECTION TERM

α_2 u	α_3 l	$\bar{k}$ 0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	2.0	3.0	4.0	5.0
2.0		.25	.15	.15	.10	.10	.10	.10	.10	.05	.05	.05	.05	0	0
1.8		.20	.15	.10	.10	.10	.05	.05	.05	.05	.05	.05	0	0	0
1.6	0.4	.15	.10	.10	.05	.05	.05	.05	.05	.05	.05	0	0	0	0
1.4	0.6	.10	.05	.05	.05	.05	.05	.05	.05	.05	0	0	0	0	0
1.2	0.8	.05	.05	.05	0	0	0	0	0	0	0	0	0	0	0
1.0	1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.8	1.2	-.05	-.05	-.05	0	0	0	0	0	0	0	0	0	0	0
0.6	1.4	-.10	-.05	-.05	-.05	-.05	-.05	-.05	-.05	-.05	0	0	0	0	0
0.4	1.6	-.15	-.10	-.10	-.05	-.05	-.05	-.05	-.05	-.05	-.05	0	0	0	0
	1.8	-.20	-.15	-.10	-.10	-.10	-.05	-.05	-.05	-.05	-.05	-.05	0	0	0
	2.0	-.25	-.15	-.15	-.10	-.10	-.10	-.10	-.10	-.05	-.05	-.05	-.05	0	0

$h_u = \alpha_2 h$

h

$h_l = \alpha_3 h$

y_2: USE $\alpha_2 = \frac{h_u}{h}$ POSITIVE IF $\alpha_2 > 1$

y_3: USE $\alpha_3 = \frac{h_l}{h}$

DO NOT USE y_2 FOR TOP STORY OR y_3 FOR BOTTOM STORY

TABLE 3