EARTHQUAKE AS DYNAMIC PHENOMENON AND EARTHQUAKE RESISTANT STRUCTURES

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I. Seismic Ground Movements.

In order to carry out antiseismic computations, it is absolutely necessary to determine the seismic forces, which act on the structure in question. These forces are caused by the oscillatory ground movements, through which the earthquake becomes evident.

The resulting oscillatory movements, caused by an earthquake in a position T, located on the surface of the earth, is the combination of three seismic actions, the directions of which are through the vertical, horizontal and the perpendicular to the plane described by the first two directions.

These oscillations are neither pure harmonic nor damped harmonic. Their nature depends on the uniformity and continuity of the geological substrata, furthermore, apart from the elastic waves of the matter, through which the earthquake is dissipated, the motion also includes shock waves due to the disturbance of equilibrium in the seismic focus.

An accurate picture of the seismic movement of the ground in a certain location, is given by seismographs, which have recorded the movement in the three main directions, provided that seismographs are satisfactorily damped.

Let the analytical expressions for the curves thus recorded, be:

$$x=f_1(t) \qquad y=f_2(t) \qquad z=f_3(t) \qquad (\alpha)$$

Relations (a) express, in parametric form, the path of a particle on the ground. Should such a path be a straight line relation and the motion be a pure harmonic, provided that relations (a) represent simple harmonic motions of equal periods and without original phase, or equal phase. Really in this case the equations (a) take the form:

$$x=a.\sin\omega t \quad y=b.\sin\omega t \quad z=c.\sin\omega t \qquad (b)$$

From equations (b) it is evident that:

$$x=\frac{a}{c}z \qquad y=\frac{b}{c}.z$$

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It follows therefore, that the path of the particle coincides with the interception of two planes, the first of which contains the y axis and the second the x axis and also, that this straight line passes through the origin.

More-over the motion of the particle is pure harmonic, for the position of a point P (x,y,z) on the trajectory, as the latter is defined from equations, (b) is also defined by the radius of the collinear vector OP, which measures the displacement of the point from the position of equilibrium.

If r be the vector OP, we may write the relation

$$r = \sqrt{a^2+b^2+c^2} \cdot \sin wt$$

The above relations represent simple harmonic motion.

Should equation (a) not represent simple harmonic motions, the following transcendental equations are derived by cancelling out t

$$x = f_1(z) \quad y = f_2(z) \quad \text{or} \quad f(x,z)=0 \quad f(y,z)=0$$

The above transcendental equations, define two cylindrical surfaces, the first of which contains the y axis and the second the x axis. Since the generatrices of the two surfaces intercept each other at right angles, it is concluded, that the trajectory is a three dimensional curve.

Since therefore the curves recorded by seismographs do not represent pure harmonic motion, the trajectory of the ground particle under the influence of earthquake action is a three dimensional curve.

This curve can be represented by a graph. Seismograms are suitably magnified and axis t is subdivided to a number of small equal parts, while the corresponding coordinates of the seismogram are found. Combining the seismograms in pairs, two dimensional curves will result, which are the projections of the trajectory and at the same time the generatrices of the cylindrical surfaces mentioned above.

Examining now the consecutive positions, which the particle occupies along its trajectory, we can draw the following conclusions:

(a) The particle will obey an oscillatory law, but will not, during its motion generally pass through the equilibrium position. The opposite may happen, provided that the ordinates of all threeseismograms are simultaneously zero.

(b) In the region of the position of equilibrium the velocity tends to its maximum value,while in the region of maximum displacement the acceleration is a maximum. Thus we may otherwise state,that by the position of equilibrium the seismic energy is of kinetic nature,while by the position of maximum displacement,it becomes dynamic. The last observation will only apply to points along the trajectory, at which no shock effects have interfered.

2. Seismic forces and forces due to inertia.

Let us refer to two systems of coordinates having as their origins O and O' in order to measure the movement of a structure.At zero time, let the two systems coincide, while O be the system rigidly connected to the earth,the latter forming a system of inertia; let also O' follow the movement of the earthquake -imposed vibration.

As system O' will follow a motion of variable velocity,the structure will be acted upon by inertia forces,the result of which will be the elastic deformation of the structure,this being the relative motion of the structure.The motion of O' towards O,is the participated motion of the structure.The absolute motion is the sum of the two motions above.

According to the equation of the heavy mass and the inert mass,we can describe the phenomenon of inertia of the structure in terms of the accelerating system O' as phenomena of a dynamic field,the intensity of which at any moment is equal to acceleration developed by O'.Thus the structure at the time of the three seismic motion is,as refered to system O',under the influence of three dynamic fields of special nature,of which the two are horizontal and the third vertical.Their intensities are alternating and their magnitude varies with time. The determination of the dynamic magnitudes,developed on the structure,can be made using d' Alembert' s principle,which does not change the dynamic phenophenon to a static one,but allows the determination of dynamic magnitudes to be made by a static method.

We cannot change the problem to a static one,taking as static loads the maximum values of the seismic forces, as we shall develop below.

3. Seismic coefficients of the ground and buildings

To the seismic acceleration of the ground corresponds the seismic coefficient of the ground, while to the relative seismic acceleration of the building corresponds the seismic coefficient of the building,from which the seismic forces are determined. Since the relative acceleration id different from a building to another,according to its characteristics, it follows that the seismic coefficient of the building, which is always different from the seismic coefficient of the ground, changes from one building to another.

4. Simple harmonic motion

The seismic forces are usually determined on the assumption that the seismic motion is simple harmonic. Hence it is necessary to know the characteristics of the motion, to obtain an estimate of the errors which this assumption may lead to.

From the equations of simple harmonic motion we may derive the following useful conclusions :

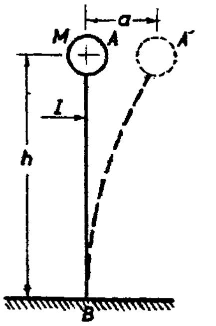
During the harmonic motion, the particle is displaced from the position of equilibrium on either side with a retarded motion and returns from the position of maximum amplitude with an accelerated motion.

According to the above, it is impossible for a body, or any elastic medium, to move from the position of equilibrium, describing from the beginning a harmonic motion, for it is impossible for a mass at rest to be set in motion under the action of a negative acceleration, i.e by a force having a direction opposite to the direction of the motion.

There must be a disturbance by forced motion, or a force, which will create the initial disturbance and once this initial cause is nullified, it is possible to have harmonic motion from the position of maximum amplitude, and thus the necessary conditions are fulfilled.

5. The fundamental Period of Structures.

We now examine the structure in Dr.No I of which the upper end A has been displaced from position A to position A'. A' is consequently freed to describe a



Dr. No 1

motion, the equation of which we shall determine. No allowance is made for damping and it is assumed that the displacement a = AA' is within the elastic limit of the structural material. The differential equation expressing the motion is :

$$\frac{d^2y}{dt^2} + w^2 y = 0 \quad (12) \qquad \text{wher } w = \sqrt{\frac{D'g}{M}}$$

and D' = static load applied horizontally at A inducing unit displacement.

The solution of the above equation is

$$y = a \sin wt \quad (13)$$

The motion is then pure harmonic of original phase $\frac{\pi}{2}$ and of a periode:

$T_\theta = \frac{2\pi}{\sqrt{\frac{D'g}{M}}}$ — By ignoring the influence of shear forces we obtain $D' = \frac{3EJ}{h^2}$ and $T_\theta = 2\pi\sqrt{\frac{Mh^3}{3EJg}}$

T is the fundamental period of the structure and it is independent of the original displacement, or any other external force.

Generally, once the direction of motion is altered, the moment of inertia of the structure will also alter amd therefore these will result in a different fundamental period.

6 . Motion of a structure when its base is set to simple harmonic motion.

We examine the same structure of Dr. No I and we set its base on a pure harmonic motion. According to para 4 the harmonic motion starts at the position of maximum displacement and will be of the form:

$$y_1 = a \cos w_1 t \quad (15)$$

The differential equation of motion of point A is

$$\frac{d^2y}{dt^2} + w^2 y = a w^2 \cos w_1 t \quad (16)$$

where $w^2 = \frac{D'g}{M}$

From the equation of absolute motion we derive :

$$y = \frac{a w^2}{w^2 - w_1^2} \cos w_1 t - \frac{a w_1^2}{w^2 - w_1^2} \cos wt \quad (17)$$

The equation of relative motion

$$y = \frac{a w^2}{w^2 - w_1^2} (\cos w_1 t - \cos wt) \quad (18)$$

When $w = w_1$ the above equation becomes

$$y = \frac{a}{2} \cdot wt \sin wt \quad (19)$$

which is the mathematical expression for resonance.

We should not confuse resonance with the variation of dynamic magnitudes as determined from equation (18), in which, however small differences for w w$_1$, we obtain definite values for dynamic magnitudes.

The equation of relative acceleration in terms of the fundamental period of the structure T_0 and the period of motion of the base, T_1 is :

$$\gamma = \frac{4a\pi^2}{T_1^2 - T_0^2} \cos \frac{2\pi}{T_0} . t - \frac{4a\pi^2 T_0^2}{T_1^2 (T_1^2 - T_0^2)} \cos \frac{2\pi}{T_1} . t \qquad (20a)$$

To find the maximum and minimum values of acceleration we should solve the trigonometrical equation

$$B \sin \frac{2\pi}{T_1} t - A \sin \frac{2\pi}{T_0} t = 0$$

This solution is obtained by a simple arithmetical method (given in the original paper) which we omit for the sake of brevity.

By this method, the determination of maximum and minimum acceleration, is reduced to finding the integer and positive values of Diophantes' binomial.

7. Arithmetical example

I) Data for structure in Dr. No I.

$h = 285$ cm $M = 15.200$ kg $E = 210.000$ kg.cm^{-2} $J = 20 \times 20^3 : 12 = 13.332$ cm^4

The motion of the base is determined by the equation

$$y_1 = 2 \cos \frac{2\pi}{0,8} . t$$

It is required to determine the maximum acceleration of A.

We find the fundamental period of the structure $T_0 = 1,3$ sec and maximum acceleration $\gamma_{max} = 271,04$ cm. sec^{-2}

2) For the same problem and for the same data but with an increased cross-section from 20X20 cm to 25X25 cm we find

$$T_0 = 0,83 \text{ sec} \qquad \gamma_{max} = 3.346,11 \text{ cm sec}^{-2}$$

The conclusion is that a small change in the cross section and therefore in the natural period resulted to thirteen times as much increase of the relative acceleration

8. The damping of the motion

I) Taking into account the damping of the motion, the differential equation of the fundamental oscillation of the structure is :

$$M \frac{d^2 y}{dt^2} + R \frac{dy}{dt} + Dy = 0 \qquad (26)$$

where R is the coefficient of damp~~i~~ing and D is the resist ance factor.

The solution is

$$y = \alpha . e^{-pt} \cos Wt \quad (27) \quad \text{where} \quad p = \frac{R}{2M} \quad \text{and} \quad W = \sqrt{\frac{D}{M} - \left(\frac{R}{2M}\right)^2}$$

The fundamental period, when damping is taken into account is

$$T_\theta' = 2\pi . \sqrt{\frac{4M^2}{4MD - R^2}}$$

2) Taking damping effects into consideration the relative motion of the structure at the base of which harmonic motion is imparted, can be expressed in terms of the following differential equation.

$$M\frac{d^2y}{dt^2} + R\frac{dy}{dt} + Dy = M\alpha w_1^2 \cos w_1 t \quad (29)$$

The solution to the above equation is

$$y = \frac{\alpha W w_1 \sin\varphi}{2pW} e^{-pt} \cos(Wt + \psi) + \frac{\alpha w_1 \sin\varphi}{2p} \cos(w_1 t + \beta) \quad (32)$$

where

$$tg\psi = \frac{p}{W} \cdot \frac{w_1^2 + w^2}{w^2 - w_1^2} \quad \sin\beta = \frac{2pw_1}{\sqrt{(w^2 - w_1^2)^2 + 4p^2 w_1^2}} \quad \text{and} \quad tg\beta = \frac{2pw_1}{w^2 - w_1^2}$$

By inspecting the roots of equation (32) it follows that when $p \neq o$, there will always exist a definite value for the root for any pair of values for w and w_1 In other words, resonance is impossible to occur. If however $p = o$ and $w = w_1$ we arrive again at equation (18), where resonance can occur.

9. Coefficient of Damping

From the fundamental equation of oscillation

$$y = e^{-\frac{R}{2M} t} \alpha . \cos Wt$$

we obtain that two consecutive displacements have a constant ratio equal to $K = e^{\frac{R}{2M} T_9}$ It is then observed, that each maximum displacement is decreased by a constant amount to give the following maximum displacement. Solving the above equation for T and substituting with the values already found for T we obtain

$$\frac{R}{2M} = \frac{L}{T} \quad \text{where} \quad L = \sqrt{\frac{4\pi^2}{\pi^2 \frac{\log^2 e}{\log^2 K} + 1}} \quad \text{and} \quad T = \sqrt{\frac{4\pi^2 M}{D'g}}$$

the corresponding values for

10. Arithmetical example

It is required to find the relative accelera tion for example No.I para 7, for L=0,3

We first determine the equation of relative motion

$$y = 3{,}136 \times e^{-0{,}23t} \times \cos\left(\frac{2\pi}{4{,}3} . t + 6°2'\right) - 3{,}132 \times \cos\left(\frac{2\pi}{0{,}8} . t + 5°23'\right)$$

The equation of acceleration is :

$$\gamma = 193{,}18 \cos\left(\frac{2\pi}{0{,}8} t + 5°23'\right) - 1{,}288^{-t} \times 73.59 \cos\left(\frac{2\pi}{1{,}3} t + 11°28'\right)$$

The maximum value of the acceleration is then found using a method similar to the one used in para 6.

We then find

$\gamma_{min} = -235{,}20\ cm.sec^{-2}$ as compared to $274{,}04\ cm.sec^{-2}$ the latter being the value for for the motion where damping is ignored.

II. The earthquake as a harmonic motion.

This assumption regarding the nature of earthquake action does not correspondto reality for the following reasons.

1) It confines the seismic action to longitudinal waves, while observations prove, that the critical moment at which the strength of a structure is tested, occurs only when all three seismic motions have been developed.

2) The longitudinal waves, as it is shown by seismo grams, are not in themselves harmonic waves for they include effects from shock waves to the latter of which maximum values for acceleration are due and they do not obey the sine rule. The maximum accelerations due to shock waves develop within a short interval of time. This time interval is about 4xIO when the period for the shock wave is 0,6 sec.

It is concluded therefore, that seismic motions are basically different to the pure harmonic motion. By applying harmonic analysis to the seismic motion, we can arrive at a primary harmonic motion, followed by a converging series of secondary harmonic terms, the periods of which are only fractions of the original period. The relative acceleration is thus found by superimposing harmonics, the form of which is given by equation (20a).

The conclusions derived in para 6 are used to arrive at the following statement : The change in the fundamental period of the structure in a given direction, has a direct bearing on the magnitude of relative acceleration. Such a phenomenon may have either favorable or unfavorable results, regarding the strength of the structure and it will depend on the nature of seismic motion, according to which it will develop in a given direction.

We may also draw the conclusion that on firm ground, rigid strustures(small period) are worst loaded, while on weak grounds, structures of low rigidity (large period) are again loaded as much. This theoretical conclusion is verified by a number of observations made in Greece during recent earthquake destructions.

I2. The accurate determination of seismic forces acting on a structure is so far impossible

It is impossible to express in terms of an analytical equation the form of seismographic curves. A determination can be achieved only by an approximation, using harmonic analysis and graphical methods of solution; furthermore, for a severe earthquake, it is not possible to obtain sufficient damping for the seismograms.

If such a difficulty exists for the analysis of seismograms recorded for aprevious earthquake, it is more difficult still to determine accurately the seismic forces for a future earthquake, the seismograms of which are unknown.

Even if we could determine with a satisfactory approximation the maximum seismic forces, it would not be enough to solve the problem of antiseismic computation, as a static one and this we shall explain in the following paragraph.

I3. Seismic loads.

In the case of static loads we assume that external forces are gradually and uniformly built up from zero to their final value and as a result the work done is converted to elastic energy without any kinetic energy.

It is totally different the case of dynamic loads, where they act instantaneously and of value different to zero (or even if they have initial velocity.) In this case the work done by external forces is partly converted to elastic energy and partly to kinetic energy. When the strain reaches a value equal to that produced by static loading, half of the work will have been transformed to elastic energy and the other half to kinetic energy. By the time the kinetic energy has been absorbed, the strain produced will be twice the strain produced by the equivalent static load. Furthermore, the motion is reversed and there shall be an oscillation with reference to the position of static strain. By damping the motion the structure finally comes to a rest and then the whole of kinetic energy will have been transformed to work done by the internal friction of the system.

In order to determine the maximum dynamic deflection, the influence due to damping effects at the beginning of motion is small and it can be neglected. The oscillation described in the above is determined from the relationship connecting the intensity and strain. When these relationships are linear and homogeneous i.e. when the law of proportion is fulfilled, then the necessary and satisfying conditions for the development of simple harmonic motion is fulfilled.

Referring to fig.I we have indeed this structure loaded at A by a horizontal dynamic load P and the differential equation when y is measured from the postion of static deflection is :

$$\left(\frac{dy}{dt}\right)^2 + \omega^2 y^2 = A \quad (34)$$ where $\omega^2 = \frac{Dg}{m}$ $A = \frac{Pf}{m}$ and f the static deflaction. Integrating, we obtain for A the equation of motion

$$y = f \cdot \cos \omega t \quad (35)$$

When y is measured from the position of equilibrium, the equation becomes

$$y = f - f\cos \omega t \quad (35a)$$

Multiplying both sides by the resistance operator D we determine the force, which corresponds at any time to the strain developed.

$$P_1 = P(1 - \cos \omega t) \quad (36)$$

Assuming that at time to the dynamic load is instantaneously zero and if $t_0 = \lambda \frac{T_0}{4}$ we obtain at time to :

$$P_1 = P(1 - \cos \cdot \frac{\pi}{2} \lambda)$$

Therefore for $\lambda = \frac{1}{3}$ $P_1 = 0{,}134P$ and for $\lambda = \frac{1}{10}$ $P_1' = 0{,}0123P$

Once the force producing the disturbance becomes zero, the strain will contin. until the kinetic energy is zero and there shall be an additional deflection f_2. At such a moment the whole work done by external forces will have been transformed to elastic energy. Namely

$$Pf_1 = (f_1 + f_2) D \frac{f_1 + f_2}{2}$$

and in terms of ratio :

$$f_1 + f_2 = 2f \sin \frac{\pi}{4} \lambda \quad (37)$$

for $\lambda = \frac{1}{10}$ $\quad f_1 + f_2 = 2f \times 0{,}0785$

It is then derived that, should the cantilever be loaded with a static load 2P, the resulting stresses will be thirteen times as much if compared to the actual ones.

It is not possible therefore, to consider dynamic loads of short duration equal to static ones.

In order to compute time within which the dynamic deflection is developed, it is necessary to compute the equivalen mass, taking into account the mass of the structure which, if concentrated at A, will give the same kinetic energy, to that given by the mass of the structure.

After an inspection is carried out, we arrive at the following expression for the equivalent mass.

$$m' = m \int_0^h \frac{f^2_{(x)}}{f^2} \cdot dz \quad (38)$$

where m is the mass of the structure per linear length, $f(x)$ the equation of elastic line and f the static deflection.

According to the above, the seismic action on structures is a dynamic phenophenon, which we cannot at any moment interrupt its action, converting it to static action.

Strains produced by earthquake are develoned in the form of elastic waves within the matter of the structure and strains, as well as intensity, do not only depend on position, but they are also periodical functions of time.

To control the resistance of an earthquake-proof structure, it is essential to determine the stresses of all points in terms of time and once the maximum value is found, the strength of the matter against oscillation is known, the resistance of each structure can be checked.

The theoretical approach to the problem of aseismic structures is, however, impossible to be achieved, for one should be able to determine the dynamic functions of seismic motions and also the relative acceleration at any point on the structure together with the resistance developed against the dynamic loading.

14. Aseismic structures computations

According to the above, we conclude that the methods used in practice, by which we transform dynamic loads to static ones lead to the peculiar situation, where we check by static means the structures using loads of unknown magnitude, which are in turn determined on entirely wrong basis, such computation is not only useless, but it may prove to be harmful for the structure.

It is quite possible, that following such a computation, dimensions are increased, thus decreasing the fundamental period and thus for a particular type of seismic action the relative acceleration is increased; furthermore, with an increased acceleration, will be multiplied by the increased mass of the structure. The fact that structures computed on this basis, do not suffer destruction from earthquakes, does not prove the suitability of the method, when the total strength of the structure is not ascertained, but such an ascertainment does not exist. Experience proves, that structurs designed by present methods, may in case of severe earthquake avoid destruction of the static frame, but partitions and non-bearing parts, may be destroyed to an extent of 50% of the total value of the building.

Regarding the safety of persons, it is imperiled by the destruction of not only the frame of the structure, but also the destruction of walls and pattitions.

From the examination of the subject regarding earthquake as a dynamic phenomenon, we are not justified to alter the fundamental period of the building, as we cannot foresee whether future seismic action will find such a change a disadvantage or an advantage.

We should therefore design the structures for the usual loads and keeping to the dimensions thus derived, achieveantiseismic strength by suitable constructional arrangements.

Such arrangements are briefly described below and explained in theoretica l terms, keeping in mind that, such structures, have been tested successfully during the recent earthquakes in Thessaly.

It has been observed that in districts of Greece which suffered earthquake destruction that houses built of stone or brick were susceptible to collapse easier than houses built of adobe (sun-dried bricks).

The resistance of the last type of building, is entirely due to wooden joints, arranged in horizontal layers and especially to those, which are arranged horizontally with in the side-door posts. Experience from earthquakes has led builders of village houses, who have no theoretical knowledge, to use correst arrangements, in order to achieve aseismic buildings, which are built of weak and friable materials, such as adobe (sun-dried bricks).

15. Aseismic systems based on design and their disadvantages

1. Single storey or two-storey buildings, made of natural or artificial stone-work, are constructed using four horizontal reinforced concrete closed framing. This type of construction does not secure the buildings from cracks, which are formed diagonally, by the side-door posts, due to shear forces, which cannot be resisted at the top and bottom of two consecutive horizontal framings. To place a fifth horizontal framing at the middle of the height of the side-door post is not sufficient. Sometimes they place vertical steel rods within the wall by the side-door post. This is again useless, for the diagonal cracks of the side-door post are due to shear stresses, which develop by bending and these cannot be resisted by the main reinforcement of the tensile side.

To build reinforced concrete walls is very expensive and such type of construction is rarely adopted for village houses.

2. For multistorey buildings with a reinforced concrete frame, the frame is designed on the assumption that the infilling walls and partitions are not capable to resist seismic forces; further still the frame is sometime reinforced by vertical reinforced concrete walls, which are capable to resist alone the seismic forces.

In both cases and apart from the method used to determine the seismic forces and the fact, that it is erroneous to convert seismic dynamic loads to static loads, the assumptions regarding the distribution of seismic forces is inaccurate and will only be real, if the infilling walls and partitions either collapse, or become unable to resist external forces. The distribution of external seismic forces between resisting and non-resisting parts of the structure follows the laws of physics and is determined on the common strain basis.

In the case of columns and because they are extremely flexible, they ~~will~~ only [illegible] to resist ~~only~~ a small percentage of the total force, while the biggest part will be resisted by the walls and partitions. For this reason the worst damage is always suffered by the last.

In case, also in which reinforced concrete walls are used, it is not possible to safeguard walls and partitions against destruction.

If P' be the force resisted by a brick wall and P be the force resisted by a reinforced concrete wall, we obtain the following expression according to the methods distribution of seismic forces:

$$\frac{P'}{P} = \frac{E'}{E} \cdot \frac{a'^3 b' (h'^3 + 3a^2 h^2)}{a^3 b (h^3 + 3a'^2 h'^2)} \quad (39)$$

In the above expressions non-dashed symbols refer to the dimensions and Youngs' modulus of the reinforced concrete wall, while the dashed symbols refer to the properties of the brick wall.

When the dimensions of the two walls are identical, we obtain $\frac{P'}{P} = \frac{6}{21}$ and if the shear stress of the reinforced concrete wall is 6 kg/cm^2 the corresponding shear stress in the brick wall is 1,7 kg/cm^2 i.e. more than three times as much in excess of the allowable stress.

If $a' = 4\,m$ $a = 2\,m$ $h = h' = 3\,m$ $b = b'$, then $\frac{\tau'}{\tau} = 0{,}421$ and if $\tau = 6\,kg.cm^{-2}$ $\tau' = 2{,}53\,kg\,cm^{-2}$ and therefore five times in excess of the allowable stress. Thus the damage suffered by walls and partitions in multistorey buildings is explained.

The diagonal cracks observed at side posts are justified by their weakness to resist horizontal forces.

A post or a wall of height h, length a and breadth b, acted upon by a load of constant magnitude P at their upper end, develops a bending moment according to the equation

$$\frac{Ph^2}{2EJ} = -\frac{Mh}{EJ} \qquad \text{from which} \qquad M = -\frac{Ph}{2}$$

The deflection due to bending is :

$$f_u = \frac{Ph^3}{12EJ}$$

and the deflection due to shear for $G = 0.4E$ $\mu = 1.2$ is $f_t = \frac{3Ph}{FE}$ It follows that $\frac{f_t}{f_u} = \frac{3a^2}{h^2}$. The strain due to shear is therefore greater than that due to bending, when $a > 0.577h$ and it becomes three times as much for a=h. Regarding the relation ship of maximum stresses, we obtain:

$\frac{\tau_{max}}{\sigma_{max}} = \pm\frac{a}{2h}$ When $a > 2h$ shear stresses are greater than bending stresses. Taking into account that the compressive strength of walls is greater by far than their shear strength, the comparison should be made with the tensile stresses due to bending. The latter is usually small, because of the presence of the vertical compressive loads and therefore the proportion of shear stresses, which is derived from the above relationship, does not reflect real conditions, which in fact is still worse.

Also the time during which the strain of the post takes place is substantially smaller, compared to the time required for the strain of the main columns to develop. In other words the post absorbs in the form of elastic energy the best part of the work of seismic forces and therefore the presence of diagonal cracks is explained, from the collapse of which the destruction of the rest of the structure follows.

Again the assumptions om which the distribution of seismic forces is made is inaccurate. These assumptions are based on the idea that slabs act as rigid diaphragms and hence static deflections of vertical bearing systems are equaled. The condition of maintaining the geometrical compatibility of the structure is thus fulfilled. These assumptions would be sufficient provided the loads were static. In the case of dynamic loads, i.e. seismic action, it is necessary that equal strains are developed within equal time intervals. From this condition it follows, that during the earthquake, a continuous redistribution of forces applied to vertical resisting system, takes place, following a periodic law. Only thus it is possible to have equality between deflections and time intervals during which the former are developed.

Also the computation of deflections as based on the principle of Navier-Bernoulli is inadmissible as far as walls are concerned for the condition $\sigma_x = \sigma_y = \tau_{yx} = 0$ is not valid.

16.Constructional arrangements of aseismic structures.

To face the problem of aseismic structures correctly, it calls for a systematic study of seismograms, using harmonic analysis for the curves, representing the seismic acceleration of the ground and to keep a relation with a systematic record and the results of previous earthquakes. The analysis, applied to a sufficient number of seismograms, would offer useful corollaries, regarding their nature, their primary harmonics their curves of acceleration and also the manner according to which secondary harmonics will alter.

In spite the different forms with which the seismic actions take place, it is possible to draw from the analysis the characteristics which will simplify the theoretical study.

Experience gained from past earthquakes has however underlined some conclusions, regarding the manner according to which damage and collapse occurs: these conclusions, together with observations on the strength developed by houses built on *adobe* , reinforced by wood-frames, has led to suggested constructional arrangements, which are justified according to the theoretical study stated in this work.

These constructional arrangements, which owing to the confined space of this summary, we cannot set out and giving only a very brief description, have been applied in Greece for the construction of lecture halls.

These lecture halls have endured several earth quakes and especially the earthquake in Thessaly of March 1957 (10°. Mercalli-Sieberg), without suffering any damage.

I) Buildings single-storey or two-storey :Their seismic resistance is ensured by avoiding the basic disadvantage of non-uniform strength for natural or artificial stone and using mortar of proportions 1:3 or 1:4.

At the same time, in order to make the wall effective and especially to avail the posts to resist shear forces, horizontal joints properly reinforced are placed at intervals. Such joints are different from horizontal framings. They are constructed by the builder while wall construction is progressing using the same mortar for the stone-work, and no shuttering, every 0,30-0,40 m (two cornerstones for stone work).

The horizontal bars (for a wall width of 0,50m their no is 3) which are placed along the length of the walls correspond to the stirrups of a beam under bending and are connected with transverse rods. The reinforcement is suitably bent by the end of the posts, at the corners and at the places where the wall is intercepted. The diameters of the bars are determined for 3 categories of seismic actions. In the region of small earthquake action the reinforced joints can be confined only to the height of posts and continued up to this height in all the internal and external walls.

In the case of two-storey building, the upper storey is constructed like the ground floor of the single storey, but the reinforcement of the ground floor is increased. The required cost in excess of the cost for a usual structure is 8-10%.

2) <u>Multi-storey buildings with o reinforced concrete frame.</u>

In this case walls and patitions are reinforced with reinforced joints, according to the manner described in the above. These joints are constructedin all walls, whether the latter are with or without openings, in order to reinforce the post which exists between the openings. The reinforcement of the joints is anchored in the columns which exist on either side.

Special measures are taken for basements without walls and also for the case of a view formed by columns, with continuous spans. It is also necessary that staircases occupy the entire height of the building and their walls be of reinforced concrete. The increase in cost which we obtain using these measures is not greater than 5-6% of the total cost.

Even in this case to check shear due to horizontal seismic forces at the head of the columns is not necessary. It follows from a relative study of the matter that for a seismic coefficient equal to λ in order that the columns may resist shears the condition $\eta \geq \frac{\lambda K_b - 12}{9000 - \lambda \sigma_s}$ must be fulfilled. In this relation $\eta = F_e : F_b$ K_b is the cube strength of concrete, and σ_s is the ultimate compressive stress in the steel. For $B=160$ $K_b=144$ $\sigma_s=2400$ $\lambda=0{,}16$ it must be $\eta \geq 0{,}0041$

The proportion 0,0042 is less than the minimum allowable limit.

3. Buildings of special purpose.

Large church buildings, theaters, museums and halls of any nature, where large spans are necessary, should be constructed with reinforced concrete frames, designed to resist normal loads and all their walls must be of concrete reinforced, with a double layer of grid reinforcement. Design against seismic action is not necessary.

For all building categories one must ensure with care all kinds of projections cantilevers and loose parts such as sills, chimneys, roof-tiles etc.

Special rules also apply to the foundations of these buildings.

17. Conclusions

1. From the technical point of view, earth quake action is a dynamic phenomenon of many-sided forms, while the time, at which it shall occur, or the form under which it shall develop, we cannot foretell.

2. Seismic forces, acting on structures, are determined from the relative accelerations, developed in them and are mainly depended on the relationships between the variable periods of seismic motions and the fundamental periods of the structures, on corresponding directions.

It follows, that to increase the dimensions of the system resisting the loads, may decrease the seis mic resistance of the structure, in case of an earthquake of special form.

It is possible that a building, which is not earthquake-proof but well constructed, to resist a severe earthquake and to collapse under the action of an earth quake, of smaller intensity..

It is also possible, that the building while it may resist an earthquake of a particular form, it may run the risk of damage, when the same form of earthquake will act in another direction.

3. The assumption according, to which the earthquake is a motion with simple harmonic motion characteristics, is not real and may lead to erroneous conclusions.

4. Seismic forces are of dynamic form and vary according to a periodic law, within short time intervals and therefore they cannot be assimilated to static loads, the magnitudes of which are determined in a way quite

arbitrary and inaccurate.

5. The static computations, used in practice based on wrong assumptions, bear no relation to the real anti-earthquake problem and give rise to trouble to both designers and builders, without ensuring the safety of buildings against seismic action, the result being to burden uselessly the financial sources of the State.

6. A satisfactory degree of safety against seismic action may be achieved by following simple constructional arrangements, based on experience and on the study of the subject from the dynamic point of view.

7. It is possible that substantial progress can be made on the anti-seismic problem, by a systematic study of seismograms, together with the corresponding effects observed on buildings which have suffered earthquake action.