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EARTHQUAKE RESISTANT DESIGN OF BUILDINGS - LESSONS FROM RECENT EARTHQUAKES*

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The major problem of earthquake resistant design is often understood to be one of structural engineering in as much as vibration studies are made of an individual structure to estimate its period of vibration and damping. A seismic coefficient is then decided upon taking into account these dynamic characteristics of the structure and also an expected earthquake motion at the site The structure is then designed in the usual manner. Recently some thought has been given to the effects of foundation characteristics on the behaviour of structures subjected to earthquake motion. There are two facets to the problem, one relating to the soil properties as they are under dynamic loads and the other to the changes in the soil properties due to earthquake, and the consequent effects of these changes on the structures standing on it.

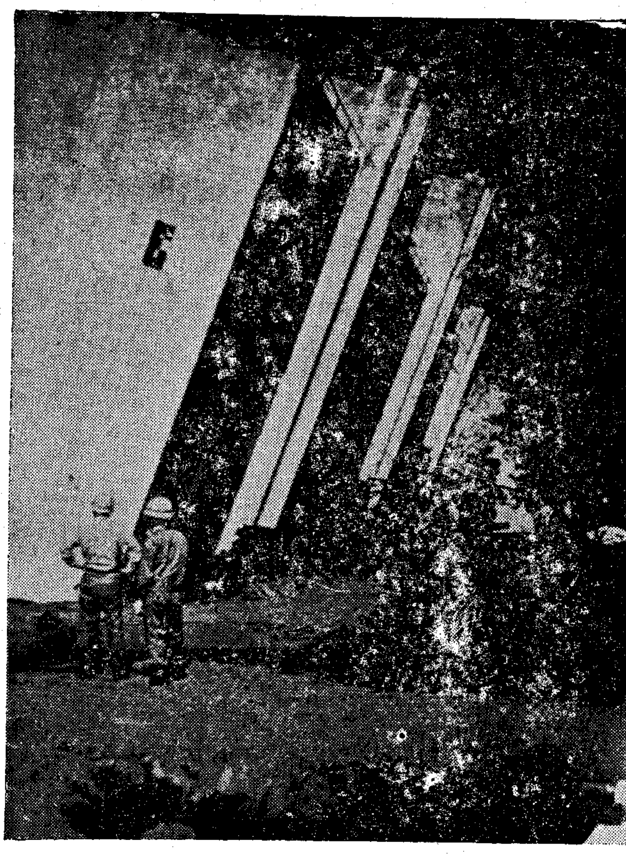

Fig. 1

If the soil is firm and does not yield plastically under the pounding of the structure caused by horizontal components of the earthquake force, the energy absorbed by it is only through elastic strains and is small compared with the energy supplied. But if the soil undergoes plastic deformation considerably, more energy is absorbed by it. This makes the structure standing on it safer against earthquakes. At the same time this yielding of soil increases the natural period of the structure and therefore has a lower response to the earthquakes provided that the natural period is greather than about 1/3 of a second For stiffer structures small yie ding does not make much difference. There has, however, to be a certain limit to this yielding and the yielding should be more or less uniform under the

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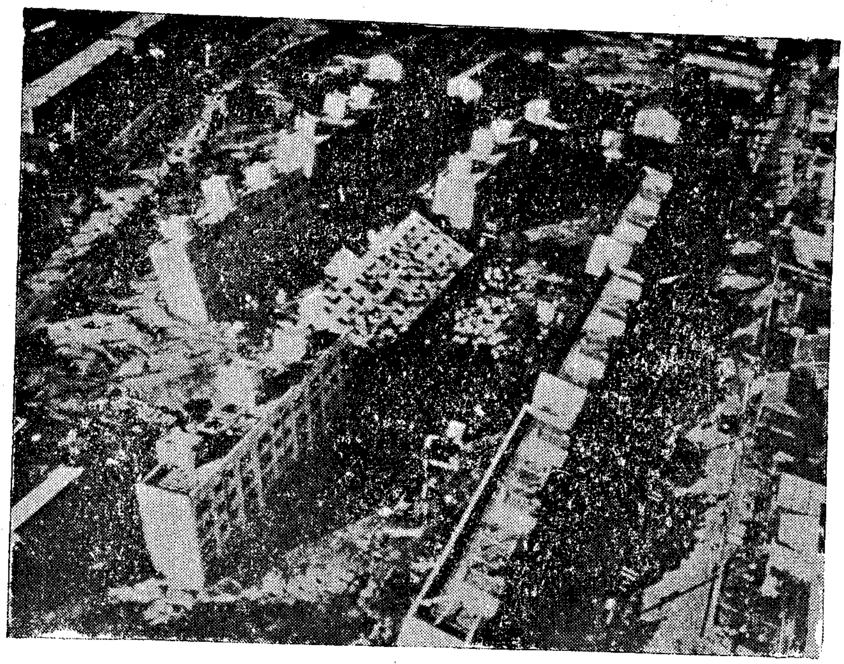


Fig. 2

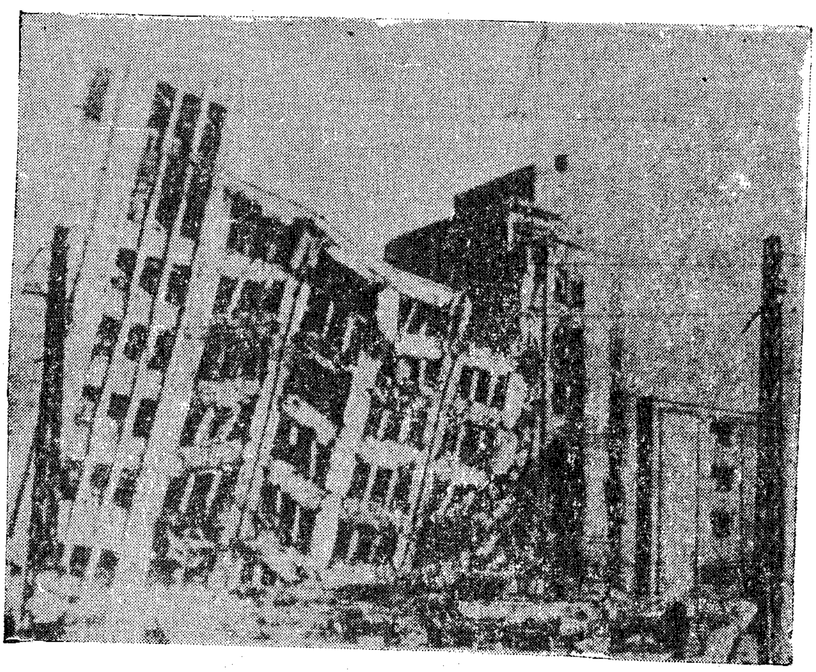
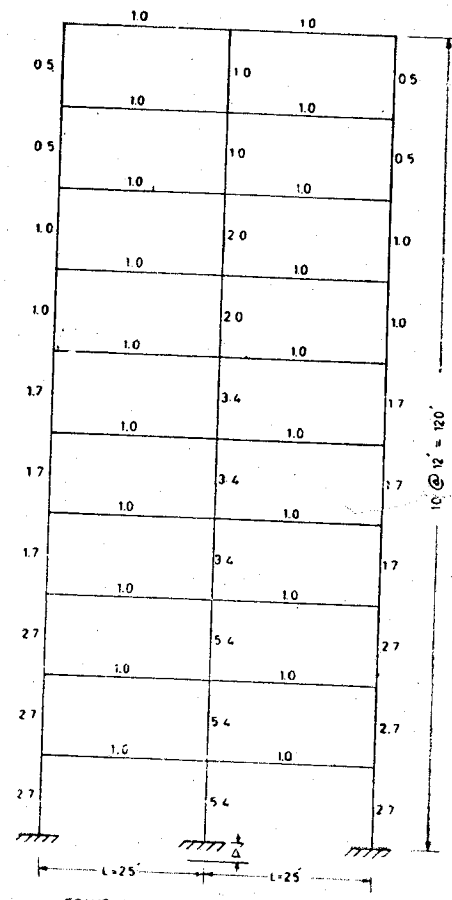


Fig. 3



FRAME CONSIDERED FOR ANALYSIS

Fig. 4

entire structure so that the stresses due to unequal settlement do not occur, These ideas are beginning to be considered in practical design only now. When settlements are large, or difference in settlements of various parts is more than a couple of inches the stresses caused in the structures are very large.

The classical example of the behaviour of buildings designed for resisting heavy earthquakes and with large factor of safety was exhibited in the Niagata earthquake of Japan in 1964. Fig 1 and 2 show how the foundation failed to support them. Buildings after buildings leaned to one side ranging from a few degrees to about 60° to the vertical during the earthquake. It can easily be worked out that this inclination means a force of 0.866 g acting

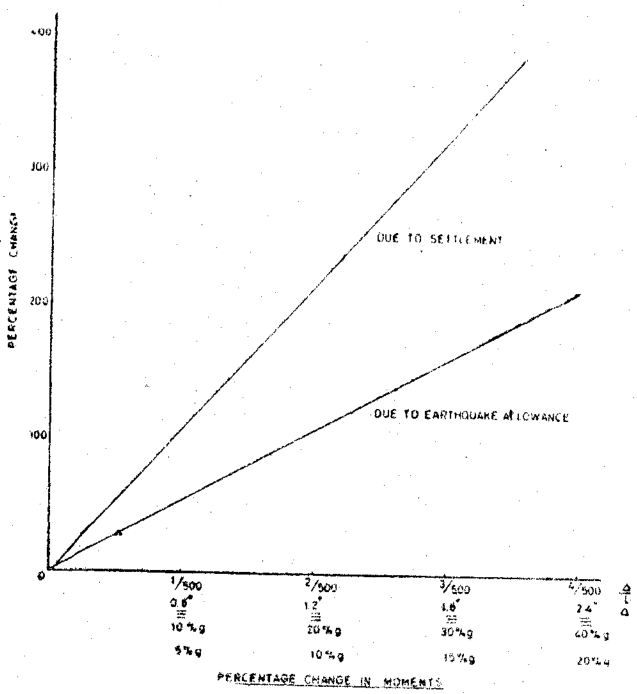


Fig. 5

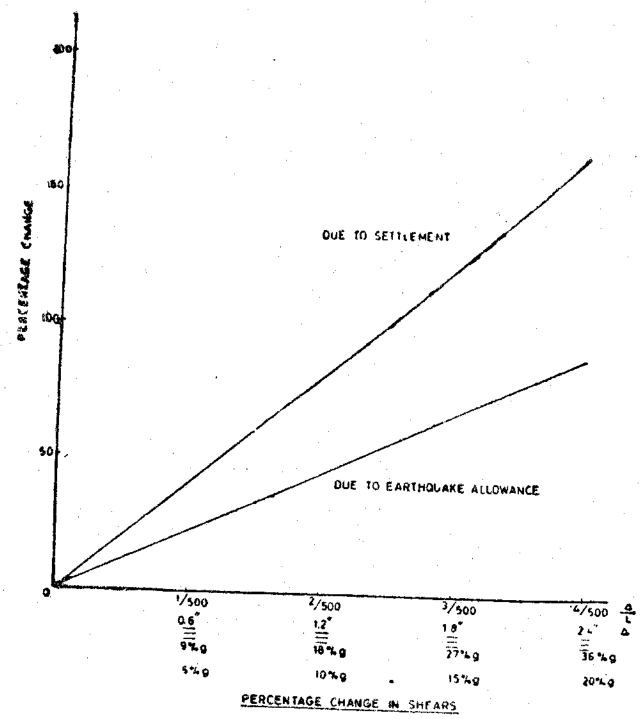


Fig. 6

normal to the face of the building. Even in Japan, a building is ordinarily designed for a horizontal force of not more than 0.20 g, and that provision was sufficient for the Niagata earthquake. If these buildings were not overdesigned and did not have an unduly high factor of safety, they should have failed structurally also, but the failure actually came through the yielding of foundation soil. The structural failure did not take place because the buildings consisted of reinforced concrete boxes well stiffened and heavily reinforced. Such an expensive design is beyond our means. It would have been much cheaper to design the foundation more adequately.

Foundation failure was also the main cause of failure of buildings during the Alaska earthquake of 1964 Since the buildings in this case were designed only for wind forces, they collapsed due to the foundation failure even though they consisted of reinforced concrete frames. They were unable to stand unequal settlements The failure of the building of the newly constructed department store is shown in figure 3. The unequal settlement in this building was 25". The foundation in all these cases was the usual shallow footings. Obviously a strong raft or piles would have been a more suitable alternative.

It will be seen from figures. 4 to 6 that if a building is subjected to unequal settlement, the rate of increase of moments and shears at critical junctions is much higher than on account of horizontal forces. It will be clear, the refore, that the failure will be more from unequal settlement than from horizontal earthquake excitation.

The future trends in earthquake engineering studies are thus a study of the behaviour of soil under dynamic forces. Highly sophisticated structural analyses are essential for studying the effects of drift and other factors in very tall structures, but the basic question of foundation design is far more important and overshadows the structural studies. That indicates the direction of future work.

I = area moment of inertia of section
A = cross-sectional area
γ = weight per unit volume
k = numerical shape factor for the cross section
K = coefficient of the foundation
y = total deflection
ψ = bending slope

Eliminating y or ψ from equations (1) and (2), one obtains the following two uncoupled equations in y and ψ

$$EI\frac{\partial^4\psi}{\partial x^4}-\frac{EIK}{kAG}\frac{\partial^2\psi}{\partial x^2}-\left(\frac{\gamma I}{g}+\frac{EI}{gk}\frac{\gamma}{G}\right)\frac{\partial^4\psi}{\partial x^2\partial t^2}+\left(\frac{\gamma A}{g}+\frac{I}{gk}\frac{\gamma K}{AG}\right)\frac{\partial^2\psi}{\partial t^2}+\frac{\gamma I}{g}\frac{\gamma}{gkG}\frac{\partial^4\psi}{\partial t^4}+K\psi=0 \tag{3}$$

$$EI\frac{\partial^4 y}{\partial x^4}-\frac{EIK}{kAG}\frac{\partial^2 y}{\partial x^2}-\left(\frac{\gamma I}{g}+\frac{EI}{gk}\frac{\gamma}{G}\right)\frac{\partial^4 y}{\partial x^2\partial t^2}+\left(\frac{\gamma A}{g}+\frac{I\gamma}{gk}\frac{K}{AG}\right)\frac{\partial^2 y}{\partial t^2}+\frac{\gamma I}{g}\frac{\gamma}{gkG}\frac{\partial^4 y}{\partial t^4}+Ky=0 \tag{4}$$

The shear slope, moment and shear are given by:

$$\text{Shear Slope}; \ \phi(x,t) = \frac{\partial y}{\partial x}-\psi \tag{5}$$

$$\text{Moment}; \ M(x,t) = -EI\frac{\partial\psi}{\partial x} \tag{6}$$

$$\text{Shear}; \ Q(x,t) = kAG\left(\frac{\partial y}{\partial x}-\psi\right) \tag{7}$$

For the simplest end configurations, the boundary conditions are the following:

$$\text{Hinged End}; \ y = 0 \text{ and } \frac{\partial\psi}{\partial x} = 0 \tag{8}$$

$$\text{Clamped End}; \ y = 0 \text{ and } \psi = 0 \tag{9}$$

$$\text{Free End}; \ \frac{\partial\psi}{\partial x} = 0 \text{ and } \frac{\partial y}{\partial x}-\psi = 0 \tag{10}$$

Frequency Equations and Modal Forms

Let us take the solutions of equations (1) to (4) in the form

$$y(x,t) = Y(\xi)\,e^{ipt} \tag{11}$$

$$\psi(x,t) = \varphi(\xi)\,e^{ipt} \tag{12}$$

with

$$\xi = x/L \tag{13}$$

where

$i = \sqrt{-1}$
p = angular frequency (restricted to real number)
L = length of the beam

Omitting the factor e^{ipt},

$$s^2 \varphi'' - (1 - b^2 r^2 s^2)\varphi + \frac{Y'}{L} = 0 \quad (14)$$

$$Y'' + s^2 (b^2 - q) Y - L\varphi' = 0 \quad (15)$$

$$\varphi^{iv} + [b^2 (r^2 + s^2) - s^2 q]\varphi'' - [b^2 (1 - b^2 r^2 s^2) - q(1 - b^2 r^2 s^2)]\varphi = 0 \quad (16)$$

$$Y^{iv} + [b^2 (r^2 + s^2) - s^2 q] Y'' - [b^2 (1 - b^2 r^2 s^2) - q(1 - b^2 r^2 s^2)] Y = 0 \quad (17)$$

where

$$b^2 = \frac{1}{EI}\frac{\gamma A}{g} L^4 p^2 \quad (18)$$

$$r^2 = \frac{I}{AL^2} \quad (19)$$

$$s^2 = \frac{EI}{kAG\,L^2} \quad (20)$$

$$q = \frac{KL^4}{EI} \quad (21)$$

The dimensionless parameter b is directly related to the frequencies of vibration, p. The dimensionless parameters r, s, and q are measures of the effects of rotatory inertia, shear deformation, and the elastic foundation, respectively. Solutions of equations (16) and (17) may be found to be

$$Y = C_1 \cosh \alpha\xi + C_2 \sinh \alpha\xi + C_3 \cos \beta\xi + C_4 \sin \beta\xi \quad (22)$$

$$\varphi = C'_1 \cosh \alpha\xi + C'_2 \sinh \alpha\xi + C'_3 \cos \beta\xi + C'_4 \sin \beta\xi \quad (23)$$

where

$$\alpha = \frac{1}{\sqrt{2}}\left[\left\{[b^2 (r^2 - s^2) + s^2 q]^2 + 4(b^2 - q)\right\}^{1/2} - \left\{b^2 (r^2 + s^2) - s^2 q\right\}\right]^{1/2} \quad (24)$$

$$\beta = \frac{1}{\sqrt{2}}\left[\left\{[b^2 (r^2 - s^2) + s^2 q]^2 + 4(b^2 - q)\right\}^{1/2} + \left\{b^2 (r^2 + s^2) - s^2 q\right\}\right]^{1/2} \quad (25)$$

The eight constants in equations (22) and (23) are not all independent, but are related by the equations (14) or (15) as follows :

$$C'_1 = \left\{\frac{\alpha^2 + s^2 (b^2 - q)}{L\alpha}\right\} C_2 \quad (27)$$

$$C'_2 = \left\{\frac{\alpha^2 + s^2 (b^2 - q)}{L\alpha}\right\} C_1 \quad (28)$$

$$C'_3 = -\left\{\frac{s^2 (b^2 - q) - \beta^2}{L\beta}\right\} C_4 \quad (29)$$

$$C'_4 = \left\{\frac{s^2 (b^2 - q) - \beta^2}{L\beta}\right\} C_3 \quad (30)$$

It should be noted that the solutions in equations (22) and (23) apply only under the the conditions

$$[b^2 (r^2 - s^2) + s^2 q]^2 + 4(b^2 - q) > 0 \quad (26a)$$

and

$$\{[b^2 (r^2 - s^2) + s^2 q]^2 + 4(b^2 - q)\}^{1/2} \pm [b^2 (r^2 + s^2) - s^2 q] > 0 \quad (26b)$$

Inequality (26a) is essentially a requirement that α and β shall not be complex. If inequality (26a) is violated, frequencies for the boundary conditions given later become complex. Since only real frequencies are physically of interest, b will always be chosen such that both the inequality (26a) and the frequency equation for the particular boundary condition are satisfied.

With b chosen so that (26a) is satisfied, (26b) requires that α and β be real. Violation of (26b) means that either α or β becomes a pure imaginary. (The forced satisfaction of (26a) precludes α and β becoming imaginary simultaneously). It is still possible, however, to have real frequencies, even with α or β imaginary. Suppose that either α or β becomes imaginary; equations (24) and (25) then become

$$\alpha = \frac{i}{\sqrt{2}}\left[-\left\{[b^2(r^2-s^2)+s^2q]^2+4(b^2-q)\right\}^{1/2}+\left\{b^2(r^2+s^2)-s^2q\right\}\right]^{1/2} = i\,\alpha' \qquad (24a)$$

or

$$\beta = \frac{i}{\sqrt{2}}\left[-\left\{[b^2(r^2-s^2)+s^2q]^2+4(b^2-q)\right\}^{1/2}+\left\{b^2(r^2+s^2)-s^2q\right\}\right]^{1/2} = i\,\beta' \qquad (25a)$$

Substitution of either α or β according to (24a) and (25a) in equations (22) and (23) then gives the solutions in terms of the real α' and β' :

For α imaginary

$$Y = C_1 \cos\alpha'\xi + C_2 \sin\alpha'\xi + C_3\cos\beta\xi + C_4\sin\beta\xi \qquad (22a)$$

$$\varphi = C'_1 \cos\alpha'\xi + C'_2 \sin\alpha'\xi + C'_3\cos\beta\xi + C'_4\sin\beta\xi \qquad (23a)$$

and for β imaginary

$$Y = C_1 \cosh\alpha\xi + C_2 \sinh\alpha\xi + C_3\cosh\beta'\xi + C_4\sinh\beta'\xi \qquad (22b)$$

$$\varphi = C'_1 \cosh\alpha\xi + C'_2 \sinh\alpha\xi + C'_3\cosh\beta'\xi + C'_4\sinh\beta'\xi \qquad (23b)$$

where again the eight constants of (22a) and (23a), as well as (22b) and (23b), are related through equations similar to equations (27 — 30).

For the homogeneous boundary conditions described by equations (8), (9) and (10), only the ratios of constants C_1, C_2, C_3 and C_4 or C'_1, C'_2, C'_3, and C'_4 are determinate. Application of any combination of the boundary conditions leads to the characteristic frequency equation. The roots of this equation are the frequency values, b, for which a nontrivial solution is valid. For each value of b, there is a corresponding natural mode.

The frequency equations and natural modes are given below for six common combinations of the boundary conditions (8), (9) and (10). In all six cases it has been assumed that the solutions to the differential equations (16) and (17) are those given in equations (22) and (23). As mentioned above, however, violation of inequality (26b) will change the form of the solution to that given in (22a) and (23a); the α and β of equations (24a) and (25a) should then be substituted into the frequency and mode equations given below to obtain the real frequencies and modes. An example of this substitution for the fixed-free beam, Case 5, is contained in the numerical problem. Frequency equations and natural modes are now given for the followinng six support conditions.

Case (1): Clamped at $\xi=0$ and $\xi=1$.

Frequency equation is

$$\sin\beta = 0 \qquad (31)$$

and natural modes are

$$Y=\sin \beta \xi \tag{32}$$

$$\varphi=\cos \beta \xi \tag{33}$$

Case (2) : Hinged at $\xi=0$ and $\xi=1$.

The frequency equation is

$$2-2 \cosh \alpha \cos \beta-\left(\frac{\eta^2-\zeta^2}{\eta\zeta}\right)\sinh \alpha \sin \beta=0 \tag{34}$$

and the natural modes are

$$Y=\cosh \alpha\xi-\cos \beta\xi-\left\{\frac{\cosh \alpha-\cos\beta}{\zeta \sinh \alpha+\eta\sin \beta}\right\}(\zeta \sinh \alpha\xi+\eta \sin \beta\xi) \tag{35}$$

$$\varphi=\cosh \alpha\xi-\cos \beta\xi-\left\{\frac{\cosh \alpha-\cos \beta}{\eta \sinh \alpha-\zeta\sin \beta}\right\}(\eta \sinh \alpha\xi-\zeta \sin \beta\xi) \tag{36}$$

where

$$\eta = \frac{(s^2 (b^2-q)+\alpha^2)}{\alpha} \tag{37}$$

$$\zeta = \frac{\{s^2 (b^2-q)-\beta^2\}}{\beta} \tag{38}$$

Case (3) : Free at $\xi=0$ and $\xi=1$.

The frequency equation, in this case, is

$$2-2 \cosh \alpha \cos \beta-\left\{\frac{\zeta\beta^2}{\eta\alpha^2}-\frac{\eta\alpha^2}{\zeta\beta^2}\right\}\sinh \alpha \sin \beta=0 \tag{39}$$

The natural modes are

$$Y=\zeta\beta \cosh \alpha\xi-\eta\alpha \cos \beta\xi-\left\{\frac{\eta\alpha\, \zeta\beta (\cosh \alpha-\cos \beta)}{\eta\alpha^2 \sinh \alpha-\xi\beta^2 \sin \beta}\right\}\cdot(\alpha \sinh \alpha\xi+\beta \sin \beta\xi) \tag{40}$$

$$\varphi=\eta\alpha \cosh \alpha\xi-\zeta\beta \cos \beta\xi-\left\{\frac{\eta\alpha^2 \sinh \alpha + \zeta\beta^2 \sin \beta}{\alpha\beta (\cosh \alpha - \cos \beta)}\right\}(\beta \sinh \alpha\xi - \alpha \sin \beta\xi) \tag{41}$$

Case (4) : Clamped at $\xi = 0$ and hinged at $\xi =1$.

The frequency equation is

$$\frac{\zeta}{\eta} \tanh \alpha + \tan \beta = 0 \tag{42}$$

The natural modes are

$$Y=\cosh \alpha\xi-\cos \beta\xi-\left\{\frac{\eta\alpha \cosh \alpha - \zeta\beta \cos \beta}{\eta\zeta(\alpha \sinh \alpha+ \beta \sin \beta)}\right\}(\zeta \sinh \nu\zeta + \eta \sin \beta\xi) \tag{43}$$

$$\varphi= \eta \sinh \alpha\xi - \zeta \sin \beta\xi - \left\{\frac{\eta \sinh \alpha - \zeta \sin \beta}{\cosh \alpha - \cos \beta}\right\}(\cosh \alpha\xi - \cos \beta\xi) \tag{44}$$

Case (5) : Clamped at $\xi = 0$ and free at $\xi = 1$.

The frequency equation is

$$\left\{\frac{\alpha\eta}{\beta\zeta}+\frac{\beta\zeta}{\alpha\eta}\right\} \cosh \alpha \cos \beta + \left(\frac{\beta^2 - \alpha^2}{\alpha\beta}\right) \sinh \alpha \sin \beta - 2=0 \tag{45}$$

The natural modes are

$$Y = \cosh \alpha\xi - \cos \beta\zeta - \left\{\frac{\beta \sinh \alpha - \alpha \sin \beta}{\zeta\beta \cosh \alpha - \alpha\eta \cos \beta}\right\} (\zeta \sinh \alpha\xi + \eta \sin \beta\xi) \tag{46}$$

$$\varphi = \cosh \alpha\xi - \cos \beta\xi - \left\{\frac{\alpha \sinh \alpha + \alpha \sin \beta}{\alpha\eta \cosh \alpha - \beta\zeta \cos \beta}\right\} (\eta \sinh \alpha\xi - \zeta \sin \beta\xi) \tag{47}$$

Cose (6) : Hingeg at $\xi=0$ and free at $\xi=1$.

The frequency equation is

$$\frac{\eta\alpha^2}{\zeta\beta^2} \tanh \alpha + \tan \beta = 0 \tag{48}$$

The natural modes are

$$Y = \alpha \sinh \alpha\xi + \beta \sin \beta\xi + \left\{\frac{\eta\alpha^2 \sinh \alpha + \zeta\beta^2 \sin \beta}{\eta\zeta\, \alpha\beta\, (\cosh \alpha - \cos \beta)}\right\} (\zeta\beta \cosh \alpha\xi - \eta\sigma \cos \beta\xi) \tag{49}$$

$$\varphi = \eta\alpha \cosh \alpha\xi - \zeta\beta \cos \beta\xi + \left\{\frac{\eta\alpha^2 \sinh \alpha + \zeta\beta^2 \sin \beta}{\alpha\beta\, (\cosh \alpha - \cos \beta)}\right\} \beta \sinh \alpha\xi - \alpha \sin \beta) \tag{50}$$

Numerical Example

Inspection of the frequency equations (31), (34), (39), (42), (45) and (48) for the six common combinations of boundary conditions presented above indicates that considerable study and labor is entailed in the solution of each of the equations if roots are to be computed for various combinations of the parameters s, r, and q. Further, the desirability of a graphical presentation of these roots is evident, if the quantitative effects of shear, rotatory inertia, and elastic foundation are to be easily comprehended.

As a particular numerical example, the first four frequencies for a clamped-free beam (Case 5) have been computed from the frequency equation (45). A ratio of $E/G=8/3$ and a shape factor $k = 2/3$ have been assumed. Under these assumptions, $s = 2r$; and the variables α, β, η, and ζ become functions of the rotatory inertia parameter, r, and the foundation parameter q. Accordingly, the frequency equation (45) and its associated roots, b, are functions of these same two parameters, r and q.

In terms of r, q, and b the frequency equation becomes

$$\left\{\frac{4r^2 (b^2-q) + \alpha^2}{4r^2 (b^2-q) + \beta^2} + \frac{4r^2 (b^2-q) - \beta^2}{4r^2 (b^2-q) + \alpha^2}\right\} \cosh \alpha \cos \beta + \left(\frac{\beta^2 - \alpha^2}{\alpha\beta}\right) \sinh \alpha \sin \beta - 2 = 0 \tag{51}$$

where α and β are derived from equations (24) and (25) with the condition

$s = 2r$:

$$\alpha = [\tfrac{1}{2} \{r^4 (3b^2-4q)^2 + 4(b^2-q)\}^{1/2} - \tfrac{1}{2} r^2(5b^2-4q)]^{1/2} \tag{52}$$

$$\beta = [\tfrac{1}{2} \{r^4 (3b^2-4q)^2 - 4(b^2-q)\}^{1/2} + \tfrac{1}{2} r^2(5b^2-4q)]^{1/2} \tag{53}$$

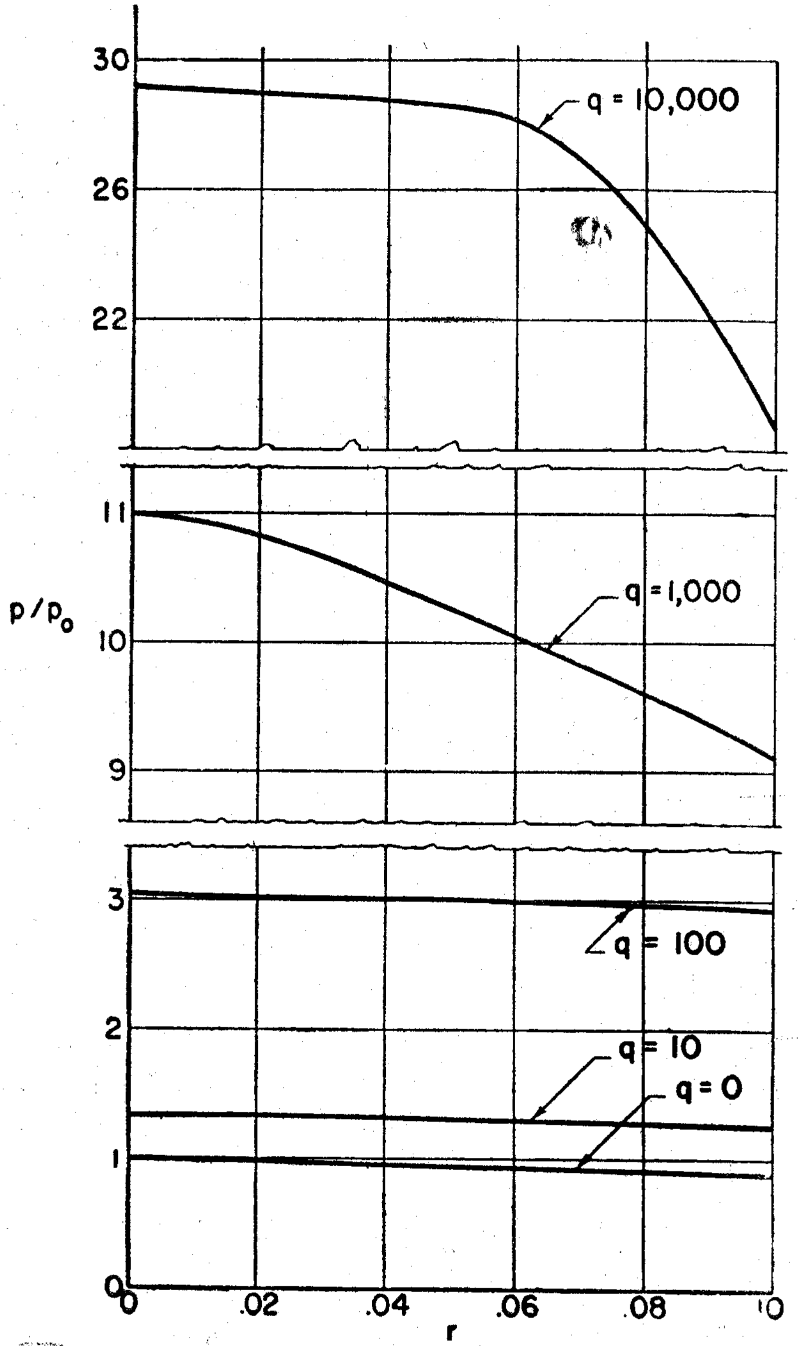

30
26
22
q = 10,000
11
10
9
q = 1,000
p / p_o
3
2
1
0
q = 100
q = 10
q = 0
0
.02
.04
.06
.08
.10
r

Fig. 1

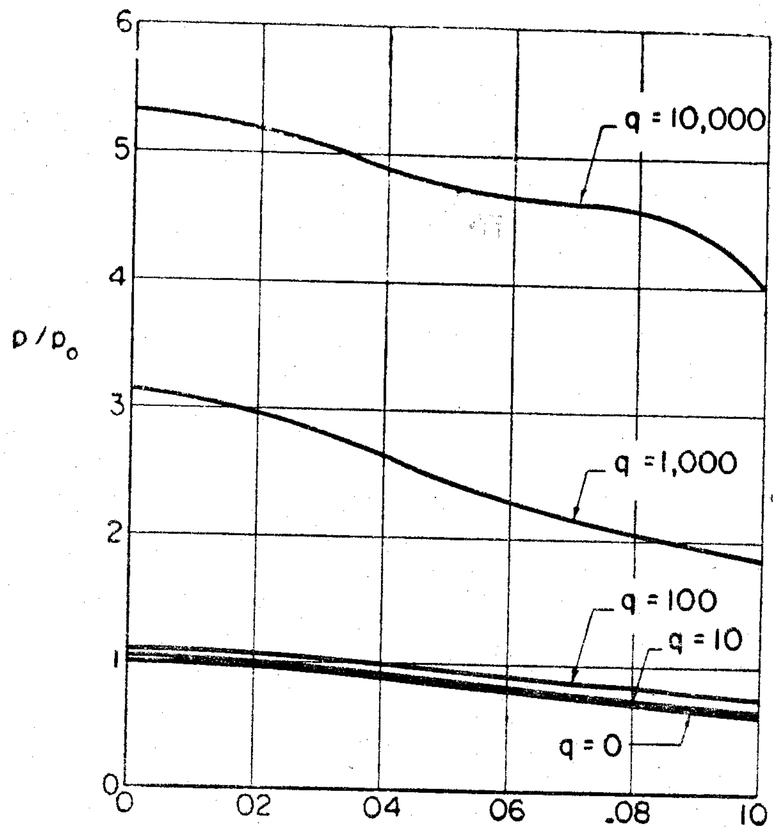


Fig. 2

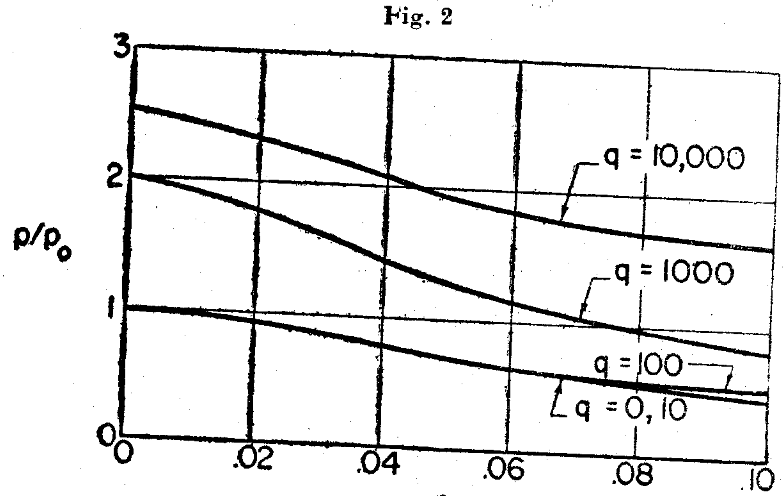


Fig. 3

Equation (51) and the associated equations (52) and (53) form the basis for the solutions presented in figures 1 — 4. As has already been noted, the frequency equation (51) is valid only when a and β are real. It is quite conceivable, however, that a or β could become a pure imaginary function for some values of the parameters r and q. [The case of a and β complex is excluded on the basis of the comments following (26a)]. When a or β is imaginary it is then necessary to make the substitution given in one of the equations (24a) or (25a) in the frequency equation (53). The frequency equation then assumes the following froms:

For a imaginary

$$\left[\frac{4r^2(b^2-q)-a'^2}{4r^2(b^2-q)-\beta^2}+\frac{4r^2(b^2-q)-\beta^2}{4r^2(b^2-q)-a'^2}\right]\cos a'\cos\beta+\frac{\beta^2+a'^2}{a'\beta}\sin a'\sin\beta-2=0 \qquad (54)$$

and for β imaginary

$$\left[\frac{4r^2(b^2-q)+a^2}{4r^2(b^2-q)+\beta'^2}+\frac{4r^2(b^2-q)+\beta'^2}{4r^2(b^2-q)+a^2}\right]\cosh a\cosh\beta'-\left(\frac{a^2+\beta'^2}{a\beta'}\right)\sinh a\sinh\beta'-2=0 \qquad (55)$$

It isinteresting to note that for any one set of the parameters r and q, it is not immediately obvious that there are no roots with β imaginary. For the range of parameters studied, however, no roots were found with β imaginary.

In each of the figures 1 to 4 p_0 is the frequency of a fixed-free beam *in vacuo* from classical theory, the corresponding values of which are

p_0=3.52, 22.03, 61.70, 120.91

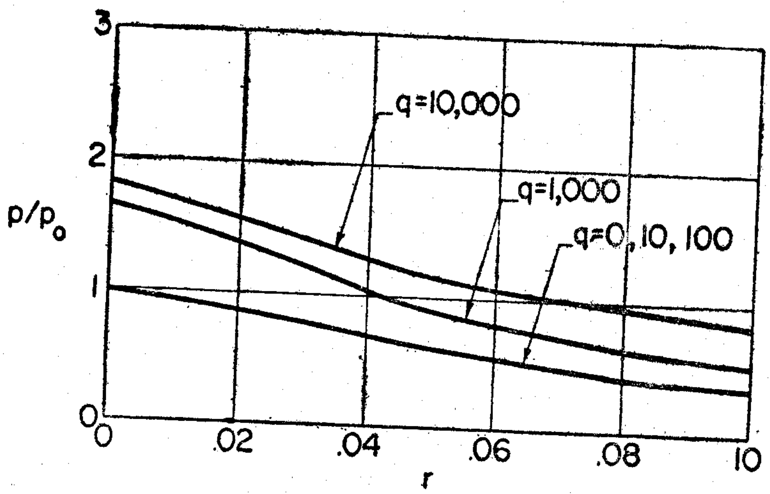


Fig. 4

These figures are the graphical representation of p/p_0 versus r for values of q = 0, 10, 10^2, 10^3 and 10^4 for the first four modes of a finite fixed-free beam on an elastic foundation with the range of r from 0 to .10. It is seen that the effect of shear deformation and rotatory inertia increases with the increasing foundation modulus and depth of the beam. Similar conclusion is reached by Tseitlin (15) in his study of beams of infinite length on any elastic foundation. The effect of foundation modulus, of course,decreases, whereas the effect of shear deformation and rotatory inertia increases for the higher modes.

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A STUDY OF A TWO STOREYED STEEL FRAME UNDER STATIC AND DYNAMIC LOADS

A.R. Chandrasekaran* and Krishen Kumar**

Synopsis

This paper describes tests on two storeyed mild steel framed structures. Static lateral hysteretic load deflection measurements were obtained at various strain levels. Also, free vibration tests were carried out by giving initial displacements at different strain levels. The framed structure was mounted on a shake table which was given steadystate sinusoidal motions at various frequencies. Finally, the frame was tested to failure. Theoretical computations of spring constants and periods were compared with those of experiments. Damping obtained from different techniques were compared with each other.

Introduction

A design criterion for structures located in seismic zones is to have elastic behaviour during small size shocks which might occur more often in the area and to permit plastic deformations for the few large size shocks. Therefore, in order to find the response of structures due to severe ground motion it is necessary to determine the restoring forces and dissipative forces of systems under cyclic loads subjected to deformations beyond yield level.

The amount of experimental data available concerning hysteretic load deflection relationships at high strain levels is meagre. Jacobsen[1] has reviewed thirteen references dealing with reversed cyclic loading beyond the normal elastic conditions of joints, wood frames, built up beams and concrete frames, Hanson[2] has recently reported tests on a number

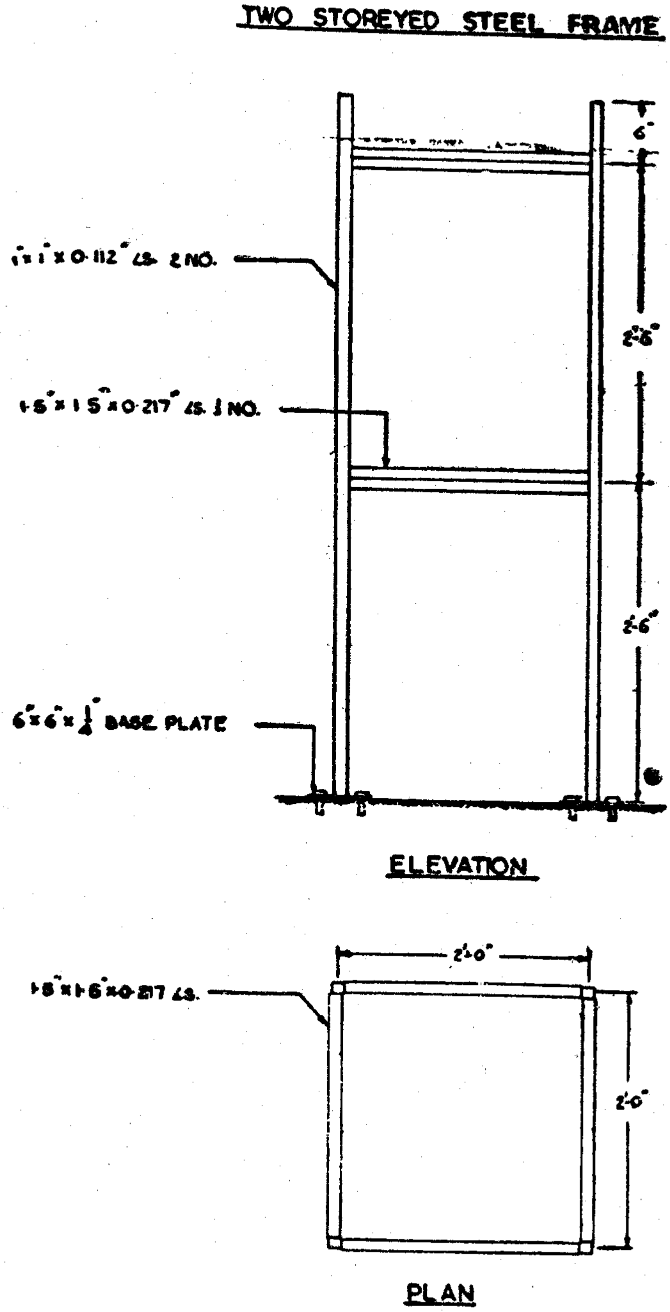

Fig. 1

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of single storeyed mild steel structures and showed that differences between the static and dynamic hysteretic load deflection curves were, in general, smaller than the changes in static curves caused by the deterioration of the material.

Experimental Structure

A two storeyed steel frame model, as shown in fig. 1, was used for tests. A head room of 2.30m was available over a 1.80m × 1.20m size shake table and the dimensions of the frame were chosen such that it could be conveniently accommodated on the table. The size of the columns were the minimum sections that could be fabricated from available angle sections 1″ × 1″ × 1/8″ size. The ratio of moment of inertia per unit length of beam to that of column was kept as four.

Theoretical Analysis of Model Frame

Theoretical analysis was carried out to determine the influence coefficients due to lateral loads and to predict the frequencies of vibrations and mode shapes. Also, from the measured lateral force deflection characteristics, hysteretic damping values have been calculated.

The mass of the structure has been considered to be concentrated at the storey levels by assuming half the weight of the columns between adjacent floors to be acting at the floor levels. Two different floor masses are used and the mass matrices [M], are as follows:—

(a) Without floor slabs

$$[M] = \begin{bmatrix} 0.148 & 0 \\ 0 & .318 \end{bmatrix} \times 10^{-2} \quad \text{Kg. sec.}^2/\text{mm} \qquad (1)$$

(b) With slabs placed on the frame at the two floor levels

$$[M] = \begin{bmatrix} 1.073 & 0 \\ 0 & 1.303 \end{bmatrix} \times 10^{-2} \quad \text{Kg. sec.}^2/\text{mm} \qquad (2)$$

Considering joint rotation, theoretical values of influence coefficients [G], and stiffness coefficients, [K], for lateral motion are as follows :—

$$[G] = \begin{bmatrix} 19.5 & 21.6 \\ 21.6 & 45.4 \end{bmatrix} \times 10^{-3} \quad \text{mm/kg.} \qquad (3)$$

$$[K] = \begin{bmatrix} 108.4 & -51.6 \\ -51.6 & 46.6 \end{bmatrix} \quad \text{Kg/mm} \qquad (4)$$

Using standard procedures, the undamped frequencies and mode shapes could be determined and they are as follows :

(a) Without slabs

$$f_1 = 12.55 \text{ c.p.s} \; ; \; f_2 = 45.52 \text{ c.p.s.} \qquad (5)$$

$$\{\Phi^{(1)}\} = \begin{Bmatrix} 1.000 \\ 1.922 \end{Bmatrix} \; ; \; \{\Phi^{(2)}\} = \begin{Bmatrix} 1.000 \\ -2.2414 \end{Bmatrix} \qquad (6)$$

(b) With floor slabs

$$f_1 = 5.93 \text{ c.p.s.} \quad ; \quad f_2 = 17.65 \text{ c.p.s.} \tag{7}$$

$$\{\Phi^{(1)}\} = \begin{Bmatrix} 1.000 \\ 1.813 \end{Bmatrix} \quad ; \quad \{\Phi^{(2)}\} = \begin{Bmatrix} 1.000 \\ -0.455 \end{Bmatrix} \tag{8}$$

Hysteretic Lateral Load Deflection Relationships

These were experimentally determined by applying a horizontal force in turn at each storey level and measuring the deflections at both the storey levels. The deflections were measured at various stages of a complete reversal of the applied loads so that hysteretic load deflection relationships could be obtained over a full cycle of loading. Hysteretic curves for various intensities of lateral loads, are shown in figs. 2, 3 and 4. These were used to evaluate influence matrix, frequencies and damping.

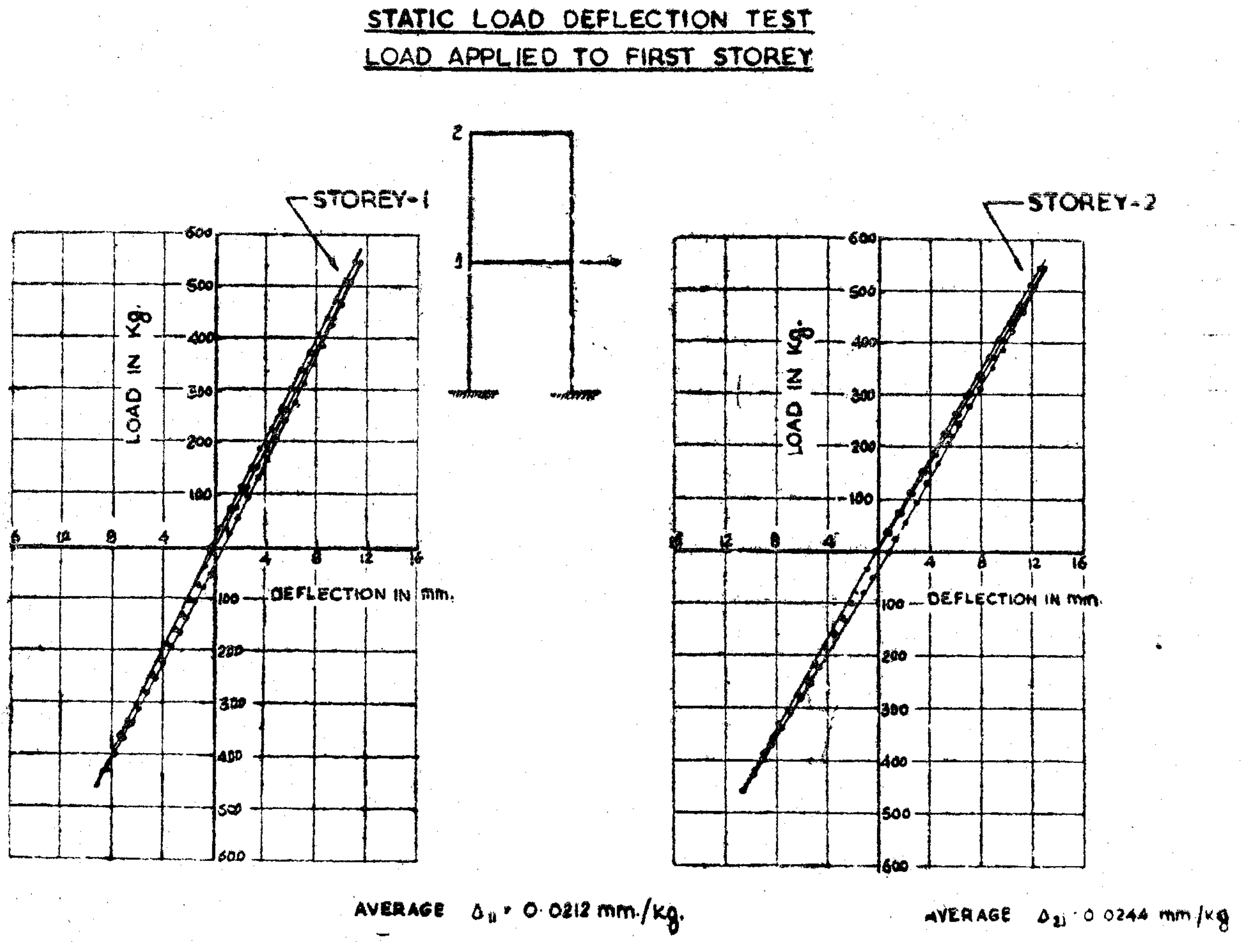

Fig. 2

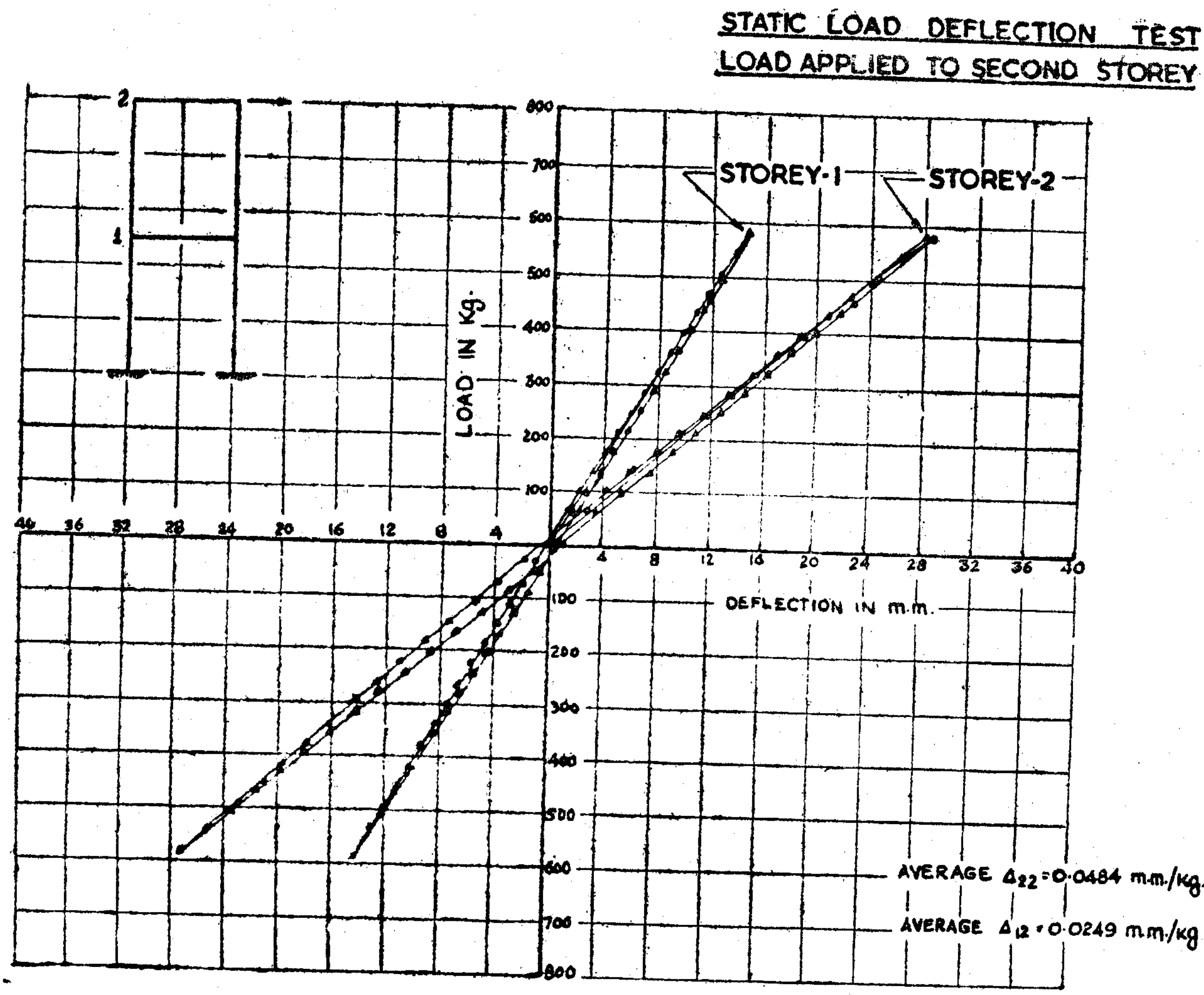

Fig. 3

For small intensities of lateral loads, where the structure essentially behaves linearly, the influence coefficients were obtained as the average of the slope of force deflection curve on each side of the loading. The influence matrix evaluated from experiments is as follows:—

$$[G] = \begin{bmatrix} 21.1 & 24.8 \\ 24.3 & 48.2 \end{bmatrix} \times 10^{-3} \quad \text{mm/Kg.} \tag{9}$$

This matrix compares favourably with that obtained from theory.

Using the above matrix, frequencies and mode shapes would be as follows :

(a) Without slabs.

$$f_1 = 12.10 \text{ c.p.s.} \quad ; \quad f_2 = 47.50 \text{ c.p.s.} \tag{10}$$

$$\{\phi^{(1)}\} = \begin{Bmatrix} 1.000 \\ 1.795 \end{Bmatrix} \quad ; \quad \{\phi^{(2)}\} = \begin{Bmatrix} 1.000 \\ -0.253 \end{Bmatrix} \tag{11}$$

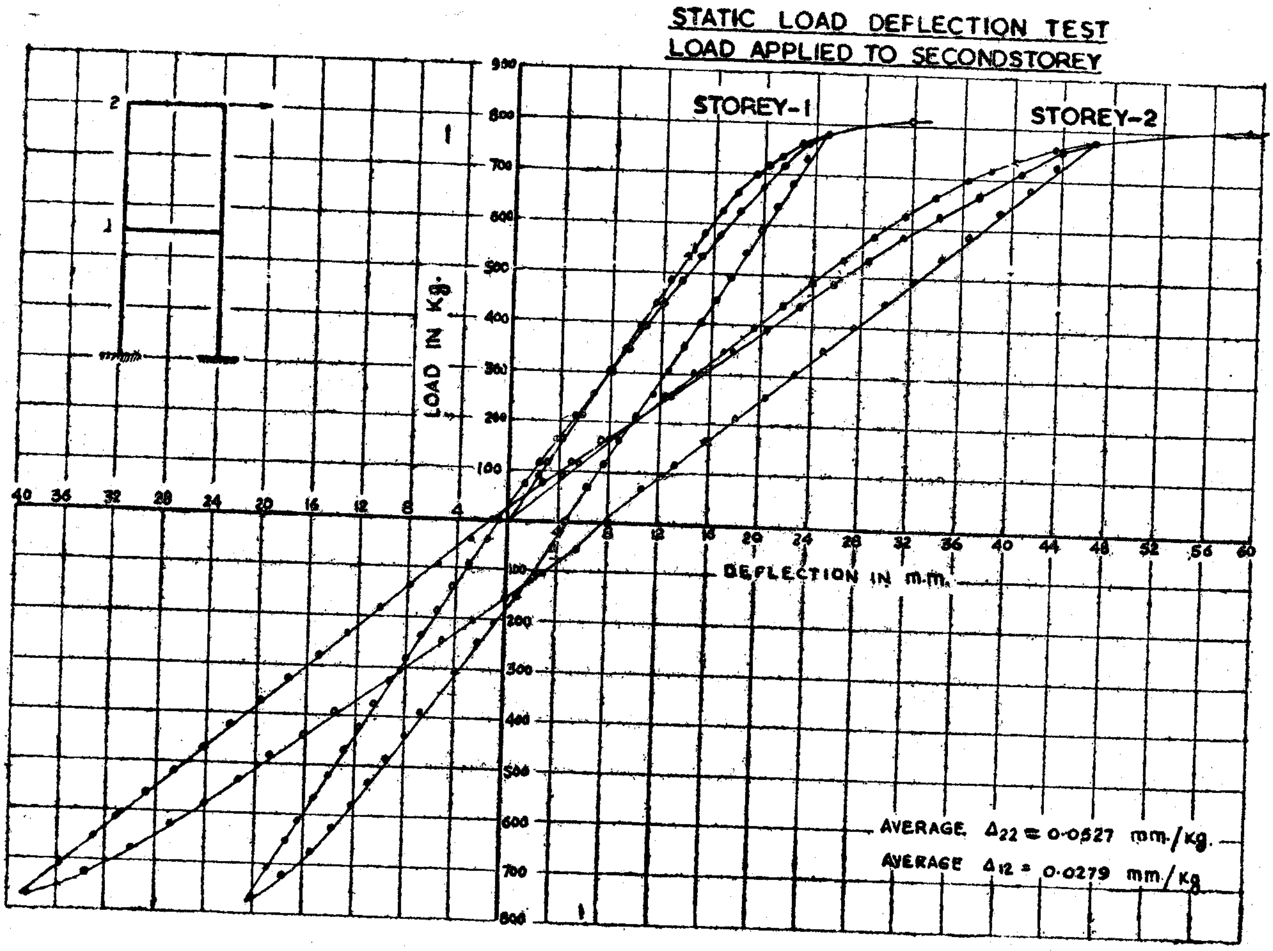

Fig. 4

(b) With floor slabs.

$f_1 = 5.70$ c.p.s. ; $f_2 = 18.50$ c.p.s. (12)

$$\{\Phi^{(1)}\} = \begin{Bmatrix} 1.000 \\ 1.700 \end{Bmatrix} \quad ; \quad \{\Phi^{(2)}\} = \begin{Bmatrix} 1.000 \\ -0.472 \end{Bmatrix} \tag{13}$$

These values also are in close agreement with those of theory.

For higher intensities of lateral loads, influence coefficients are obtained by considering the secant modulus of the curve. This equivalent linearisation, involving a reduction in stiffness, was used for evaluating frequencies in the nonlinear range. The computed values of frequencies at various levels of lateral loads are given in table 1.

The damping in a framed structure is mainly due to the hysteretic behaviour of the restoring force deflection relationship. For a single degree of freedom system, damping due to hysteresis is given by

$$\zeta = \frac{1}{2\pi} \quad \frac{A}{k\,x^2} \tag{14}$$

where ζ = percentage of critical damping

A = area of the hysteresis loop

k = secant modulus corresponding to x

x = maximum amplitude of vibration

Though hysteresis damping is not exactly defined for multiple degree of freedom systems, damping has been calculated using force deflection relationship of the top storey. These values are given in Table 2.

An attempt was made to find whether Jennings's[3] equation could be fitted to the hysteretic curves obtained experimentally. It was found that the parameters were different for the various curves corresponding to different intensities of loading. For a load P_o of 630 Kg., a was equal to 0.2 and $\gamma=11$. This gave a damping of 1.15%. At that load level, from the area of hysteretic curve, damping worked out to be 1.3%.

Free Vibration Tests

The structure was subjected to free vibration tests by applying a horizontal pull at the top storey and suddenly releasing it. The resulting vibrations were recorded. Such tests were carried for various intensities of initial pull.

The frequencies of vibrations and damping obtained from the free vibration records (based on the first one or two cycles of vibration) are given in Table 1 and Table 2 respectively.

It is seen from the tables that there is good correlation between values obtained from hysteretic curves and free vibration tests. It is observed that frequencies decrease with increase in lateral load intensity thereby confirming the softening behaviour of the restoring force. Damping increases with nonlinearity and this behaviour is also as expected. The maximum change in frequency was of the order of 10%.

Forced Vibration Tests

These tests were carried out by mounting the structure on a platform supported on rollers and subjecting the platform to a steady state sinusoidal excitation in a horizontal direction. Accelerations at the two floor levels as well as at the base were measured. The relationship between the acceleration at any floor level i and that at the base is given by

$$\frac{\ddot{x}_i}{\ddot{y}_b} = \left[\left\{ 1 + \left(\sum_{r=1}^{n} B_i^{(r)} \eta_r^2 \mu_r \cos\theta_r \right) \right\}^2 + \left\{ \sum_{r=1}^{n} B_i^{(r)} \eta_r^2 \mu_r \sin\theta_r \right\}^2 \right]^{1/2} \quad (15)$$

where

$\ddot{x}_i$ = maximum absolute acceleration at floor level

$\ddot{y}_b$ = maximum base acceleration

$B_i^{(r)}$ = mode participation factor in the r^{th} mode of vibration

$$= \frac{\phi_i^{(r)} \cdot \sum_{j=1}^{n} m_j \phi_j^{(r)}}{\sum_{j=1}^{n} m_j (\phi_j^{(r)})^2}$$

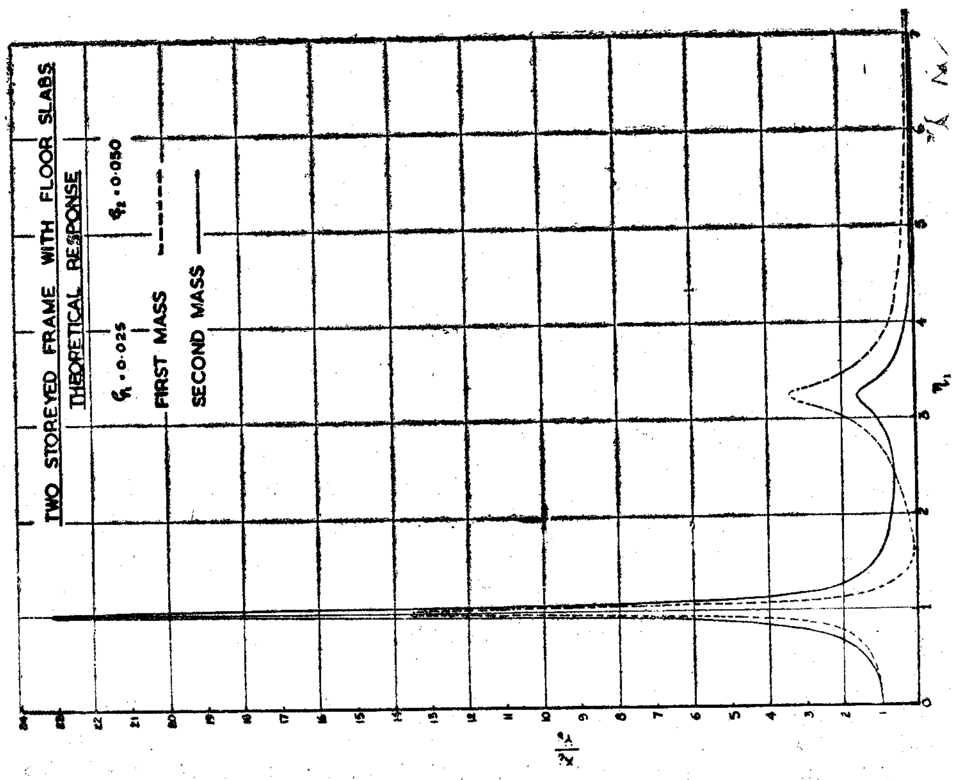
TWO STOREYED FRAME WITH FLOOR SLABS
THEORETICAL RESPONSE
ζ1 = 0·025
ζ2 = 0·050
FIRST MASS
SECOND MASS

Fig. 5

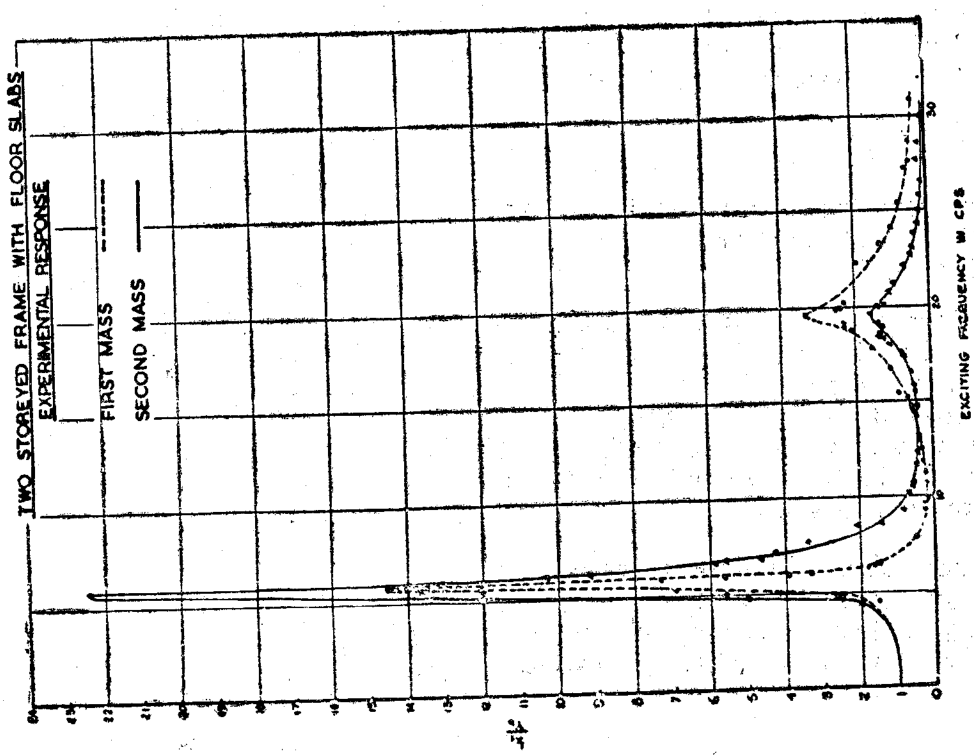
TWO STOREYED FRAME WITH FLOOR SLABS
EXPERIMENTAL RESPONSE
FIRST MASS
SECOND MASS
EXCITING FREQUENCY IN CPS

Fig. 6

μ_r = dynamic amplification factor in the r^{th} mode

$= [(1-\eta_r^2)^2 + (2\,\eta_r\,\zeta_r)^2\,]^{-1/2}$

$\theta_r = \tan^{-1}[2\,\eta_r\,\zeta_r/1-\eta_r^2]$

$\eta_r = \omega/p_r$

ζ_r = damping factor in r^{th} mode

p_r = undamped natural frequency of vibration in the r^{th} mode

ω = frequency of excitation

The theoretical response curves relating $\ddot{x}_1/\ddot{y}_b$ and ω have been given in fig. 5. The damping values have been assumed to be equal to those obtained from steady state response tests. The experimental response curves are plotted in fig. 6 and there is a close agreement between the theoretical and experimental response curves.

The excitation frequency at resonance correspond to the natural frequency of the system. The two resonant frequencies are 5.72 c p.s. and 19.56 c p.s. The damping factors worked out on the basis of measured acceleration values at resonance are equal to 0.025 and 0.050 for the first and second modes of vibration. These apparently higher values of damping, compared to the free vibration tests in the elastic range, may be due to instability of the vibrating device at resonance.

For systems with small damping, the total damping may be assumed to be a combination of absolute damping and relative damping[4]. Absolute damping would decrease with higher modal frequencies whereas relative damping would increase. For this system experiments show that damping is greater in the second mode compared to that of the first, indicating that damping is of relative type. This system ought to have greater interfloor damping and this behaviour is confirmed.

Ultimate Load Test

The structure was finally tested to failure by applying lateral loads. The frame failed at an ultimate load of 872 kg.

The ultimate load has been evaluated theoretically. The beam and column sections were separately tested to failure to determine the plastic moment capacity of these sections. The average values were found to be 150750 mm. Kg. for the beam (M_{pb}) and 77000 mm. Kg. for the column (M_{pc}).

The mechanism of failure is as shown in fig. 7. This gives the least value of the lateral load F_u applied at the top storey level. Equating the energy dissipated to the external work done,

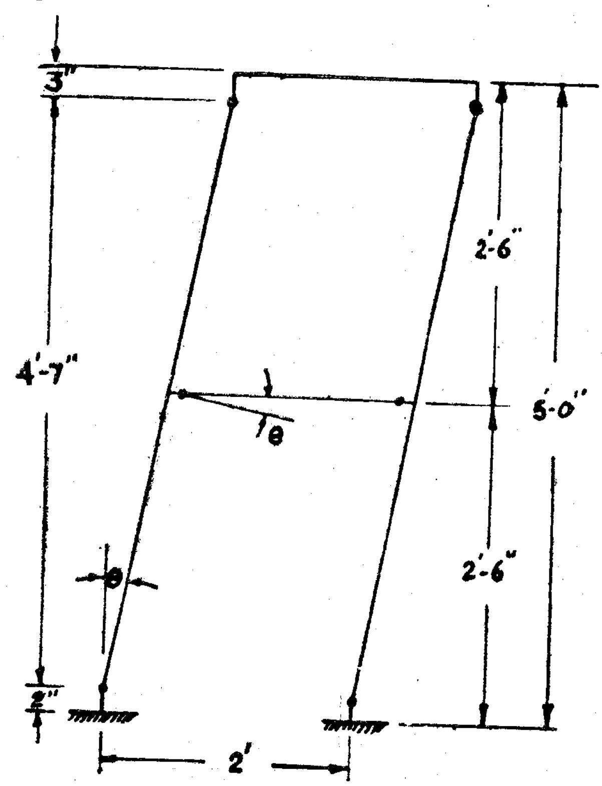

Fig. 7

$$4\, M_{pc}\, \theta + 2\, M_{pb}\, \theta = 1396 \times \frac{F_u}{2} \times \theta \qquad (16)$$

and $F_u = 872$ Kg.

The axial column thurst at ultimate load is found to be less than 10% of the axial thrust capacity of the column, so that the thrust does not affect the plastic moment capacity of the column.

The ultimate load test results are in agreement with that of theory.

Conclusions

Frequencies were calculated theoretically for elastic behaviour. These compared well with those obtained from hysteresis curves and free and forced vibration tests. In the nonlinear range, the secant modulus method of estimating influence coefficients proved to be a good approximation. Frequencies decrease with increase in nonlinearity, confirming the softening restoring force characteristics of the system. There is a fair agreement between damping factors calculated by different methods. Damping increases with nonlinearity. From forced vibration tests it was observed that damping in second mode is greater than that in the first mode. This would indicate that damping is predominantly due to inter–floor damping. The ultimate load test results are also consistent with theory.

Acknowledgements

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TABLE 1

Fundamental Frequency of Frame (Without Slab)

Sr. No	Load in Kg.	Frequency in cycles per second	
		Load—Defletion Curve	Free Vibration Records
1.	250	12.00	12.00
2.	500	11.90	12.00
3.	630	11.80	11.80
4.	680	11.70	11.60
5.	725	11.50	11.40
6.	770	11.10	11.20
7.	820	—	11.10
8.	860	—	11.00

TABLE 2

Damping Factors

Sr. No.	Load in Kg.	Percentage of critical Damping	
		Hysteresis Curves	Free vibration Records
1.	580	1.1	1.2
2.	630	1.3	1.5
3.	780	5.0	4.1

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DESIGN AND CONSTRUCTION OF MASONRY BUILDINGS IN SEISMIC AREAS

A. S. Arya*

Introduction

In the past severe earthquakes all over the world, the masonry buildings have generally been damaged the most because of their heavy weight, small or no tensile strength, small shearing resistance, lack of proper bonding between longitudinal and cross walls and poor-workmanship. Yet such buildings are being put up in the conventional way for reasons of climatic suitability, cheapness and local availability of materials, widespread knowledge of methods of construction etc. It is quite apparent that it will not be possible to do away with this kind of construction in the seismic areas particularly in the developing countries. Therefore finding effective methods of strenthening such buildings is of paramount importance so that the danger to life and property during future earthquakes is minimised. An important requirement of such methods is that they should be cheap so that the additional cost of strengthening is not more than what one would like to spend on insurance.

For closer examination a building is dissected into various components to establish where each component lacks strength against earthquake forces and suggest means to strengthen them. The discussion here is based on brick buildings in India not taller than four storeys. The conclusions can easily be interpreted for similar construction in other countries. It is assumed that the bricks are not weaker than the building mortar in tensile, shear and compressive strengths.

Structural Action

During ground motion, an inertia force acts on the mass of the building due to which the buildings tend to remain stationary while the ground moves. If the building is rigidly fixed into ground, the inertia forces will cause horizontal shears in the building the magnitude of which will be a function of the ground motion and the stiffness and damping characteristics of the building. However, if relative motion is possible between the ground and the building, the building may slide to and fro and the forces generated, during the earthquake, which would tend to shear the building, would be small. This type of action may be visualised if the ground under a railway wagon is imagined to shake along the direction of rails. It has clearly been demonstrated in the 1930 Dhubri (Assam) earthquake and the 1934 Bihar-Nepal earthquake that where a relative displacement was possible between the ground and the superstructure, the building suffered less than other similar buildings fixed into the ground.

As a result of the shaking, the roof tends to separate from the supports, the roof covering tends to be dislodged, walls tend to tear apart and if unable to do so the walls tend to shear off diagonally in the direction of motion. If filler walls are used within steel, concrete or timber framing, they may fall out of the frame bodily unless properly tied to the framing members. Thus on the whole it is indicated that a necessary condition for earthquake resistance is that the various parts of the building should be adequately tied together.

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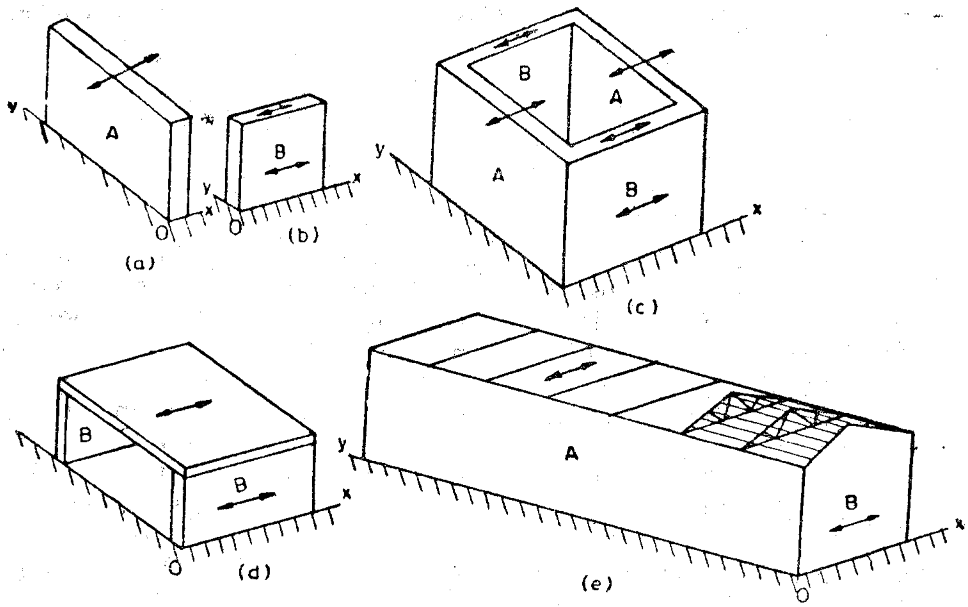

Fig. 1

Consider the structural elements and structures shown in Figure 1. In (a), wall A is free standing and the ground motion is acting transverse to it. The force acting on the mass of the wall tends to overturn it. The resistance of the wall is obviously very small. In (b), the free standing wall B is subjected to ground motion in its own plane. It is clear that in this case, because of its large depth in the plane of bending, the wall will offer great resistance. Such a wall is termed as shear well. It may be noted that for ground motion parallel to y-axis, wall A will act as shear wall and wall B will topple over. Now consider the combination of walls A and B as shown in (c). For the x-direction of motion as shown, walls B act as shear walls and besides themselves they offer resistance against the collapse of walls A as well. Walls A now act as vertical slabs supported on two vertical sides and bottom and subjected to the inertia force on their own mass. Near the vertical edges, the wall will carry bending moments in the horizontal plane for which the masonry has little strength. Consequently cracking and separation of the walls may occur. If however a horizontal bending member (like a timber beam or reinforced concrete or reinforced brick runner) were inserted at a suitable level in wall A and continued in wall B, this will take care of the bending tensions in the horizontal plane. So far as vertical bending is concerned, the wall gets aprecompression due to self weight and can be made to take care of bending tensions. The situation will be the same for walls B for ground motion along y-axis. Thus the bending member will be required in walls B also. Therefore a horizontal runner, strong in bending, is required alround the enclosure. Such a runner is termed as **band** and depending upon at which level it is used, it is called **roof band, lintel band, gable band or plinth band.**

In Figure 1(d), a roof slab is resting on walls B and earthquake is along the x-axis. Assuming that there is enough adhesion between the slab and the walls, the slab transfers its inertia force at the top of walls B, causing shearing and overturning actions in them. To be able to transfer its force on to the walls, the slab must have enough strength in bending in the horizontal plane. Whereas R. C. or R. B. slabs shall possess such strength inherently, other types of roofs or floors such as timber joists with timber plank or brick tile covering will have to be connected together and fixed to the walls suitably so that they are able to transfer their inertia force to the walls This point is discussed in greater detail later. After this load transfer, the walls B must have enough strength as shear walls to withstand the applied inertia force and its own inertia force It is quite clear that the structure shown at (d), when subjected to ground motion along y-axis, will collapse very easily because walls B have very little bending resistance in the y-direction. Hence bracing arrangement, either by shear walls or by diagonal braces, must be provided in the y-direction.

Lastly, consider the barrack type structure carrying roof trusses as shown at (e). The trusses rest on walls A and the walls B are gabled to receive the purlins of the end bays. The ground motion is along x-axis. The inertia forces will be transmitted from sheeting to purlins, trusses and from trusses to walls A. The end purlins will transmit some force directly to gable ends. In order that the structure does not collapse, the following arrangements must be made :

(i) the trusses must be anchored into the walls by holding down bolts,

(ii) walls A, which do not get much support from the walls B in this case, must be made strong enough in vertical bending as cantilever, or

(iii) some suitable arrangement must be made to transmit the force horizontally to end walls B which are well suited to resist the force by shear wall action.

Strengthening of walls A in vertical direction may be done by providing adequate pillasters under the roof trusses. For horizontal transfer of forces, a band may be provided at top of wall to which the roof trusses are conneted, or diagonal bracing may be provided in plan at the level of main ties of the trusses which should extend from one gable end to the other and be suitably connected to the end walls B. The inertia force of top half height of walls A may be assumed to be resisted by such a band or bracing system.

Now if ground motion is along y-direction, walls A will be in a positon to act as shear walls and all forces may be transmitted to them. In this case, the purlins act as ties and struts and transfer the inertia force of roof to the gable ends. A sloping band is required at the top of gable ends to transfer this inertia force from gable ends to the walls A where this band joins roof band at the top of walls A.

The above discussion leads to the following requirements for structural safety of masonry buildings during ground motions:

1. A free standing wall must be designed as vertical cantilever.

2. A shear wall must be capable of resisting all horizontal forces due to its own mass and those transmitted to it.

3. The roof or floor elements must be tied together and be capable of transferring their inertia force to shear walls by bending in horizontal plane.

4. The trusses must be anchored to supporting walls and have an arrangement of transfering their inertia force to end walls.

5. The shear walls must be present along both the axes of the building.

6. The walls must be effectively tied together to avoid separation at vertical joints due to shaking.

Horizonthal bands may be provided for tying the walls together and transferring the inertia load horizontally to the shear walls.

Design of Bands

As stated above, the bands are required for tying the walls together and imparting horizontal bending strength to them. The forces, which the bands are subjected to, are usually indeterminate. In most cases arbitrary dimensions are adopted. An approximate method is suggested here for arriving at minimum dimensions of the band at any level. This is explained by the following numerical example.

Example 1

Figure 2 shows a single room building with its roof removed. The walls are 20 cm thick in modular bricks built in 1 : 6 cement sand mortar. The roof, which is other than R. C. or R. B. slab, weighs 600kg/m². The bands are required for earthquake coefficient of 0.16.

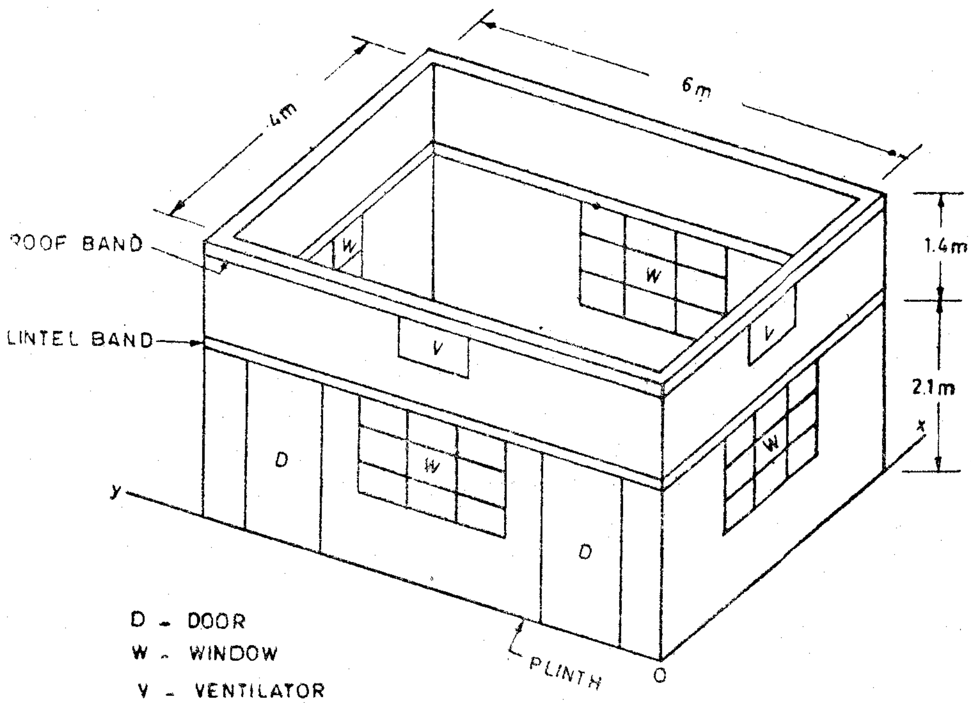

Fig. 2

Considering the door and window openings, two bands will be suitable, one at lintel and the other at roof level. A band may be necessary at plinth level also under certain conditions. Let the earthquake force act along x-axis. The lintel band divides the long wall in two portions, the lower portion spanning between plinth and lintel band and the upper portion spanning between lintel band and roof band.

Taking weight of masonry 1920 kg/m³, weight of 20 cm wall is 384 kg/m².

Checking Vertical Bending of Wall

Vertical span is 2.1 m with flat ends.

Moment $= 0.16 \times 384 \times 2.1^2/12 = 22.6$ kgm/m

$$\text{Stress} = \frac{22.50 \times 100}{100 \times 20^2/6} = \pm\ 0.338 \text{ kg/cm}^2$$

Weight of wall and roof per m approximately

$$= 384 \times 1.4 + \frac{600 \times 6\ 4 \times 4.4}{2\ (6.2 + 4{\cdot}2)}$$

$$= 1350 \text{ kg.}$$

$$\text{Stress} = \frac{1350}{100 \times 20} = 0.675 \text{ kg/cm}^2$$

Combined stresses $= 0.675 \pm 0.338 = 1.013, 0.337$ kg/cm² (compressive)

Therefore the wall is safe.

Lintel Band

Neglecting openings, horizontal load on lintel band

$$q_h = 0.16 \times \frac{21 + 1.4}{2} \times 384 = 107 \text{ kg/m}$$

Assuming continuity of band at corners, bending moment in band

$$M = 107 \times 6.2^2/10 = 411 \text{ kgm}$$
$$F = 107 \times 6/2 = 321 \text{ kg}$$

Taking the following stresses for design,

allowable bending compression in concrete	50 kg/cm²
allowable shearing stress in concrete	5 kg/cm²
allowable tensile stress in steel	1400 kg/cm²
modular ratio	18

permissible increase in stress due to earthquake $33\frac{1}{3}\%$ and taking the compression steel into account (because reinforcement is required on both faces due to reversible nature of force) or using steel beam theory, the area of steel on each face

$$A_t = \frac{41100}{1.333 \times 1400 \times 15} = 1.471 \text{ cm}^2$$

The thickness of the band may be determined from the consideration of diagonal tension. Limiting the shear stress to the allowable value, thickness

$$b = \frac{321}{1.333 \times 5 \times 0.867 \times 17.5} = 3.17$$

A minimum thickness of 7.5 cm may be adopted. One bar 14mm dia may be used on each face. Links consisting of 6 mm dia with hooks at both ends may be used @ 15 cm/cc to keep longitudinal bars in position. Figure 3 show the reinforcement detail near a corner.

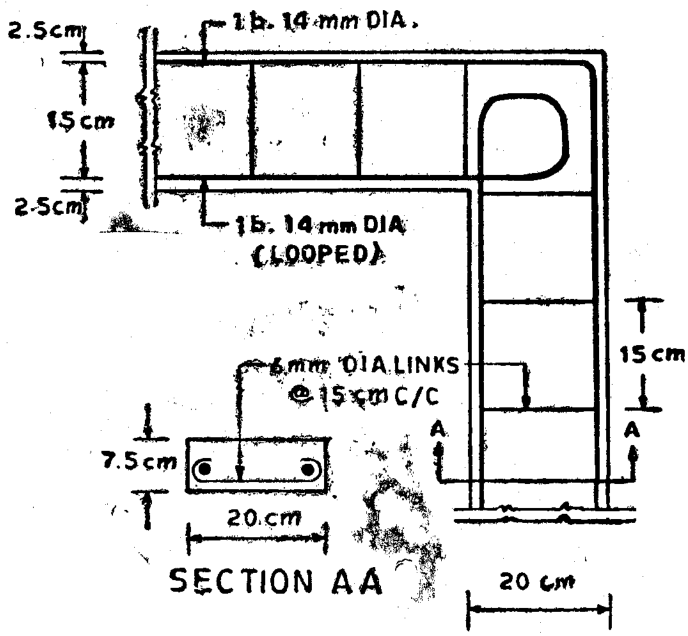

Fig. 3

As an alternative to the reinforced concrete, reinforced brickwork may be used. In the present case, the steel requirement is the same. Two 10 mm dia bars may be located in two consecutive motar courses on each face. The motar must be 1:3 cement sand mortar, its thickness being such as to provide at least 6mm cover to the bar, say 25 mm in the present case. Figure 4 shows the cross section and the arrangement of bending in plan.

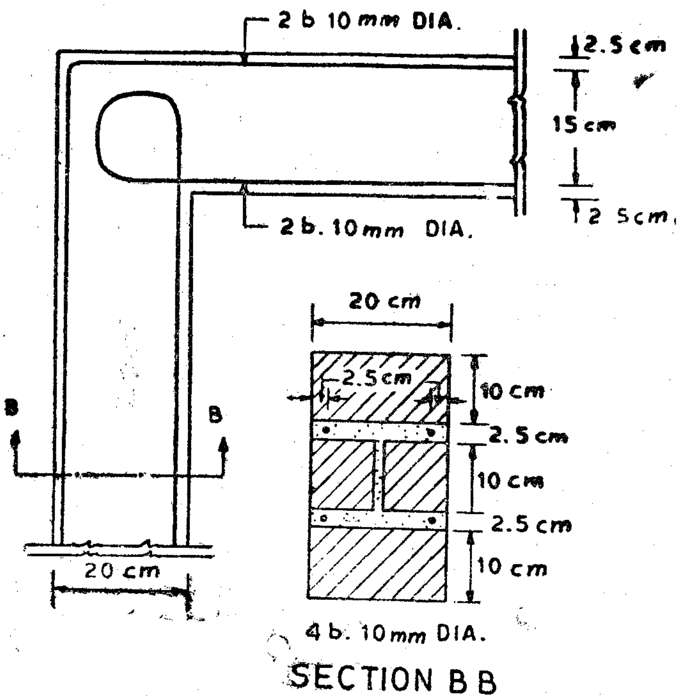

Fig. 4

Roof Band

The inertia force due to load as coming on the roof band depends on the arrangement of roofing elements. Let us assume that the roofing elements are connected with the long walls. Inertia load on roof band

$$q_h = 0.16\,(600 \times 4.4/2 + 384 \times 1.4/2) = 254 \text{ kg/m}$$

As for lintel band the section of 7.5 cm × 20 cm may still be maintained. In this case two bars 22 mm dia, one on each face, will be found to be sufficient.

Example 2

A barrack, 15 m long between end walls and 6 m wide carries A C. sheet roofing over timber trusses. The roof weighs 100 kg/m² of horizontal area. The walls are 30 cm thick and 4 m tall above plinth, the top level of door and window openings being 2.5 m. The strengthening is required against an earthquake coeffecient of 0.107.

Anchorage

Holding down bolts as required for wind shall suffice.

Roof Band

Span $= 15 + 0.30 = 15.3$ m

Load from roof $= 0.107 \times 100 \times 6.6/2 = 35.4$ kg/m

Load from wall $= 0.107 \times 1920 \times 0.30\,(4.0-2.5)/2 = 46.1$ kg/m

Total load $= 81.5$ kg/m

$$M = \frac{81.5 \times 15.3^2}{10} = 1905 \text{ kg/m}$$

$$F = 81.5 \times 15/2 = 611 \text{ kg}$$

Proceeding as in Example 1, a 10 cm × 30 cm band in M 150 concrete is sufficient when reinforced with 4 bars 16 mm dia, one at each corner and 6 mm diameter stirrups 25 cm apart.

Horizontal Bracing

Alternative to the roof band, horizontal bracing system may be used. Let the trusses be 3 m apart and the main tie be divided into 3 panel lengths of 2.1 m each. Let a horizontal truss be provided as shown in Figure 5, which will be able to take the horizontal force from roof as well as both the longitudinal walls. In large halls such trusses may be provided along both the long walls.

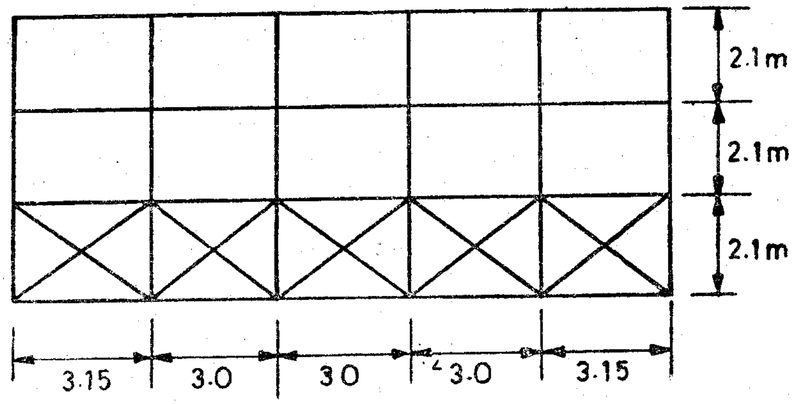

Fig. 5

Panel point load $= (35.4 + 2 \times 46.1) \times 3 = 383$ kg.

Maximum shear in end panel $= 383 \times 2 = 766$ kg.

Maximum force in tension diagonal assuming the compression diagonal to buckle

$$= \frac{766}{2.1}\sqrt{3.15^2+2.1^2} = 1380 \text{ kg.}$$

Use may be made of 12 mm dia. rods.

Max. force in chord member $= (1/2.1)(766 \times 6.15 - 383 \times 3.0)$
$= 1700$

For timber trusses, the members may be of wood and for steel trusses, of steel, designed for a force of 1700 kg as usual.

Maximum force in end members $= 766 + 383/4$
$= 862$ kg.

The members at end should normally be R.C. or R.B. so as to continue around the walls as a binding member in the shear walls.

Lintel band may be designed as in Example 1.

Free Standing Walls

Free standing walls, like compound walls, may be designed in two ways. If the mortar has doubtful or no tensile strength, the design must be made for no-overturning. It may be shown that with a factor of safety of 1.5, the minimum thickness of such a wall b is given by

$$b = 1.5\, C.h \qquad (1)$$

where C is the seismic coefficient and h is the height of the wall.

But when the mortar used is believed to have a known tensile strength, the minimum thickness of wall may be arrived at by using a factor of safety of 2.0. For brick work in 1:6 cement mortar, tensile stress of 1.0 kg/cm² may be permitted for design against earthquake forces. It can be derived that in this case minimum value of b will be given by

$$b = \frac{3C\, w\, h^2}{f + wh} \qquad (2)$$

where w is the unit weight of masonry and f is the allowable tension.

Example 3

Given $h = 160$ cm, $C = 0.12$, $w = 0.00192$ kg/cm^3, $f =$ zero or 1.0 kg/cm^2, the thickness of free standing wall is required.

For no tensile strength, using Eq. (1)

$$b = 1.5 \times 0.12 \times 160 = 28.8 \text{ cm, say } 30 \text{ cm.}$$

For given tensile strength, using Eq. (2)

$$b = \frac{3 \times 0.12 \times .00192 \times 160^2}{1.0 + .00192 \times 160}$$

$$= 18 \text{ cm, say } 20 \text{ cm.}$$

Partition Walls

Non-bearing partitions should also be checked for stability against earthquakes because their collapse may result in damage to property and life. They should in general be built into the bearing walls and be stayed against deflection at top as far as possible. If the top has to be kept free, horizontal steel bars may be inserted below the top course, one on each face and anchored into the bearing walls at ends. This would act as a sort of band. The stability of wall may then be checked in bending as a vertical slab.

Roofs and Floors

The more common types of roofs and floors used in conjunction with masonry are the following :

i) Roof trusses,

ii) Jack arches,

iii) Slab or slab and beams (reinforced concrete, reinforced brickwork of cast in situ construction)

iv) Joists (timber, reinforced concrete, presstressed concrete, precast slab units, stone slabs)

A general distinction is to be made among them. Those which are flat and are bonded or tied to the masonry, have a binding effect on the walls and do not require extra tying arrangement in the form of a band for the walls at roof or floor level. The examples are slab or slab and beam construction directly cast over the walls or jack arch floors or roofs provided with horizontal ties and laid over the masonry walls through good mortar like 1 : 6 cement sand mortar or equivalent in tensile strength. Others which simply rest on the masonry walls will offer resistance to relative motion through friction only which may not be relied upon. In such cases, other positive means are to be taken to tie the roofing and flooring elements together, to fix them to the walls and also tying the walls together.

To illustrate the point, let us consider the case of a timber joints floor of conventional construction in which the joints about 7.5 cm wide by 10 cm deep are placed across spans of about 3 m at a centre to centre spacing of 25 to 30 cm. These are then covered by timber planks nailed to the joints or by brick tiles placed directly over the joists. Clayee earth is then laid for water proofing and the roof or floor may finally be finished as desired. When timber planks are used, they act as effective ties to bind the joists together. But brick tiles have no such binding effect. In fact during an earthquake a small relative displacement of the joists will be enough to bring down the tiles damaging life and property in the room. In such cases the joists must be tied togetherat fix spacing apart so that the tiles above them may not be dislodged. This may be achieved by blocking the space between the joints by means of timber blocks having the same depth as the joists and fixed to them by nailing. Besides blocking at the ends, cross-bridging planks may be nailed to the joists at mid-span.

In order to fix the so formed joist-grill to the walls, a timber runner may be used on the wall under the grill to which the joints may be spiked. The timber runners will also be capable of tying the walls together if they are sufficiently large in section and full length pieces are used in all walls and jointed together firmly at their junctions. In that case, the timer runners may be built into the walls. If separate tying arrangement is desired for the walls an R.B. or R.C. band may be provided between the wall and roof or floor. The design of such a band has been illustrated in Example 1 before.

In the case of other kinds of elements, suitable details may be worked out for connecting the units together and fixing them to the band. For example for precast R.C. planks iron straps and bolts, welding the protruding reinforcement, laying cast-in-situ concrete, etc. are some of the devices that may be used for the purpose.

Design of Shear Walls

As has been stated earlier, shear walls are the main members for transferring all earthquake forces to the foundations. Tests have indicated that the factors determining their strength are too many and varied. The greatest source of error and uncertainty is the workmanship. Any effort to use the theory of elasticity for analyzing the stresses in brick or stone masonry therefore prove futile. Hence, the simplest available approach for their analysis should suffice. One such procedure for shear walls is explained below :

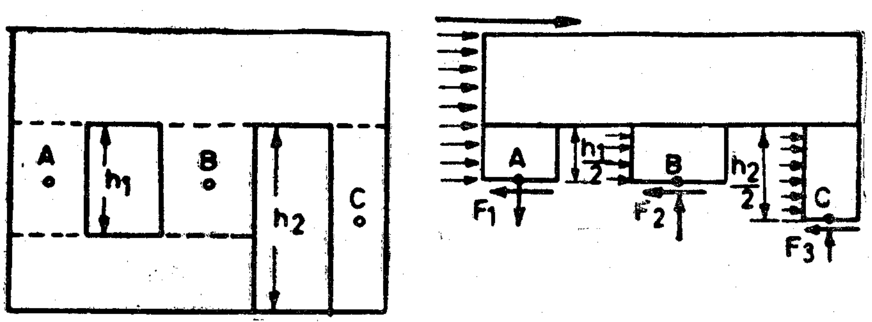

Fig. 6

Let masonry wall have openings as shown in Figure 6 and carry horizontal inertia forces due to the weight of roof and wall as indicated. For analyzing the stresses in such walls, it is assumed that the rotational deformations of the portions above and below the openings are much smaller than those of the piers between the openings and are neglected. Points of contraflexure are assumed at the mid-points of piers and shears are assumed to be shared among the piers such that their tops deflect by the same amount.

For calculating the deflections, the piers are assumed to be restrained against rotation at the ends but as free to deflect; and shear deformations are also considered. Thus if the horizontal shear in a pier is H, the deflection at its top is given by

$$\triangle = \frac{H h^3}{12 EI} + \frac{1.2 H h}{GA} \quad (3)$$

when h is the height of the pier, I its moment of inertia about the axis of bending, E the modulus of elasticity, G the modulus of rigidity and 1.2 is a factor required in the calculation of shear deflection of a member having a rectangular section. The shear stiffness of a pier may be defined 'as the load required to deflect its top by unit deflection'. Hence the shear stiffness is given by

$$S = \frac{H}{\Delta} = \frac{1}{\frac{h^3}{12\,EI} + \frac{1.2\,h}{GA}} = \frac{\frac{12\,EI}{h^3}}{1 + \left(14.4\,\frac{EI}{GA\,h^2}\right)} \qquad (4)$$

If the thickness of the wall is b and the width of pier is d, then expressing I and A in their terms and taking $G = E/2$ for masonry,

$$S = \frac{12\,EI}{h^3} \cdot \frac{1}{1 + 2.4\,(d/h)^2} \qquad (5)$$

The total horizontal shear in the piers may be distributed between them in the ratios of their shear stiffnesses. Let the shears produced be F_1, F_2 etc. Then the piers carry shears F, and bending moments $\frac{Fh}{2}$ at their top and bottom sections. The resulting values of maximum shear stress, q, and bending stress, P_b in any pier will be given by

$$q = \frac{1.5\,F}{bd} \qquad (6)$$

$$P_b = \frac{3Fh}{bd^2} \qquad (7)$$

The total weight of the structure above the top of piers W may be assumed to produce uniform direct stress P_o in them. Thus

$$P_o = W/\Sigma A \qquad (8)$$

where ΣA represents the sum of areas of all the piers in the plan of the building at any level.

The overturning forces are determined as follows :

Let the areas of the piers be A_1, A_2 and A_3 (Fig. 7) and the combined centre of gravity of the three areas be at G. Assuming the stress in the piers to be proportional to the distance from G, the forces in the three piers are ;—

$-K x_1 A_1$, $-K x_2 A_2$ and $K x_3 A_3$, where K is the coefficient of proportionality, and x_1, x_2 and x_3 are the distances of the points considered (in the three piers) from G. If M is the moment of the horizontal forces about A, we get,

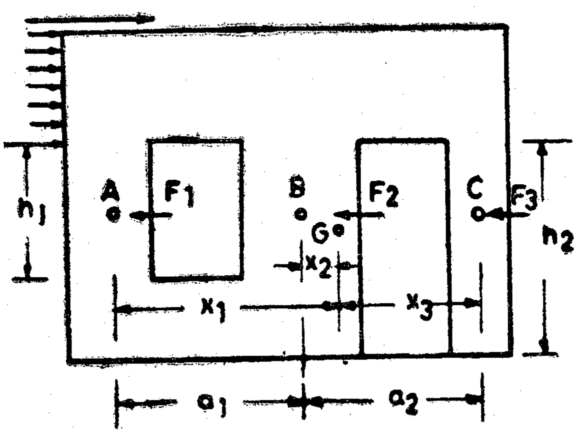

Fig. 7

$$K = \frac{M + F_3\left(\frac{h_2 - h_1}{2}\right)}{A_1 x_1^2 + A_2 x_2^2 + A_3 x_3^2} \qquad (9)$$

Knowing the constant of proportionality K the stresses due to overturning, p_t, may be found as follows :

$- Kx_1$ in pier A
$- Kx_2$ in pier B
$+ Kx_3$ in pier C (12)

Combining all the stresses, the maximum and minimum stresses on the cross section of each pier can be determined. That is, the resultant total stress p is given by

$$p = p_b + p_o + p_t \qquad (11)$$

which may be positive or negative depending upon the relative magnitudes and signs of the component stresses. Such calculations are to be made along each axis of the building for reversible earthquake force. If the stress becomes tensile, vertical steel reinforcement may be designed according to the usual theory of reinforced concrete for combined direct and banding stresses. In any case the compressive stress must remain within the safe compressive stress of the type of brickwork. For values of allowable stresses and other specifications reference may be made to IS:1905 1961. For designing the sections under direct and bending stresses combined, the charts given in Figure 8 may directly be used. The modular ratio of steel to brickwork may be assumed 40 for this purpose.

Conclusion

It may be concluded that if the tying and reinforcing arrangements as explained and illustrated in the text are adopted, a masonry building will act as one unit for resisting inertia forces caused by ground motion and have sufficient strength to withstand them. The provision of reinforcement is expected to impart some ductility to the otherwise brittle structure which will further help it to absorb energy and delay its collapse.

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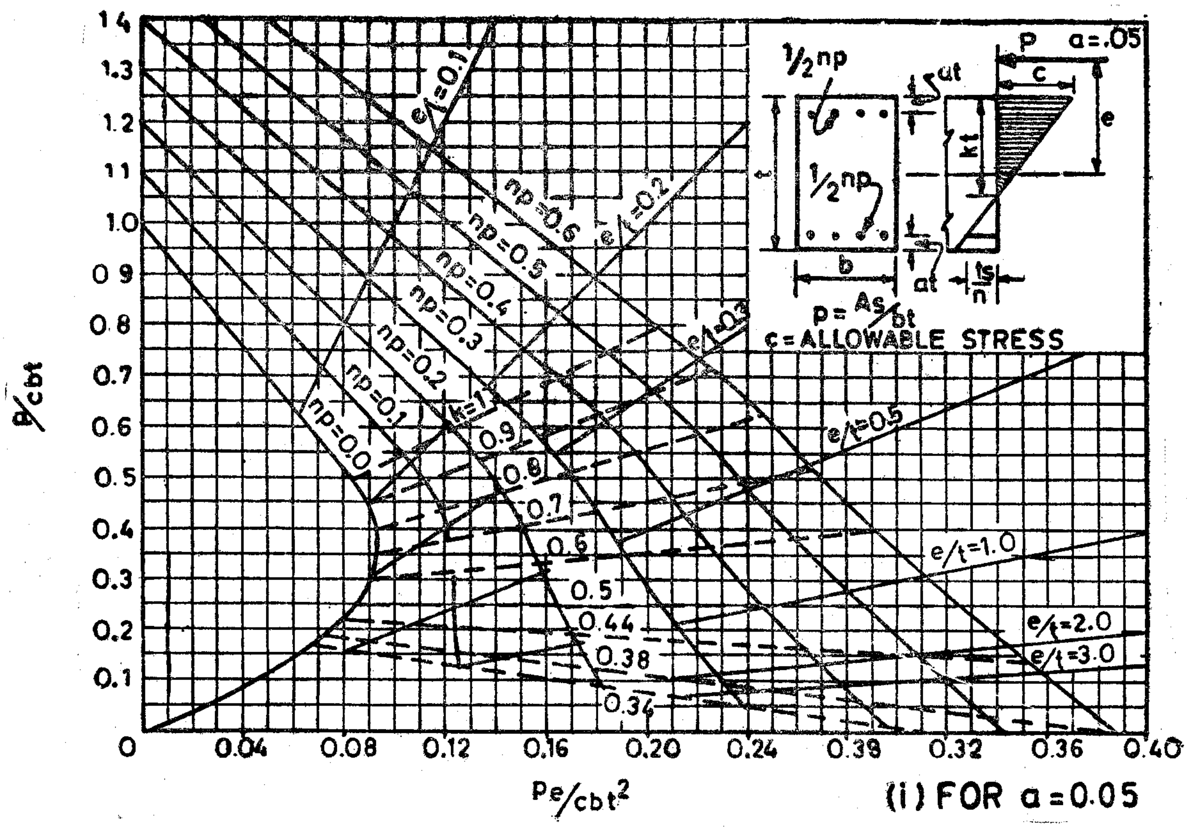


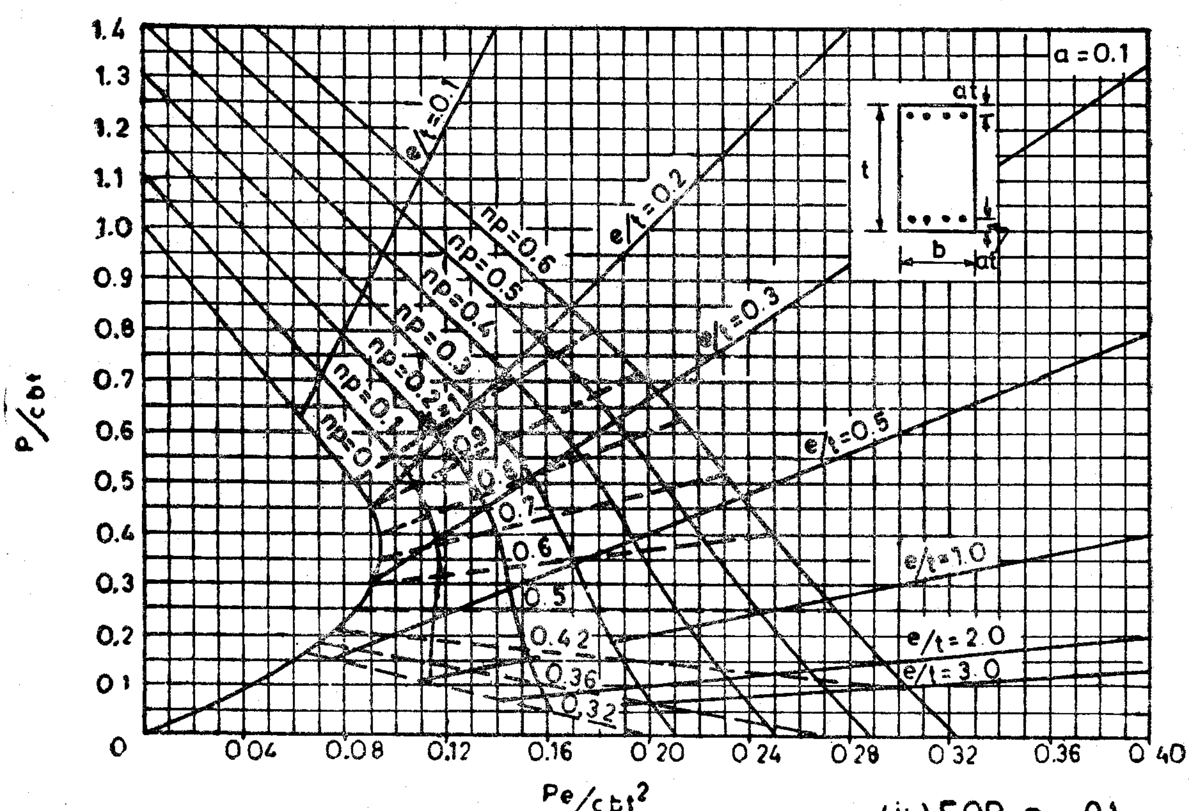
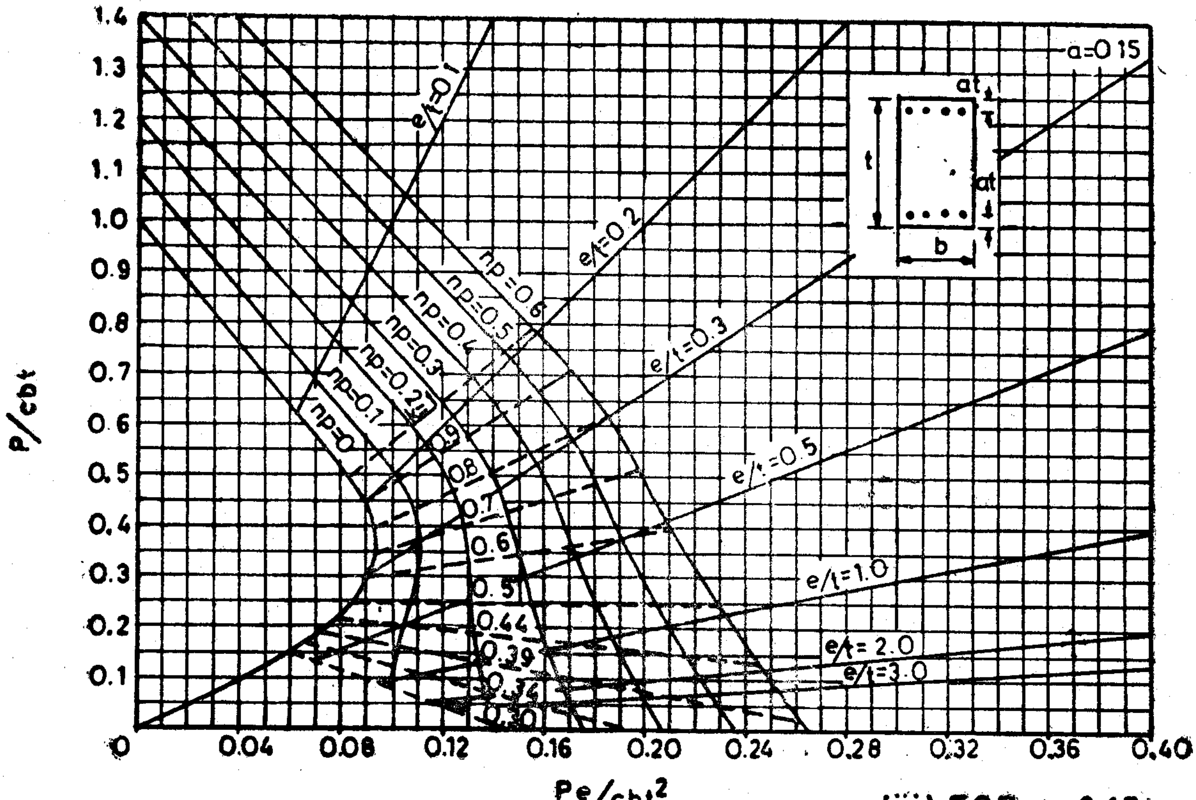
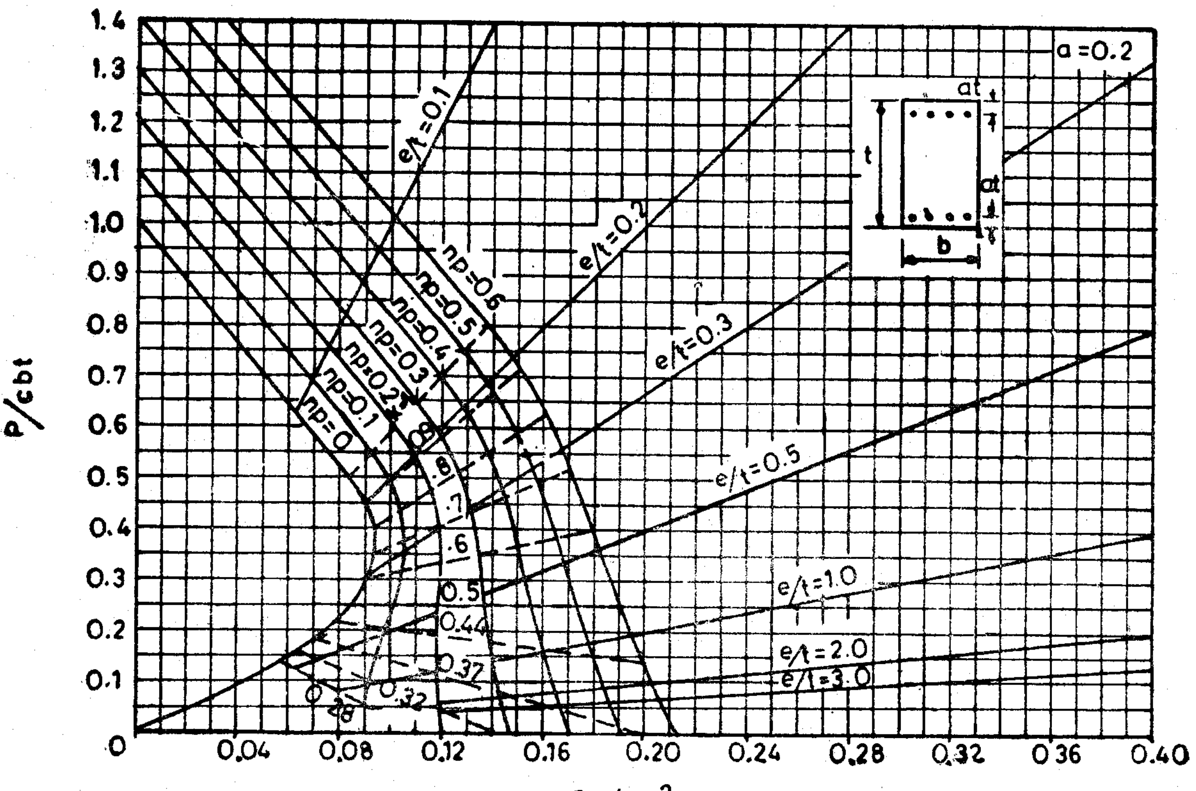


Fig. 8 (Continued)



(iii) FOR a = 0.15



(iv) FOR a = 0.2

Fig. 8

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SEISMOLOGICAL NOTES

(India Meteorological Department, New Delhi)

Earthquakes in and near about India during October 1966 to March, 1967

Date	Origin time (G.M.T.) h.	m.	s.	Epicentre Lat. (°N)	Long. (°E)	Region	Approx. depth (Kms.)	Magnitude	Remarks
1	2	3	4	5	6	7	8	9	10
Oct. *1	07	38	29.0 (CGS)	34.8	71.0	West Pakistan	25	5.3 (CGS)	Recorded at a few Indian Observatories.
Oct. 2	04	31	47.0 (CGS)	24.4	94.8	Burma-India border	65	5.2 (CGS)	—
Oct. 2	07	13	07.3 (NDI)	40 Kms. N-W of Delhi		—	—	—	Felt at Sonepat. Some kuchcha houses collapsed.
Oct. 5	13	32	46.5 (CGS)	6.6	93.6	Nicobar Islands	33	4.2 (CGS)	—
Oct. 8	00	46	10.2 (NDI)	43 Kms. N-W of Delhi		—	—	—	Felt at Sonepat.
Oct. 13	12	42	42.0 (SHL)	31.0	80 3	India-Tibet border	—	4.2 (NDI)	Recorded at many Indian Observatories.
	12	42	42.0 (CGS)	31.1	80.1	Tibet	33	5.3 (CGS)	—
Oct. 14	01	04	40.0 (SHL)	36.8	87.5	Northern Tibet	—	—	Recorded at most of the Indian Observatories.
	01	04	43.3 (CGS)	36.4	87.5	S. Sinkiang Province, China	24	5.2 (CGS)	—
Oct. 14	01	11	51.6 (CGS)	39.4	80.2	S. Sinkiang Province, China	33	4.8 (CGS)	—
Oct. 16	09	26	28.0 (SHL)	30.3	67.8	West Pakistan	—	5.6 (NDI)	Recorded at many Indian Observatories.
	09	26	36.5 (CGS)	30.0	68.6	West Pakistan	33	4.9 (CGS)	—
Oct. 18	20	34	37.1 (CGS)	24.3	94.8	Burma-India border	86	5.2 (CGS)	Recorded at a few Indian Observatories.
Oct. 20	00	53	20.0 (SHL)	34.6	78.2	N-E of Kashmir	—	—	Recorded at many Indian Observatories.
	00	53	38.7 (CGS)	33.6	78.5	Kashmir-Tibet border	27	5.0 (CGS)	—
Oct. 22	03	03	30.0 (SHL)	22.5	93.3	India-Burma border	—	5.7 (NDI)	Recorded at many Indian Observatories.
	03	03	23.5 (CGS)	23.1	94.4	Burma-India border	68	5.7 (CGS)	—

1	2	3	4	5	6	7	8	9	10
Oct. 23	00	00	01.8 (CGS)	29.9	68.2	West Pakistan	25	4.8 (CGS)	Recorded at a few Indian Observatories.
Oct. 23	18	19	09.0 (NDI)	65 Kms. N-W of Delhi		—	—	—	Felt at Rohtak.
Oct. 25	10	06	48.0 (SHL)	30.0	67.7	West Pakistan	—	5.5 (NDI)	Recorded at many Indian Observatories.
	10	06	58.1 (CGS)	29.9	68.9	West Pakistan	6	5.3 (CGS)	—
Oct. 29	08	59	42.0 (SHL)	27.3	66.4	Baluchistan	—	5.4 (NDI)	Recorded at many Indian Observatories.
Oct. 29	14	46	57.0 (CGS)	36.7	69.8	Hindukush	73	4.9 (CGS)	Recorded at a few Indian Observatories.
Oct. 29	19	28	57.2 (NDI)	39 Kms. N-W of Delhi		—	—	—	Felt at Sonepat.
Nov. 1	04	52	11.3 (CGS)	10.8	94.6	Andaman Island	33	5.0 (CGS)	—
Nov. 4	09	00	54.1 (CGS)	39.9	78.0	S. Sinkiang Province, China.	33	4.4 (CGS)	—
Nov. 4	22	17	32.1 (CGS)	34.0	72.5	West Pakistan	35	— (CGS)	—
Nov. 5	18	53	04 (CGS)	28.3	84.0	Nepal	33	5.2 (CGS)	Recorded at a few Indian Observatories.
Nov. 6	01	28	53.7 (CGS)	39.9	78.1	S. Sinkiang Province, China	49	4.5 (CGS)	—
Nov. 7	04	08	12.0 (CGS)	33.8	80.7	Tibet	33	5.1 (CGS)	Recorded at a few Indian Observatories.
Nov. 8	14	30	03.0 (CGS)	25.9	96.6	Burma	51	4.2 (CGS)	—
Nov. 9	10	11	57 (CGS)	38.9	71.5	Afghanistan USSR border	85	4.5 (CGS)	—
Nov. 12	08	28	23 (CGS)	23.4	61.6	Near coast of West Pakistan	33	4.7 (CGS)	Recorded at a few Indian Observatories.
Nov. 12	12	16	44 (CGS)	25.0	68.0	West Pakistan	5	5.1 (CGS)	Recorded at a few Indian Observatories.
Nov. 19	07	42	28.2 (CGS)	18.4	95.3	Burma	56	5.4 (CGS)	Recorded at a few Indian Observatories.
Nov. 19	19	05	38.1 (CGS)	37.0	71.4	Afghanistan USSR border	130	4.9 (CGS)	Recorded at a few Indian Observatories.
Nov. 19	20	29	47.0 (CGS)	35.8	70.8	Hindukush	140	4.6 (CGS)	Recorded at a few Indian Observatories.

1	2	3	4	5	6	7	8	9	10
Nov. 20	23	35 (CGS)	46.4	27.6	67.7	West Pakistan	36	4.8 (CGS)	Recorded at a few Indian Observatories.
Nov. 25	19	32 (NDI)	51.0	80 Kms. S-W of Delhi		—	—	3.8 (NDI)	Felt at Delhi.
Nov. 25	20	31 (CGS)	36	41.6	72.6	Kirgiz SSR	33	5.1 (CGS)	—
Nov. 29	10	11 (CGS)	02	36.4	70.2	Hindukush	228	4.5 (CGS)	Recorded at a few Indian Observatories.
Dec. 1	16	51 (CGS)	51	37.0	73.3	North western Kashmir	218	4.3 (CGS)	Recorded at a few Indian Observatories.
Dec. 6	02	30 (CGS)	53.0	36.2	70.0	Hindukush	58	4.9 (CGS)	—
Dec. 6	13	46 (NDI)	07.8	51 Kms. N-W of Delhi		—	—	3.0 (NDI)	Felt at Rohtak.
Dec. 6	23	29 (CGS)	51	36.3	70.5	Hindukush	233	4.5 (CGS)	—
Dec. 8	02	07 (CGS)	07.4	29.3	69.9	West Pakistan	37	5.1 (CGS)	Recorded at a few Indian Observatories.
Dec. 13	12	20 (SHL)	55.0	37.3	72.0	Hindukush	—	—	Recorded at many Indian Observatories.
	12	21 (CGS)	02.3	37.3	71.9	Afghanistan USSR border	126	5.3 (CGS)	—
Dec. 15	02	08 (SHL)	08.0	21.6	93.4	Burma	—	6.2 (NDI)	Recorded at most of the Indian Observatories. Felt at Shillong.
	02	08 (CGS)	03.0	21.7	94.5	Burma	81	5.7 (CGS)	—
Dec. 16	20	52 (SHL)	17.0	29.5	81.1	India-Nepal border	—	5.7 (NDI)	Recorded at most of the Indian Observatories. Felt extensively in Uttar Pradesh and adjoining areas of neighbouring states. 5 or 6 houses collapsed at Dharchula due to the earthquake. The earthquake caused cracks in some buildings at Pilibhit and Moradabad.
	20	52 (CGS)	13.5	29.6	81.0	Nepal	9	5.9 (CGS)	—
Dec. 16	22	12 (CGS)	49.2	29.7	81.0	Nepal-India border	5	5.4 (CGS)	—

1	2	3	4	5	6	7	8	9	10
Dec. 18	22	42 (CGS)	38.3	29.6	81.0	Nepal-India border	25	5.0 (CGS)	Recorded at a few Indian Observatories.
Dec. 21	22	11 (SHL)	03.0	29.0	81.1	Nepal-India border	—	5.1 (NDI)	Recorded at many Indian Observatories. Felt at Dharchula and Ash Kote.
	22	10 (CGS)	58.8	29.4	81.0	Nepal-India border	31	5.4 (CGS)	—
Dec. 25	17	07 (CGS)	01.4	37.2	70.1	Afghanistan-USSR border	91	4.6 (CGS)	—
Dec. 26	01	28 (CGS)	04.3	35.9	69.9	Hindukush	180	5.0 (CGS)	Recorded at a few Indian Observatories.
Dec. 29	16	41 (NDI)	28.4	42 Kms.N-W of Delhi		—	—	—	Felt at Sonepat.
Dec. 29	21	35 (CGS)	20.2	29.9	68.3	West Pakistan	14	4.6 (CGS)	Recorded at a few Indian Observatories.
Jan. 1	02	59 (CGS)	33.8	10.7	92.8	Andaman Islands	60	5.2 (CGS)	—
Jan. 1	03	35 (CGS)	42.9	7.6	94.4	Nicobar Islands	38	4.2 (CGS)	—
Jan. 2	10	31 (CGS)	20.6	29.8	69.1	West Pakistan	16	— (CGS)	Recorded at a few Indian Observatories.
Jan. 2	22	17 (NDI)	55.0	30.8	79.0	U.P.	—	4.7 (NDI)	Recorded at many Indian Observatories. Felt at Delhi and Mukteswar.
	22	17 (CGS)	53.4	30.7	79.3	Tibet India border	4	4.3 (CGS)	—
Jan. 4	11	26 (CGS)	45.4	23.4	93.9	Burma India border	38	5.4 (CGS)	Recorded at few Indian Observatories.
Jan. 5	10	07 (SHL)	58.0	39.5	73.9	Kirgiz SSR	—	5.8 (NDI)	Recorded at many Indian Observatories
	10	07 (CGS)	58.3	39.4	72.9	Kirgiz-SSR	11	5 3 (CGS)	—
Jan. 13	14	04 (CGS)	30.8	23.8	94.6	Burma India border	91	4.4 (CGS)	Recorded at a few Indian Observatories.
Jan. 14	00	27 (NDI)	18.5	39 Kms. N-W of Delhi		—	—	—	Felt at Sonepat.
Jan. 14	10	59 (CGS)	24.5	39.1	70.6	Tadshik SSR	25	4.9 (CGS)	—
Jan. 20	05	09 (CGS)	19.0	32.3	69.9	West Pakistan	66	—	Recorded at a few Indian Observatories.
Jan. 20	05	16 (CGS)	39.8	32.3	69.8	West Pakistan	70	5.1 (CGS)	Recorded at a few Indian Observatories.

1		2	3	4	5	6	7	8	9	10
Jan.	21	13	31	30.4 (CGS)	3.3	97.3	N. Sumatra	102	5.3 (CGS)	—
Jan.	22	12	10	00.0 (SHL)	10.0	93.0	Andaman Islands	—	—	Recorded at a few Indian Observatories.
		12	09	52.3 (CGS)	8.8	93.7	Nicobar Island	36	4.9 (CGS)	—
Jan.	22	16	11	56.6 (CGS)	36.7	71.3	Afghanistan USSR border	154	4.5 (CGS)	—
Jan.	25	01	50	09.0 (SHL)	37.3	70.6	Hindukush	224	6.0 (NDI)	Recorded at most of the Indian Observatories. Felt at Srinagar.
		01	50	19.4 (CGS)	36.6	71.6	Afghanistan USSR border	281	5.7 (CGS)	Felt widely.
Jan.	26	11	02	53.6 (CGS)	30.0	68.7	West Pakistan	33	4.9 (CGS)	—
Jan.	28	02	58	33.7 (CGS)	30.2	69.5	West Pakistan	39	4.5 (CGS)	—
Jan.	30	07	09	30.1 (CGS)	25.7	90.4	India-East Pakistan border	33	—	Recorded at a few Indian Observatories.
Jan.	30	21	05	35.0 (SHL)	26.0	95.5	North Burma	—	5.9 (NDI)	Recorded at most of the Indian Observatories.
		21	05	30.4 (CGS)	26.2	96.2	Burma	44	5.5 (CGS)	—
Feb.	2	01	18	56.6 (CGS)	40.6	75.3	Kirgiz-Sinkiang border	33	4.6 (CGS)	—
Feb.	2	07	37	54.9 (CGS)	39.7	75.5	S. Sinkiang Province, China	39	5.3 (CGS)	Recorded at a few Indian Observatories.
Feb.	8	17	17	50.0 (SHL)	23.3	93.2	Lushai Hills, Assam	—	5.6 (NDI)	Recorded at many Indian Observatories.
		17	17	45.7 (CGS)	23.2	93.9	Burma India border	33	5.1 (CGS)	—
Feb.	10	05	46	27.9 (CGS)	33.0	75.5	Kashmir	27	4.9 (CGS)	Recorded at a few Indian Observatories.
Feb.	11	08	05	08.4 (CGS)	36.7	71.1	Afghanistan USSR border	58	4.6 (CGS)	Recorded at a few Indian Observatories.
Feb.	12	16	06	47.8 (CGS)	35.8	71.0	Hindu Kush	100	5.2 (CGS)	Recorded at a few Indian Observatories.
Feb.	14	01	36	04.7 (CGS)	13.7	96.5	Andaman Islands	27	6.8 (CGS Surface wave)	Felt at Bangkok, Thailand.

1		2 3 4	5	6	7	8	9	10
Feb.	15	05 57 35.0 (SHL)	20.8	93.6	Burma	—	6.3 (NDI)	Recorded at most of the Indian Observatories.
		05 57 24.6 (CGS)	20.4	94.1	Burma	10	5.5 (CGS)	—
Feb.	17	02 44 54.5 (NDI)	40 Kms. N-W of Delhi		—	—	—	Felt at Sonepat.
Feb.	20	14 23 48.2 (CGS)	33.7	75.7	East Kashmir	33	4.8 (CGS)	Recorded at a few Indian Observatories.
Feb.	20	15 18 42.0 (SHL)	33.2	76.4	Kashmir	—	5.3 (NDI)	Recorded at most of the Indian Observatories. Felt at Srinagar and Mukerian.
		15 18 39.9 (CGS)	33.7	75.3	East Kashmir	24	5.7 (CGS)	Felt at Lahore, Pakistan.
Feb.	21	12 37 50.0 (NDI)	32.8	74.5	Kashmir	—	4.9 (NDI)	Recorded at many Indian Observatories. Felt at Srinagar.
		12 37 44.5 (CGS)	33.6	75.3	East Kashmir	31	5.1 (CGS)	—
Feb.	25	21 36 00.3 (CGS)	5.9	94.0	N. Sumatra	40	4.4 (CGS)	—
Mar.	2	09 51 48 (CGS)	41.4	71.4	Kirgiz SSR	59	4 2 (CGS)	—
Mar.	2	11 47 15 (CGS)	28.5	86.4	Tibet	44	4.9 (CGS)	Recorded at a few Indian Observatories.
Mar.	6	11 28 49.4 (CGS)	3.7	95.8	Off West coast of North Sumatra	57	5.1 (CGS)	Recorded at a few Indian Observatories.
Mar.	11	06 31 09.0 (CGS)	36.4	70.6	Hindu Kush	220	5.0 (CGS)	Recorded at a few Indian Observatories.
Mar.	11	16 56 48.7 (CGS)	28.4	94.4	India-China border.	7	5.3 (CGS)	Recorded at a few Indian Observatories.
Mar.	11	18 45 41 (CGS)	29.2	81.4	Nepal	8	4.0 (CGS)	Recorded at a few Indian Observatories.
Mar.	14	06 57 46.0 (SHL)	30.0	95.6	Sinkiang Province, China	—	6.0 (NDI)	Recorded at most of the Indian Observatories.
		06 58 04.6 (CGS)	28.4	94.3	India-China border	24	5.9 (CGS)	—
Mar.	14	14 35 12 (CGS)	36.5	70.6	Hindu Kush	193	4.8 (CGS)	Recorded at a few Indian Observatories.
Mar.	16	12 13 20.6 (CGS)	26.9	96.6	Burma	6	4.9 (CGS)	—

1	2	3	4	5	6	7	8	9	10
Mar. 24	11	11	43 (CGS)	34.6	70.0	Afghanistan	61	4.2 (CGS)	Recorded at a few Indian Observatories.
Mar. 26	03	08	27 (CGS)	27.2	67.5	W. Pakistan	21	5.8 (CGS)	Recorded at a few Indian Observatories.
Mar. 27	08	09	46 (SHL)	15.8	80.0	Near Ongole in Andhra Pradesh	—	4.5 (NDI)	Recorded at most of the Observatories. Felt at Bombay and Vijaywada.
Mar. 29	06	53	11 (CGS)	27.3	100.1	Yannan Province, China	33	4.9 (CGS)	—
Mar. 30	13	09	51 (CGS)	36.2	70.9	Hindu Kush	172	4.4 (CGS)	—

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REVIEWS

A Proposed Correlation between Elastic Strain Rebound Increments and Isostatic Gravity Anomalies by R.K.S. Chouhan* (Nature-Corres. 802-Geophsics-Illus.)†

Elastic strain rebound increments have been extensively considered by Benioff and others[1-6]. The results of their investigations indicate that strain rebound increments vary from region to region. Their characteristics are found to be linear and non-linear, tending to asymptotic and irregular. Such elastic strain rebound increments are simply a plot of the cumulative strain factor, $J^{\frac{1}{2}}$, versus time, where J is the seismic wave energy and can be calculated from Gutenberg-Richter's relation[7] as used by Benioff:

$$\text{Log}\, J = 9.0 + 1.8M$$

where M is the magnitude of the earthquake.

The elastic strain rebound increments of the Andaman Nicobar[5] and Bihar Nepal[5] regions shown in Figs. 1 and 2 are very similar. Both consist of two linear segments of the form $S = A + Bt$, where S is the strain rebound increments and where A and B are constants and where t is the time in Julian days. The activities and phases of strain recovery are also similar. The isostatic gravity anomalies given by the Survey of India and by Woollard[8] show that negative gravity anomalies of the order of—40 to—60 mgal prevail in both regions. This correlation is at first surprising, but an investigation of the elastic strain rebound increments has led me to enquire whether other similarities exist in these regions. The earthquakes used in plotting the elastic strain rebound increments are shallow focus shocks and thus the foci of all the earthquakes are located more or less in the crust. For this reason, I feel that some other similarities may exist in these regions.

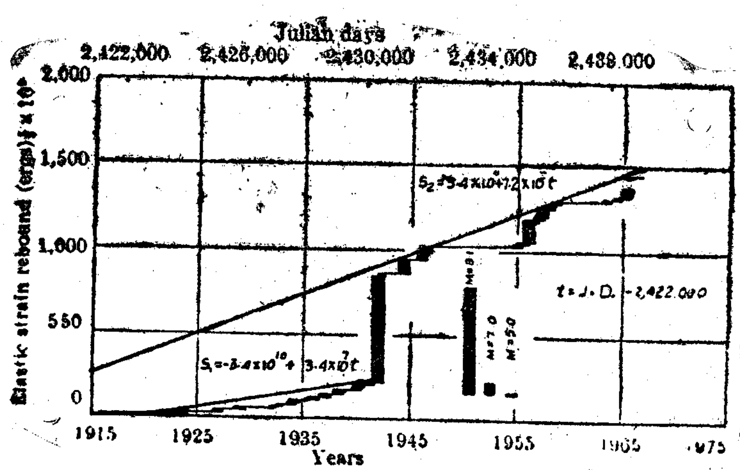

Fig. 1. Strain rebound characteristics of Andaman-Nicobar shallow focus earthquakes

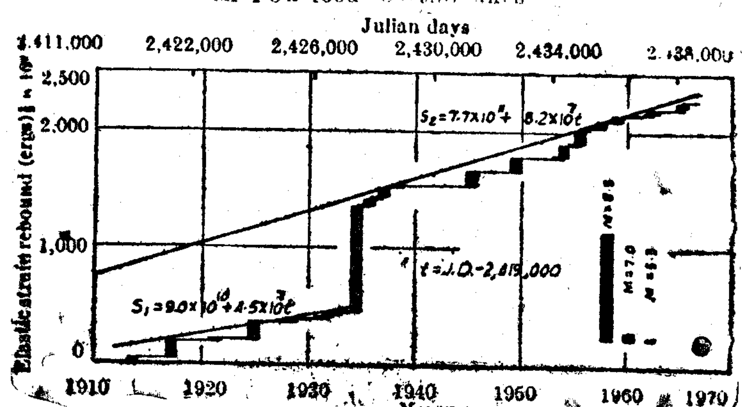

Fig. 2. Strain rebound characteristics of U.P., Bihar, Nepal earthquakes (shallow)

In this type of correlation the magnitude of the range should be about the same. Other regions which show similar elastic strain rebound increments are Dhubri[5], Kashmir[5] and North Japan[4], where they consist of a single segment of the form

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$S' = a + b \log t$, where S' is the elastic strain rebound increment, a and b are constants, and t is the time in Julian days. These three regions show identical earthquake activity, and the fact that the rate of strain release decreases with time indicates that creep occurs. Here the isostatic gravity anomalies are of the order of -60 to -80 mgal and thus fit the correlation. It would seem that a definite relation exists between the elastic strain rebound increments and the isostatic gravity anomalies.

Applying this correlation to the Sadiya[5] and Tonga Kermadec[4] regions, where only shallow shocks are used in the construction of elastic strain rebound increments, it is observed that the elastic strain rebounds are of a similar nature and that they are very irregular. If this correlation is valid, it can be said that the isostatic gravity anomalies prevailing in these two areas must be similar in size and sign.

This investigation has for the main been confined to India, but such a correlation can also be established for other regions of the world. I thank Dr. V.K. Gaur for his help and Dr. Jai Krishna, Director, School of Research and Training, for providing facilities for this work.

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Foundation Subjected to Vibration (Annotated Bibliography) by Shamsher Prakash and D. C. Gupta (School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee) Rs. 5-00.

The bibliography contains 119 references on the behaviour, design and construction of machine foundations. These are arranged subject-wise in seven different groups for the convenience of the user. All important publications on the subject upto date have been included. The bibliography will be useful to the scientific workers interested in the behaviour of mechine foundations in particular and profession of civil engineering and industrial organisation in general.

FOURTH WORLD CONFERENCE ON EARTHQUAKE ENGINEERING, SANTIAGO, CHILLE

JANUARY, 1969.

The International Association of Earthquake Engineering will hold the Fourth World Conference on Earthquake Engineering in Santiago, Chille in the week of January, 1969. The Following themes have been selected for the various Technical Sessions to permit coverage of the broad range of interest in Seismology and Earthquake Engineering :

SESSION THEMES

Observation in Recent Earthquakes
Seismicity and Ground Motion
Constructions Materials and Elements
Response of Structures
Design of Small Buildings
Soils and Soils Structures
Design of Large Buildings
Foundation and Soil–Structure Interaction
Design of other Structures
Design Critetria
Construction Practices
Research Programs
Repair and Strengthening Structures

All interested engineers and scientists are invited to contribute papers as per following schedule :

One page abstract is to be submitted, not later than January 1968 to **Professor Grahm H. Powell, Secretary of Technical Committee, Fourth World Conference on Earthquake Engineering, 416 McLaughlin Hall, University of California, Berkeley, California, 94720 U.S.A.** The tentative selection of papers will be made from these abstracts and the authors will then be invited to submit the complete paper prepared in accordance with the prescribed format detailed below by May, 1968. Final acceptance of the paper will be indicated by **June 1, 1968.**

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Since both preprints and proceeding, both abstracts and complete papers are to be reproduced directly from the original copy of the authors typescripts, it is essential that rules be established to assure uniformity of appearance :

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3. Paper : High quality white, size : "Post Quarto" $10\frac{1}{2}'' \times 8\frac{1}{4}''$.
4. Use one side of paper only, margins : $1\frac{1}{2}''$ binding margin, $\frac{1}{2}''$ right side, $\frac{1}{4}''$ top and botom.

5. Type "Elite" (12 letters per inch), single space between lines, double space between paragraphs, indent first line of each paragraph five spaces.
6. Mathematical terms, symbols, equations, and other features that can not be typed, should be neatly inserted into the text black ink.
7. Tables, photographs, diagrams or other illustrations shall not be included in the text, but each shall be prepared on a separate $10\frac{1}{2}'' \times 8\frac{1}{4}''$ sheet complete with captions, and shall be suitable for $\frac{1}{2}$ reduction in size in the final reproduction.
8. Each terms of a nomenclature or glossary shall be treated as a paragraph, double space from the previous term and indented five spaces on the left.
9. Footnotes shall be affixed on the bottom of the page where referenced by a number superscript enclosed by parentheses in the text. Use Roman numbers for footnotes; Arabic numbers for references to the bibliography.
10. "One-page" abstract submitted for paper selection shall be confined to one page of text and may be suplemented by upto four pages of illustrations prepared in accordance with item 7 above for reduction to one page, making the reproduced abssract one or two pages total. The format of the abstract, otherwise, shall follow this outline.
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    i) Title and author's name. Provide a footnote to indicate the position and affiliation of the author.
    ii) A 100 words, maximum, synopsis of the paper.
    iii) Glossary of terms, if required.
    iv) Body of the manuscript.
    v) Bibliography.
    vi) Figures, photographs, diagrams, etc.
    vii) Tables.
12. *Airmail* abstracts first, to reach the Secretary of the Technical Committee by January 1, 1968, or earliar. The same for the completed paper on the tentative acceptance based on review of the abstract by the appropriate paper selection committee.

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1. Only papers, which have not been previously published or offered for publication elsewhere, will be considered. The authors must agree not to publish a paper elsewhere when it is under cousideration and print in the Bulletin of the ISET.
2. Manuscript must be typed-written in English or Hindi with two-line spacing on one side of the paper only.
3. Three copies of the manuscripts must be submitted.
4. The paper should be limited to not more than 6000 words.
5. The use of the first person should be avoided, the writer being referred to as "the Author"
6. All mathematical symbols should be defined where they appear first in the text.
7. Drawings or sketches should not be included in or pasted on the pages of the manuscript and should be submitted separately with the paper.
8. Each article should be accompanied by an "abstract" of its subject matter, with special references to any conclusions, and it should not exceed 300 words:
9. A set of conclusions must be given at the end of the article.
10. Bibliographical references should be given as follows :—
    (a) In the text. the author's name and the year of publication or number in the list of reference cited should appear in parentheses as (Gutenberg 1959) or Gutenberg (1959), or Gutenberg (4).
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    Name, intials, year of publication.
    Title of work, Source (in full), volume number, page number (beginning) page number (end), date.

Example :—

Aggarwal. S L. (1964) "Static and Dynamic Behaviour of a Vertical Pile Subjected to Lateral Loads", Master of Engineering Thesis, University of Roorkee, Roorkee, 1964.

Arya, A.S. and Y P. Gupta, (1966), "Dynamic Earth Pressure on Retaining Walls Due to Ground Exitations", Bull., Ind. Soc. of Earthquake Technology, Vol. III, No. 2, pp. 5-16, May, 1966.

ILLUSTRATIONS

1. Drawing should be made on tracing linen or paper in dense black drawing-ink, the thickness of lines being consistent with a reduction to one half or less in the process of reproduction, details shown should represent the minimum necessary for a clear uuderstanding of what it is desired to illustrate.
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