SEISMIC ANALYSIS OF CORE-WALL BUILDINGS

by

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ABSTRACT

This paper is concerned with the static and dynamic analyses of a core-wall building. In the first part the elastic response of a core-wall building to earthquake ground motions is analyzed. The maximum response values of six core-wall buildings to two recorded earthquakes are obtained and the resultes are compared with those computed by modal analysis of "root mean square" type. In the second part the author proposes the method to evaluate the shears and moments of structural members of a core-wall building subjected to lateral static forces. The method includes the elastic-plastic analysis as well as the elastic one.

INTRODUCTION

When a building has a box-shape wall extending over the height of the building as shown in Fig. 1, it is stated to be a core-wall building. The mechanical behavior of a tall core-wall building subjected to earthquake ground motion or lateral forces may be characterized by a so-called boundary effect, which means the interaction between the core-wall and the adjacent open frames, and this effect gives a large amount of lateral resisting capacity to the core-wall.

Since this type of building structure is complicated in its mechanical behavior, a simplification to a certain extent is necessary. It is intended in the first part of this paper to obtain the general feature of the elastic responses of core-wall buildings to earthquake ground motions, and for this purpose a simple reduced system is considered. This system is a combination of a uniform shear beam type structure and a uniform cantilever type structure having elastic restraint to rotation, and these structures represent the open frames and the core-walls, respectively.

In the second part the author proposes the method of static analysis of core-wall buildings subjected to lateral forces. The method includes the elastic-plastic analysis as well as the elastic one. The main part of the analysis is the solution of simultaneous equations in which the number of unknowns is reduced to the number of stories of the building. This method can be also applied to the dynamic analysis.

PART I DYNAMIC ANALYSIS

ASSUMPTIONS

To investigate the general effect of earthquake ground motions on the stress distributions of the structure, it is assumed that:

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1. The masses and the stiffnesses are uniformly distributed over the height of the building.

2. All the members of the structure are distorted within an elastic range.

3. The open frames vibrate as a shear beam type structure, and the core walls behave as a cantilever standing on the foundation and are subjected to restaint action from the adjacent beams.

4. There is no foundation rotation.

With these assumptions the system can be analyzed as a combination of shear beam type structure and a flexural cantilever type structure having restraint proportional to its rotation.

FUNDAMENTAL RELATIONSHIP

Open frames:

As the open frames are assumed to be the shear beam, the shear at a section located a distance x from the base is expressed as follows:-

$$Q_F(x) = G_F \frac{\partial y}{\partial x} \tag{1}$$

Where G_F is the distributed rigidity of the frame (see Fig. 3).

Core walls:

The core wall is represented by the cantilever and the boundary effect from the adjacent beams is taken into account as the flexural resistance proportional to the slope $\partial y/\partial x$. Then the bending moment of the wall is expressed as

$$M_w(x) = -EI\frac{\partial^2 y}{\partial x^2} = \int_x^H Q_w(x)\,dx' - \int_x^H K_B \frac{\partial y}{\partial x}\,dx' \tag{2}$$

Where EI is the flexural rigidity of the wall and K_B is a sum of the effective stiffness of adjacent beams (see Fig. 4). Differentiating equation (2), the expression for the shear of the wall is obtained.

$$Q_w(x) = -EI\frac{\partial^3 y}{\partial x^3} + K_B\frac{\partial y}{\partial x} \tag{3}$$

EQUATION OF MOTION

Substituting the above fundamental relationship (1) and (3) into the D'Alembert's principle, the following differential equation of motion is derived.

$$EI\frac{\partial^4 y}{\partial x^4} + \rho A\frac{\partial^2 y}{\partial t^2} - (G_F + K_B)\frac{\partial^2 y}{\partial x^2} = 0 \tag{4}$$

where $\rho A = M/H$ = mass of the building per unit length.

If the system has the damping force proportional to the strain velocity, the equation of motion (4) is modified as

$$EI\left(1+r_i\frac{\partial}{\partial t}\right)\frac{\partial^4 y}{\partial x^4} + \rho A\frac{\partial^2 y}{\partial t^2} - (G_F+K_B)\left(1+r_i\frac{\partial}{\partial t}\right)\frac{\partial^2 y}{\partial x^2} = 0 \qquad (5)$$

where r_i is the coefficient of internal friction.

Putting $\alpha^2 = \rho A H^4/EI = MH^3/EI$, $\gamma = (G_F+K_B)H^2/2EI$ and $y = Y + y_0$ equation (5) becomes (y_0 = ground displacement)

$$\left(1+r_i\frac{\partial}{\partial t}\right)\frac{\partial^4 Y}{\partial x^4} + \frac{\alpha^2}{H^4}\frac{\partial^2 Y}{\partial t^2} - \frac{2\gamma}{H^2}\left(1+r_i\frac{\partial}{\partial t}\right)\frac{\partial^2 Y}{\partial x^2} = -\frac{\alpha^2}{H^4}\frac{d^2 y_0}{dt^2} \qquad (6)$$

SOLUTION BY MODAL EXPRESSION

The general solution of equation (6) can be obtained as the sum of the solution of free vibration and the particular solution, both of which are expressed by superposition of modal values.

$$Y = \sum_{s=1}^{\infty} {}_s\beta \cdot {}_s u(x) \cdot {}_s q'(t) + \sum_{s=1}^{\infty} {}_s\beta \cdot {}_s u(x) \cdot {}_s \bar{q}(t) = \sum_{s=1}^{\infty} {}_s\beta \cdot {}_s u(x) \cdot {}_s q(t) \qquad (7)$$

Where ${}_s u(x)$, ${}_s q(t)$ and ${}_s\beta$ are the normal function, normal coordinate and the participation factor in s-th mode, respectively. The normal coordinate ${}_s q(t)$ is obtained as the solution of the following ordinary differential equation:-

$$\frac{d^2 {}_s q}{dt^2} + 2\,{}_s h \cdot {}_s n \cdot \frac{d\,{}_s q}{dt} + {}_s n^2 \cdot {}_s q = -\frac{d^2 y_0}{dt^2} \qquad (8)$$

Where ${}_s h$ and ${}_s n$ are the fraction of critical damping and natural circular frequency without damping in s-th mode, respectively. The participation factor ${}_s\beta$ is determined in this case by

$${}_s\beta = \int_0^H {}_s u(x)\,dx \Big/ \int_0^H {}_s u^2(x)\,dx \qquad (9)$$

The expression of the normal function ${}_s u(x)$ takes the form

$$u(x) = C_1 \sin(p_1 x/H) + C_2 \cos(p_1 x/H) + C_3 \sinh(p_2 x/H) + C_4 \cosh(p_2 x/H) \qquad (10)$$

or $$u(\xi) = C_1 \sin p_1\xi + C_2 \cos p_1\xi + C_3 \sinh p_2\xi + C_4 \cosh(p_2\xi) \qquad (10')$$

where $$p_1 = \sqrt{\sqrt{\alpha^2 n^2 + \gamma^2} - \gamma}\ , \quad p_2 = \sqrt{\sqrt{\alpha^2 n^2 + \gamma^2} + \gamma} \qquad (11)$$

$$\xi = x/H$$

To compute above values natural frequency n must be determined first. n is calculated by the frequency equation which is derived from the boundary

conditions. In the case where the core-wall is fixed at the base the following frequency equation is obtained:

$$2p_1^2p_2^2 - p_1p_2(p_1^2-p_2^2)\sin p_1 \sinh p_2 + (p_1^4+p_2^4)\cos p_1 \cosh p_2 = 0 \qquad (12)$$

Accordingly, the following expressions for the displacement shears and moments are derived as the modal values:-

Displacement:-

$$Y = \left[(p_1^2\cos p_1 + p_2^2\cosh p_2)(p_2 \sin p_1\xi - p_1 \sinh p_2\xi) - p_1p_2(p_1\sin p_1 + p_2\sinh p_2)(\cos p_1\xi - \cosh p_2\xi)\right]C\,e^{int} \qquad (13)$$

Shear in the open frames:-

$$Q_F = \left[p_1p_2(p_1^2\cos p_1 + p_2^2\cosh p_2)(\cos p_1\xi - \cosh p_2\xi) + p_1p_2(p_1\sin p_1 + p_2\sinh p_2)(p_1\sin p_1\xi + p_2\sinh p_2\xi)\right]\left\{\frac{G_F}{H}\right\}C\,e^{int} \qquad (14)$$

etc.

These modal values in 1st to 3rd mode for $\gamma = 20$ are illustrated in Fig. 5.

EARTHQUAKE RESPONSE OF SOME IDEAL BUILDINGS

(a) Sample buildings

Six different buildings are considered in the response analysis. The number of stories and the column sections of these buildings are as in the following table. These dimensions are chosen as having simple integer values of α and γ. The natural periods of buildings are also shown in Table 1. The fraction of critical damping is assumed to be 0.05 for the first mode in every building.

	Number of stories	α^2	γ	Column Section(cm)	Natural Period (Fundamental) (Sec)
BUILDING A	12	1	5	70 x 70	0.876
" B	12	1	10	85 x 85	0.686
" C	20	10	20	75 x 75	1.66
" D	20	10	50	110 x 110	1.125
" E	40	100	100	80 x 80	2.61
" F	40	100	200	110 x 110	1.89

(b) Method of computation

The analog computer "SERAC" has been used to integrate equation (8) and to superpose the modal values. Each modal value for displacement, shear and moment were obtained by digital computer according to equations in the preceding section. The computed shears in core-wall of frame C are illustrated in Fig. 6.

(c) Earthquake ground motion

The following two types of earthquake motions are used.

EL Centro 1940 NS type
Tokyo 101 1956 NS type

(d) Results

The results are obtained for displacement, shear and moment responses as illustrated in Fig. 6. The maximum values in each response are plotted in Fig. 7 - 18 by the (•) mark.

The (X) mark in the same figures shows so-called "root mean square" values, which are difined as

$$Y_{max} = \sqrt{\sum_s {}_sY_{max}^2} \quad , \quad etc. \tag{15}$$

and the solid lines represent the maximum values when the first mode only is considered.

Comparing these maximum values it can be said that

(1) "Root mean square" values are always very close to the exact maximum values except in a very few cases.

(2) In most cases the first mode maxima are nearly equal to the exact maxima, and this means the effect of higher modes is very small. The exceptional case is seen in Frame F, for which the maximum values of the first and second modes are comparable.

PART II STATIC ANALYSIS

PRINCIPLES OF THE ANALYSIS

The structure of the core-wall building can be represented by the frame in which the core-wall is considered as a one-dimensional member expressed at the center line of its section and the end portion of the adjacent beams framed into the wall is considered as a rigid zone (see Fig. 2).

To analyze this representative frame subjected to lateral forces, the proposed method employs the following principles:-

1. Initially, the frame is cut off at the boundary of the core-wall and the open frame, and the interaction between the two parts is taken into account in the analysis to satisfy the continuity of the boundary. By this procedure the number of unknowns in the fundamental equations can be reduced to the number of stories.

2. In the analysis of the core-wall, the bending and shearing deformation as well as the boundary effect from the adjacent beams is considered.

3. It is assumed that the open frame is translated horizontally as the shear type structure, and that the lateral rigidity of the frame is expressed by the formula in Muto's D-method.

4. The plastic deformation is considered in the portion where the stress concentration takes place.

BOUNDARY EFFECT

a. Boundary effect on the core-wall

The beam reaction to the core-wall due to the boundary effect can be represented by the resisting moment at the center line of the wall based on the following assumptions:-

1. The other end of the beams connected in the wall plane has no rotation and vertical translation.

2. The other end of the beams connected perpendicular to the wall has rotation to be determined by the joint rigidity.

Then the resisting moment of the beam is expressed as follows (see Fig. 4):

$$M_B = 6EK_0\,\theta \cdot \Sigma k_{Bc} \qquad (16)$$

where M_B = total resisting moment of adjacent beams
θ = joint rotation at the wall center line
k_{Bc} = effective stiffness ratio of an adjacent beam
$= \left\{\frac{1}{3} + \frac{l_a}{l} + \frac{1}{2}\left(\frac{l_a}{l}\right)^2\right\} k_B$ ... for a beam in the wall plane (17)
$= (l_a/l')^2 k_B'$ ... for a beam in the transverse plane (17')

(l, l' = span length of the adjacent beam, l_a = distance from the wall center to the adjacent beam, k_B, k_B' = stiffness ratio of the adjacent beam)

b. Boundary effect on the open frame

Because of the boundary effect the rigidity of the column next to the core-wall in the wall plane is larger than that computed by the formula in Muto's D-method. In this analysis the rigidity of the boundary column is approximately calculated by the following formula:-

$$D_{cn}' \fallingdotseq 1.5\,D_{cn} \qquad (18)$$

where D_{cn} = lateral force distribution coefficient of the boundary column in Muto's D-method

As for the boundary beams the moment distribution method is applied to compute the end moment of the beams taking the rotation of the core-wall as the enforced deformation. This computation will be performed after the wall rotations are determined as the solution of the three term equations.

FUNDAMENTAL EQUATIONS

a. Open frame

According to the formula in Muto's D-method the fundamental relationship for open frames is expressed as follows (see Fig. 3):-

$$R_n = Q_{Fn} / G_{Fn} \quad \text{or} \quad Q_{Fn} = G_{Fn} \cdot R_n \tag{19}$$

where $R_n = \delta_n / h_n$ = rotation of the column in the n-th story
Q_{Fn} = shear force in the open frames in the n-th story
$G_{Fn} = \Sigma D_{cn} \cdot 12EK_0 / h_n$ = frame rigidity in the n-th story
ΣD_{cn} = summation of the D-values of the n-th story column,
E = Young's modulus, = Standard stiffness,
h_n = height of the n-th story

b. Core-wall and boundary beams

The core-wall is considered as a column which is connected to the boundary beams, and the fundamental relationship between end moments and rotations is expressed by the slope deflection equations. The shearing deformation of the wall is taken into consideration in a separate form.

$$R_n = \underset{\text{(bending)}}{R_{Mn}} + \underset{\text{(shearing)}}{R_{Qn}} \tag{20}$$

$$R_{Qn} = \kappa Q_{wn} / G A_n \tag{21}$$

(κ = shape factor, G = modulus of rigidity, A_n = sectional area of the wall in the n-th story)

end moments of the wall:-

$$M_{n,n+1} = 2EK_0 k_{wn} (2\theta_n + \theta_{n+1} - 3R_{Mn}) = k_{wn} (2\varphi_n + \varphi_{n+1} + \psi_{Mn}) \tag{22}$$

$$M_{n+1,n} = k_{wn} (\varphi_n + 2\varphi_{n+1} + \psi_{Mn}) \tag{23}$$

(θ_n = rotation of the wall at the n-th floor)

shear of the wall:-

$$Q_{wn} = -(M_{n,n+1} + M_{n,n+1}) / h_n = G A_n R_{Qn} / \kappa \tag{24}$$

end moment of the boundary beam:-

$$M_{Bn} = 3\varphi_n \cdot \Sigma k_{Ben} \tag{25}$$

c. Derivation of the three term equation

From the conditions of equilibrium the following equations must be satisfied:-

$$M_{n,n+1} + M_{n,n-1} + M_{Bn} = 0 \tag{26}$$

$$M_{n,n+1} + M_{n+1,n} = -Q_{wn} h_n \tag{27}$$

$$M_{n,n-1} + M_{n-1,n} = -Q_{wn-1} h_{n-1} \tag{28}$$

$$Q_n = Q_{wn} + Q_{Fn} \tag{29}$$

where Q_n = total shear in the n-th story.

From equations (19), (21) and (29) Q_{wn} is expressed as a function of Q_n and ψ_{Mn}.

$$Q_{wn} = \frac{GA_n}{GA_n + \kappa G_{Fn}} Q_n + \frac{GA_n \cdot G_{Fn}}{6EK_0(GA_n + \kappa G_{Fn})} \psi_{Mn} \qquad (30)$$

By eliminating Q_{wn} and ψ_{Mn} from equations (22) to (30) the three-term equation is obtained with respect to the rotation φ_n.

$$(n=n) \quad k_{wn}(2-3B_n)\varphi_{n+1} + \{k_{wn}(4-3B_n) + k_{wn-1}(4-3B_{n-1}) + 6k_{Ben}\}\varphi_n$$
$$+ k_{wn-1}(2-3B_{n-1})\varphi_{n-1} = B_n Q_n h_n / S_n + B_{n-1} Q_{n-1} h_{n-1} / S_{n-1} \qquad (31)$$

where $$S_n = 1 + \kappa G_{Fn}/GA_n = 1 + \Sigma D_{cn} \cdot \kappa \cdot 12EK_0 / GA_n h_n \qquad (32)$$

$$B_n = \frac{1}{1 + \Sigma D_{cn} / S_n k_{wn}} \qquad (33)$$

At the top and bottom boundaries the equations take the following special form:-

At the top:

$$(n=m+1) \quad \{k_{wm}(3-4B_m) + 6k_{Bem+1}\}\varphi_{m+1} + k_{wm}(2-3B_m)\varphi_m = B_m Q_m h_m / S_m \qquad (34)$$

At the bottom: fixed at the base

$$(n=2) \quad k_{w2}(2-3B_2)\varphi_3 + \{k_{w2}(4-3B_2) + 6k_{Be2}\}\varphi_2 = B_2 Q_2 h_2 / S_2 + B_1 Q_1 h_1 / S_1 \qquad (35)$$

elastically supported at the base

$$(n=1) \quad k_{w1}(2-3B_1)\varphi_2 + \{k_{w1}(4-3B_1) + 6k_{Be1}\}\varphi_1 = B_1 Q_1 h_1 / S_1 \qquad (36)$$

Using equations (34), (31) and (35) or (36) for uppermost, intermediate and lowermost stories respectively, the fundamental equations take the form of the simultaneous equations. Solving these equations the rotations of the wall can be obtained and, accordingly, the end moments and shears of the members can be calculated using the formulas shown in the next section.

When the building has more than two walls with different cross-section or different lateral rigidity, the following procedure is recommended:- First, the above equations are constructed with respect to the rotations of the most rigid wall and the estimated rigidities of the other walls are added to the rigidity of open frames (D-value) in the equations. Second, after calculating the rigidity of the first wall, the same procedure is traced with respect to the second wall. Third, the rigidity of the first wall is recalculated, and if a large difference is found between the initial and corrected rigidity, the same procedure must be repeated.

d. End moments and shears

End moments and shears are calculated by the following formulas:-

Shear in wall:

$$Q_{wn} = \frac{1}{S_n} \left(Q_n + 2 \sum D_{cn} \frac{\psi_{Mn}}{h_n} \right) \tag{37}$$

where ψ_{Mn} is obtained directly from the φ-values.

$$\psi_{Mn} = -\frac{B_n}{2} \left(3\varphi_n + 3\varphi_{n-1} + \frac{Q_n h_n}{S_n k_{wn}} \right) \tag{38}$$

Shear in frame $\quad Q_{Fn} = Q_n - Q_{wn} \qquad (39)$

moment in wall:

$$M_{n,n+1} = -\sum_{i=n}^{m} Q_{wi} h_i + \sum_{i=n+1}^{m+1} M_{Bi} = k_{wn} (2\varphi_n + \varphi_{n+1} + \psi_{Mn}) \tag{40}$$

CONSIDERATION OF PLASTIC DEFORMATION

a. General

When the computed moment or shear is beyond the allowable value (yield value) of the section preliminarily determined, it is recommended that the plastic deformation be considered. The following three kinds of stresses can easily go beyond the allowable stress in the case of a core-wall type structure.

1. Bending moment at the bottom end of core wall

2. Bending moment at the end of the boundary beam

3. Shear in the core wall

In this analysis the special equations are used to obtain the end moments and shears when considering the elastic-plastic type deformation for the above three cases.

b. Modified three-term equation

When taking the effect of plastic deformation into consideration, the three-term equations used in the elastic analysis are modified as follows:-

Case 1. Wall bending

Fixed at the base:

Putting $3k_{Be1} \cdot \varphi_1 = -M_Y$ in the elastic equations, the following equations at n=1 and 2 are derived. These equations are solved simultaneously together with the elastic equations for n more than 2.

$$(n=2) \quad k_{w2}(2-3B_2)\varphi_3 + \{k_{w2}(4-3B_2) + k_{w1}(4-3B_1) + 6k_{Be2}\}\varphi_2 + k_{w1}(2-3B_1)\varphi_1 = B_2 Q_2 h_2/S_2 + B_1 Q_1 h_1/S_1 \tag{41}$$

$$(n=1) \quad k_{w1}(2-3B_1)\varphi_2 + k_{w1}(4-3B_1)\varphi_1 = B_1 Q_1 h_1/S_1 + 2M_Y \tag{42}$$

where M_Y = design (yield) moment at the bottom of the wall

Elastically supported at the base:

The elastic equation (36) is replaced by equation (42). All the equations are the same as those in the case of the fixed base in this particular case.

Case 2. Beam bending

To take the plastic deformation of the boundary beam into consideration, equation (43) is used instead of equation (31).

$$(n=n)\quad k_{wn}(2-3B_n)\varphi_{n+1} + \{k_{wn}(4-3B_n) + k_{wn-1}(4-3B_{n-1})\}\varphi_n + k_{wn-1}(2-3B_{n-1})\varphi_{n-1} = B_nQ_nh_n/S_n + B_{n-1}Q_{n-1}h_{n-1}/S_{n-1} - 2M_{BY} \quad (43)$$

where M_{BY} = design (yield) moment of the boundary beam considered at center line of the wall

Case 3. Wall shear

To consider the plastic deformation of the wall in the n-th story, the elastic equation (31) is replaced by the following equation:-

$$(n=n)\quad -k_{wn}\varphi_{n+1} + \{k_{wn} + k_{wn-1}(4-3B_{n-1}) + 6k_{Ben}\}\varphi_n + k_{wn-1}(2-3B_{n-1})\varphi_{n-1} = Q_Yh_n + B_{n-1}Q_{n-1}h_{n-1}/S_{n-1} \quad (44)$$

where Q_Y = design (yield) shear of the wall

ILLUSTRATIVE EXAMPLE

To constants of the given frame are shown in Fig. 19. The computations are made using the elastic equation first, then assuming the yield moment at the bottom of the wall to be 7,700 t m (elastic moment is 10,500 tm) recalculation is made considering the yield hinge formation at the bottom of the wall. The result are shown for shear and moment distributions in Figs. 20 and 21.

ACKNOWLEDGEMENT

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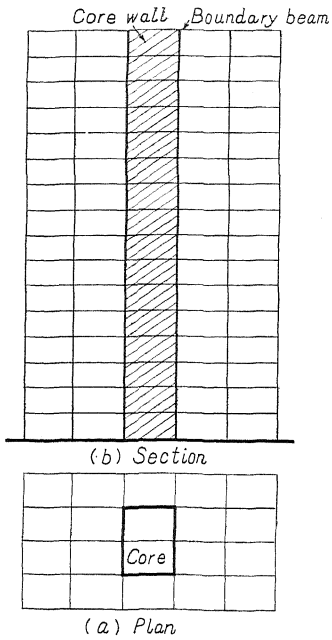


Fig. 1 Core-wall building

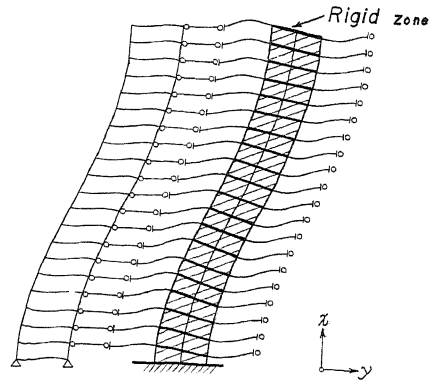


Fig. 2 Reduced vibratory system

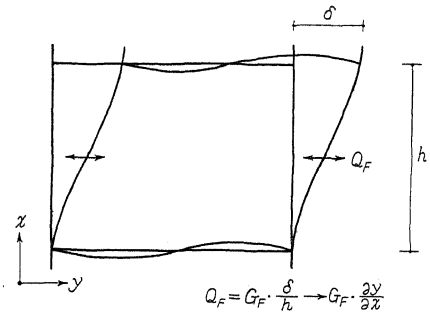


Fig. 3 Rigidity of open frames

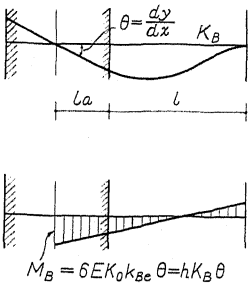


Fig. 4 Rigidity of boundary beams

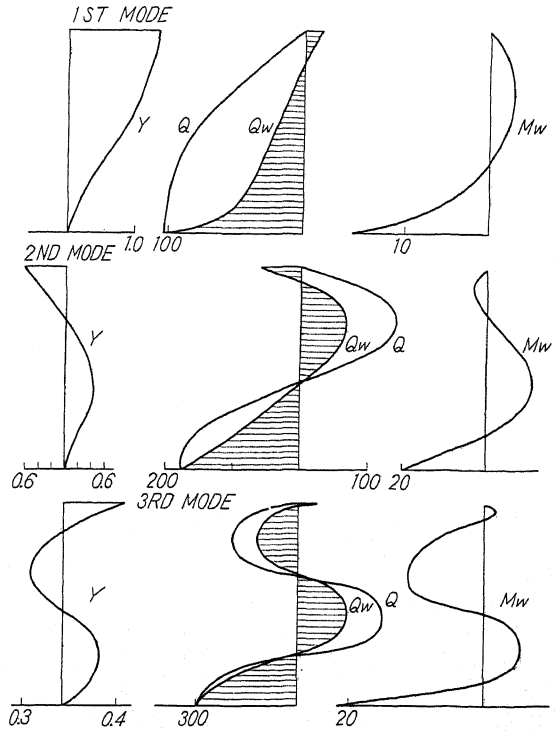


Fig. 5 Illustration of mode shapes, shears and moments

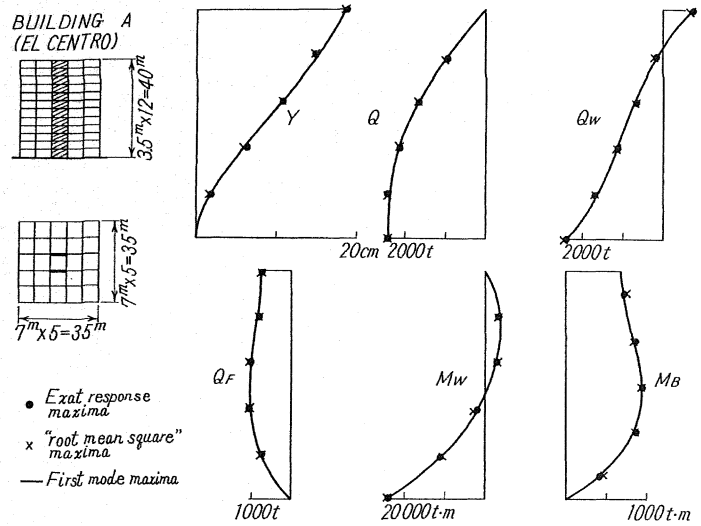


Fig. 7 Comparison of maximum response values (1)

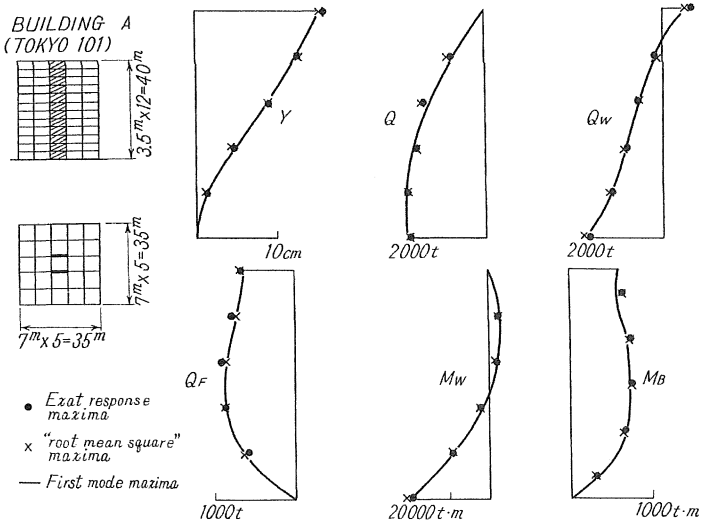


Fig. 8 Comparison of maximum response values (2)

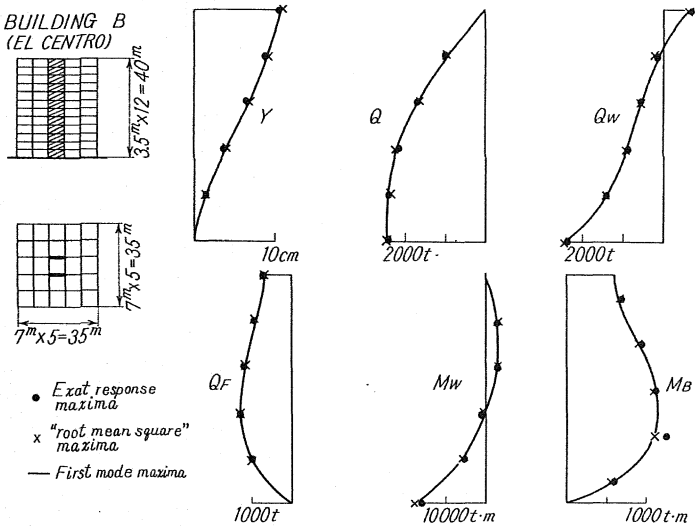


Fig. 9 Comparison of maximum response values (3)

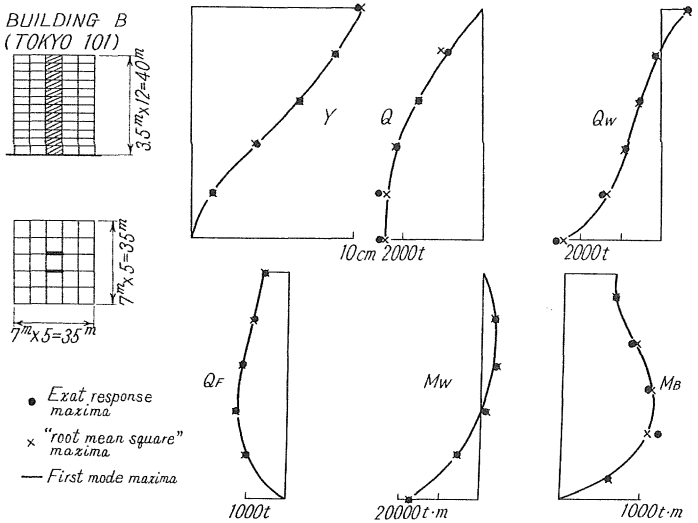


Fig. 10 Comparison of maximum response values (4)

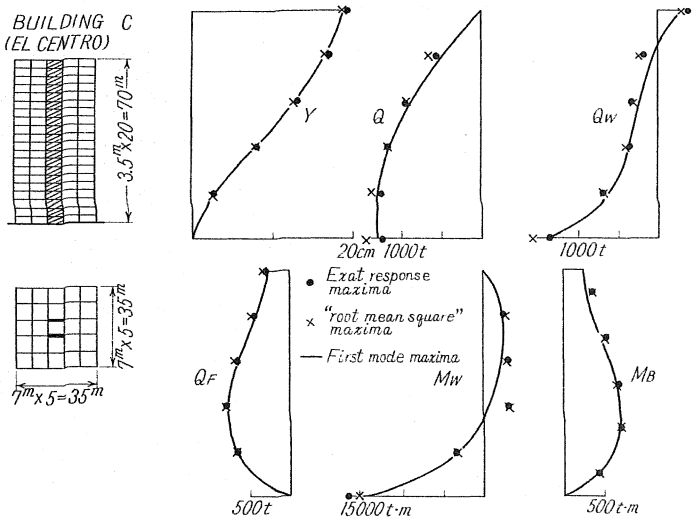


Fig. 11 Comparison of maximum response values (5)

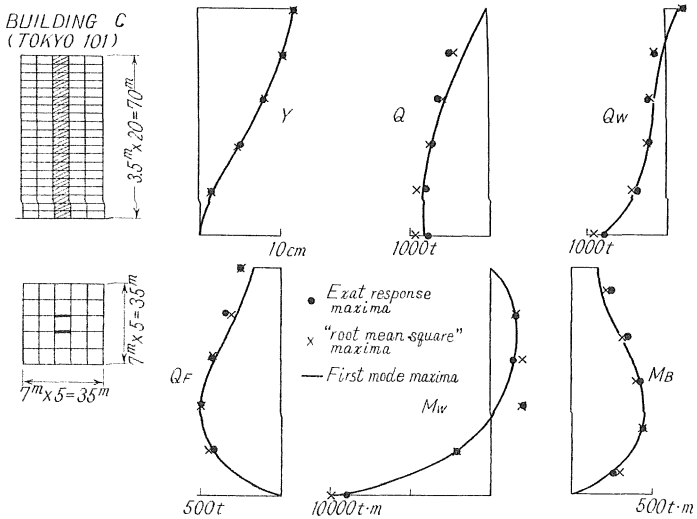


Fig. 12 Comparison of maximum response values (6)

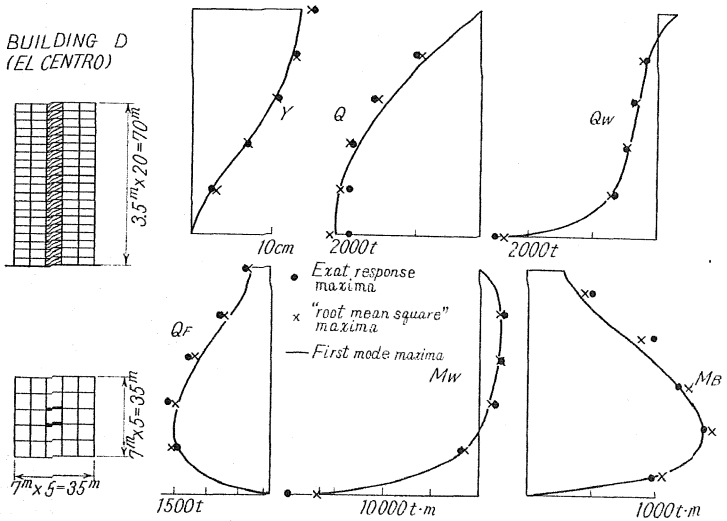


Fig. 13 Comparison of maximum response values (7)

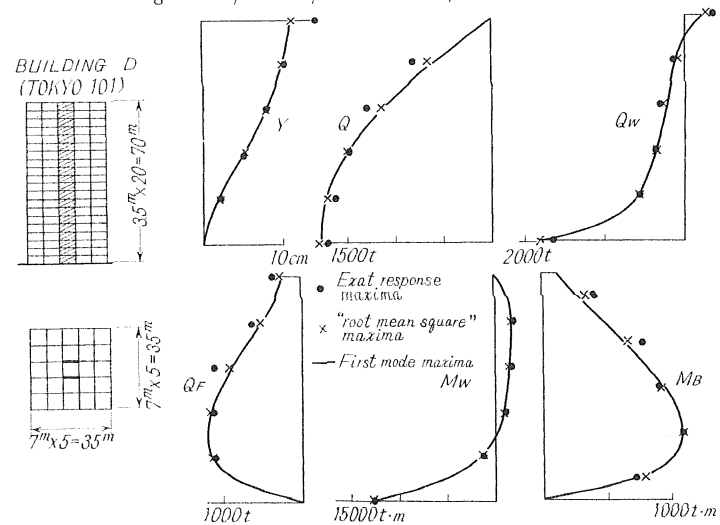


Fig. 14 Comparison of maximum response values (8)

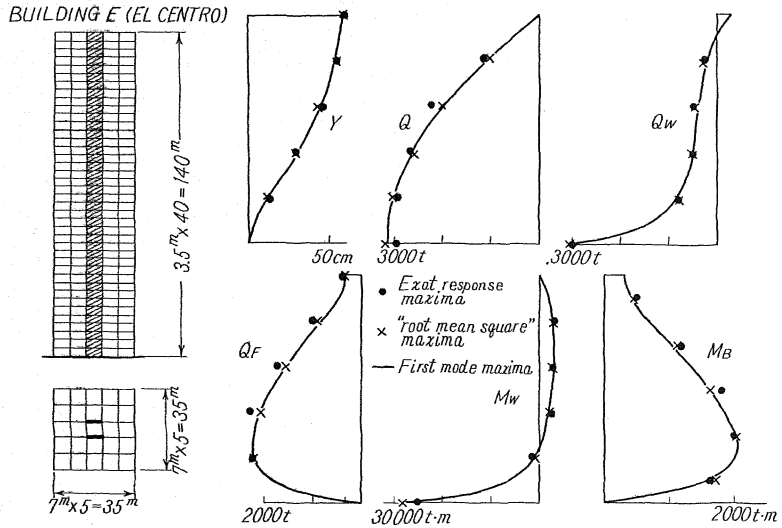


Fig. 15 Comparison of maximum response values (9)

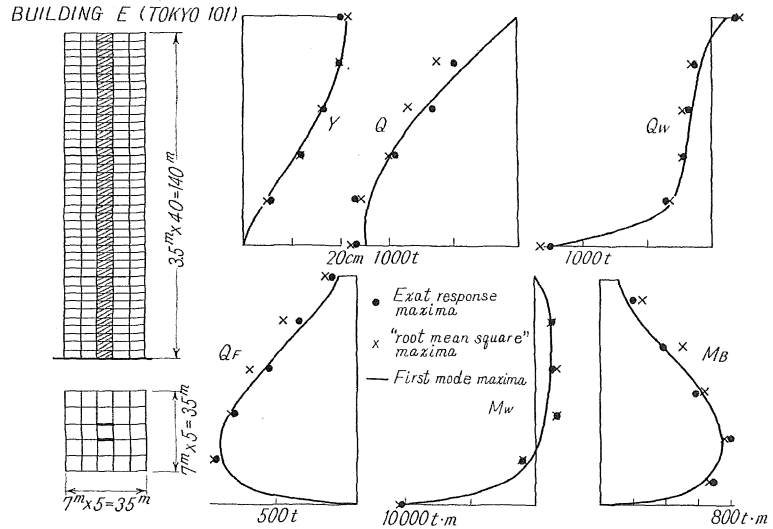


Fig. 16 Comparison of maximum response values (10)

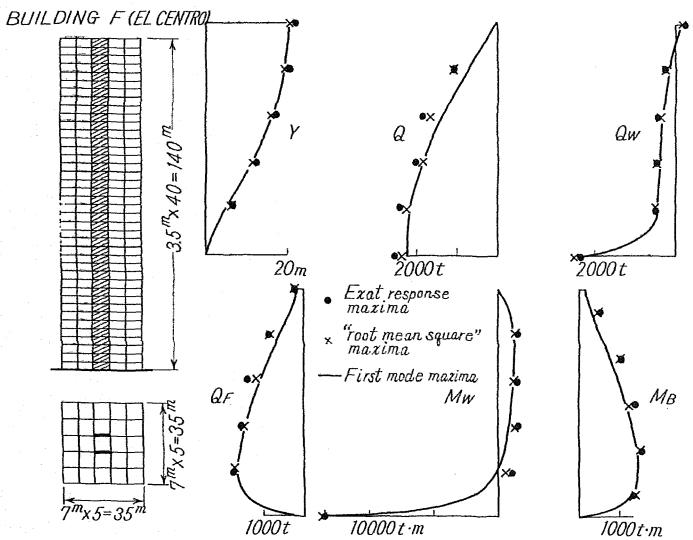


Fig. 17 Comparison of maximum response values (11)

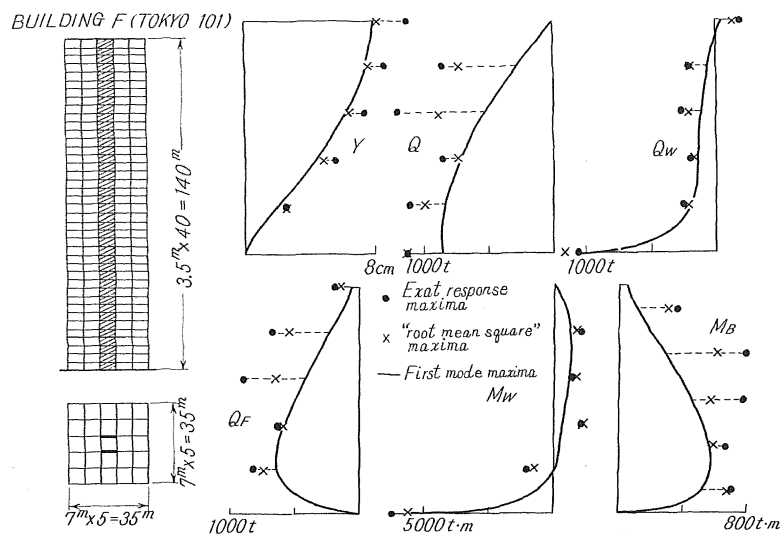


Fig. 18 Comparison of maximum response values (12)

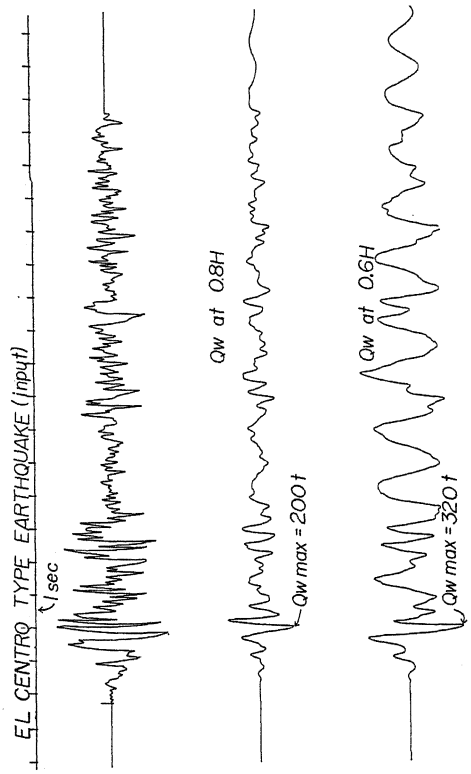
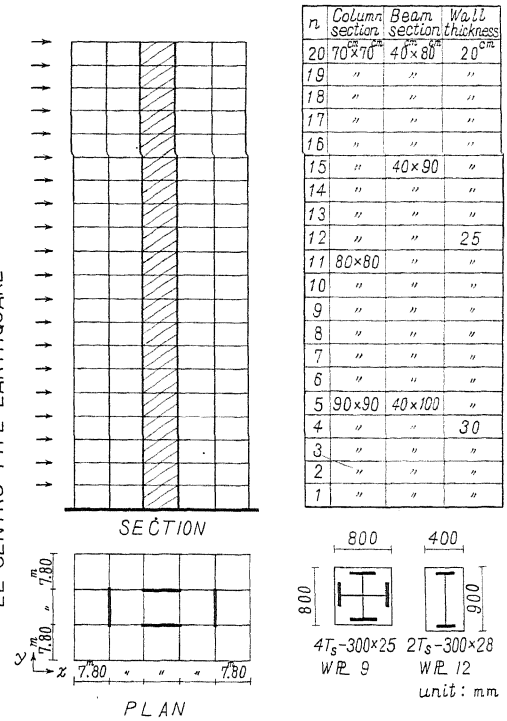


Fig.6 Illustration of responses obtained by analog computer: shear responses of core-wall in BUILDING C to EL CENTRO TYPE EARTHQUAKE



n	Column section	Beam section	Wall thickness
20	70×70 cm	40×80 cm	20 cm
19	〃	〃	〃
18	〃	〃	〃
17	〃	〃	〃
16	〃	〃	〃
15	〃	40×90	〃
14	〃	〃	〃
13	〃	〃	〃
12	〃	〃	25
11	80×80	〃	〃
10	〃	〃	〃
9	〃	〃	〃
8	〃	〃	〃
7	〃	〃	〃
6	〃	〃	〃
5	90×90	40×100	〃
4	〃	〃	30
3	〃	〃	〃
2	〃	〃	〃
1	〃	〃	〃

Fig. 19 Plan, section and constants of a sample building

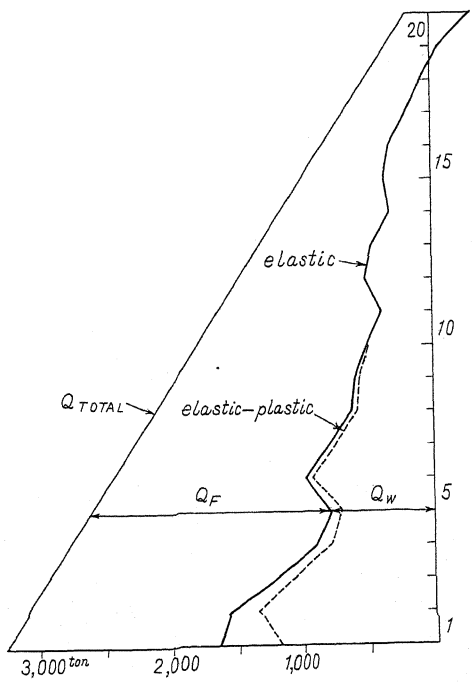


Fig. 20 Shear distribution

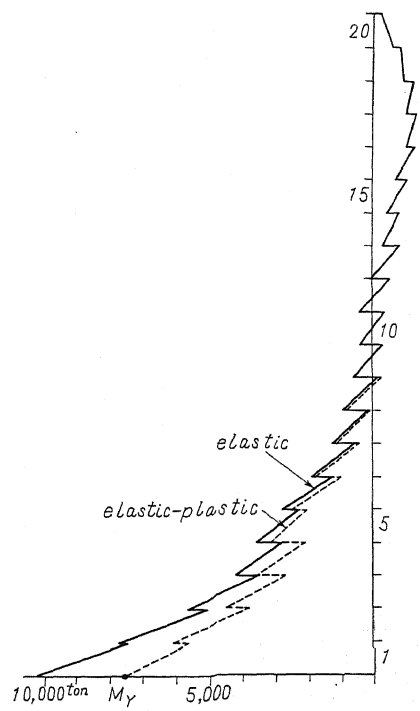


Fig. 21 Moment distribution of core wall

E R R A T A

SEISMIC ANALYSIS OF CORE-WALL BUILDINGS

BY Y. OSAWA

PAGE 459: Equation 2; should read

$$M_w(x) = -EI\frac{\partial^2 y}{\partial x^2} = -\int_x^H Q_w(x)\,dx' + \int_x^H K_B \frac{\partial y}{\partial x}\,dx'$$

PAGE 463: Equation 17; should read

$$\left[\tfrac{2}{3} + 2\left(\frac{l_a}{l}\right) + 2\left(\frac{l_a}{l}\right)^2\right] K_B$$

PAGE 465: Equation 33; should read

$$B_n = \frac{1}{1 + \Sigma D_{cn}/S_n K_{wn}}$$

PAGE 465: Equation 35; should read

$$(n=2)\quad K_{W2}(2-3B_2)\varphi_3 + \left[K_{W2}(4-3B_2) + K_{W1}(4-3B_1) + 6K_{Be2}\right]\varphi_2 = B_2 Q_2 h_2/S_2 + B_1 Q_1 h_1/S_1$$

PAGE 466: Equation 38; should read

$$\psi_{Mn} = -\frac{B_n}{2}\left(3\varphi_n + 3\varphi_{n+1} + \frac{Q_n h_n}{S_n K_{wn}}\right)$$

SEISMIC ANALYSIS OF CORE-WALL BUILDINGS

BY Y. OSAWA

QUESTION BY: N.M. NEWMARK - U.S.A.

Do you happen to have any comparisons made with the sum of the modal maxima rather than the root mean square for a comparative purpose? Do you recall what the difference was between the results you would get instead of taking the square of the sum of the squares you take the sum of the absolute value of the model responses. Were they much larger?

AUTHOR'S REPLY: Most of the sum of the modal maxima (absolute values) is about 5 to 30% larger than the exact maximum value, and the difference in much larger (around 50%) (1) for shears and moments in most places in building C and E, and (2) for shear in the upper part of the wall and for moment in the middle part of the wall in most sample buildings.

QUESTION BY: J. KRISHNA - INDIA.

In one of the diagrams elasto-plastic redistribution has occurred only in certain storeys and not in others. Was it because the upper storeys were over-designed or was there some other reason?

AUTHOR'S REPLY: In this example the plastic deformation was considered only at the bottom of the wall because the upper stories were over-designed, and the effect of the plastic deformation at the bottom of the wall produced the negligibly small amount of shear and moment in the upper stories.

QUESTION BY: C.C. CRAWFORD - U.S.A.

In the root mean square maxima is that the maxima of all the modes or just a number of them?

AUTHOR'S REPLY: The first three modes were considered in this analysis because the effect of other modes was considered to be very small.