COMMENTS ON THE NEW CHILEAN SEISMIC CODE FOR BUILDINGS.

A. Arias (I) R. Husid (I) J. Monge (I)

ABSTRACT.

The new Chilean Seismic Code for Buildings (Norma INDITECNOR 63.9 Cálculo Antisísmico de edificios) is discussed from the standpoint of the justification of its important prescriptions.

The Committee for this code was established before the destructive earthquakes that shook Chile in May 1960. When these earthquakes ocurred an almost final draft of the code had already been reached. The experience obtained in these shocks showed the need of substantial changes of approach.

A proposal that served as a basis of discussion at the Committee was made by two of the authors of this paper.

The Committee considered the prescriptions of some modern codes and had the help of an extensive comment in writing by Professors E.Rosenblueth and L. Esteva, from UNAM, Mexico . Oral comments were made by Professors D.E. Hudson and G.W. Housner (Caltech) and Professor R.W. Clough (U.C., Berkeley).

The new Chilean Code allows two methods of analysis for earthquake loads : a "static" and a "dynamic" method. The first one is based on theoretical studies of the response of simplified structures. Dynamical analyses allowed by the Code include modal analysis.

Both the static method and modal analysis are based on acceleration spectra that take into account the influence of foundation soil on structural response by using a modified version of Kanai's multiple reflection theory.

Mention is also made in the present paper of other aspects of the Code as : classification of buildings, mode superposition when using modal analysis and torsional effects.

(I) Professor of Engineering at the University of Chile.-

COMMENTS ON THE NEW CHILEAN SEISMIC CODE FOR BUILDINGS.

A. Arias (I) R. Husid (I) J. Monge(I)

Synopsis.-

The more important regulations of the new Chilean Seismic Code for Buildings are summarized. A brief justification for some of these regulations is given.

Introduction.-

After the Chillán earthquake of January 1939, more stringent regulations on earthquake resistant design of buildings were included in the Chilean General Building Code (Ordenanza General de Construcciones y Urbanización). With minor alterations, these regulations have been in force until recently (1).

In the year 1960, INDITECNOR (Instituto Nacional de Investigaciones Tecnológicas y Normalización) was studying a new code to incorporate the experience and knowledge gained in Chile and abroad until that time in the field of earthquake engineering. This project was left aside when new information on the behaviour of buildings during the two major earthquakes that shook the southern part of Central Chile became available (May 1960). Some of this information has been published in a special issue of the Bulletin of the Seismological Society of America, where further references may be found (2).

A tentative proposal for a new code was submitted to INDITECNOR's Committee in 1962 by two of the authors of the present paper (3). This proposal was accepted by the Committee as a basis for discussion. Several studies were done at the request of the Committee by some of its members to help the Committee decide on some specific matters, as for example, the acceleration spectrum to be used, the distribution of seismic forces in high rise buildings along the height, etc.

The Committee gave special attention to the prescriptions contained in several modern codes [4], specially the SEAOC Code (1960, 1963, 1965,1966) and the Mexico City Code.

Comments by Professors G.W. Housner, D.E. Hudson, E. Rosenblueth, L. Esteva and R.W. Clough were taken into account and are duly acknowledged by the authors in behalf of the Committee.

(I) Professor of Engineering, University of Chile.

The new INDITECNOR Code consists of two parts. The first one contains the mandatory regulations, the second is in essence a manual of recommended practice. The present paper refers essentially to the first part.

Scope of the Code.-

The new Chilean Code contains minimum prescriptions for the evaluation of seismic forces and the design of earthquake - resistant buildings. It does not cover neitner the design nor the evaluation of seismic forces acting on other kind of civil engineering structures. It was the opinion of the Committee that the earthquake engineering problems that arise in connection with structures other than buildings are at present insufficiently understood to warrant the possibility of making prescriptions of general validity about the analysis and design of such structures on a sound theoretical and practical basis.

Classification of buildings.-

Buildings are classified according to two criteria :

I. According to their destination or use, buildings are classified into three classes :

(a) Governmental and municipal buildings, public utility buildings (as post offices, electric power plants, police stations, fire stations, pumping plants, etc.); buildings of special importance in case of emergency (as hospitals, first aid posts, etc.) or whose contents are of great value (as libraries, museums, etc.) and buildings where a great number of people frequently congregate (as schools, stadiums, theaters, churches, terminal transport stations, etc.).

(b) Buildings for private or public use but where people do not usually congregate (department or office buildings, houses, hotels, restaurants, etc.); industrial buildings, warehouses, and buildings whose failure may endanger adjacent constructions belonging to this same group or group (a).

(c) Isolated buildings not classified in (a) or (b) (as for example granaries and stables) or whose failure does not endanger buildings belonging to groups (a) or (b); provisional buildings other than houses.

II. According to their structural characteristics, buildings are classified into three classes :

(d) Buildings in general, excepting those included in groups (e) and (f).

(e) Buildings having rigid diaphragms at each story level (ceiling of upper story included).

(f) Buildings having rigid diaphragms as in group (e), provided with rigid frames of adequate ductility to resist the total of the horiz^ntal seismic forces.

The total base shear used for design purposes depends on which of the classes is the building included in, according to the two criteria mentioned above.

"Static" and "dynamic" methods of analysis.-

The Code allows the use of two methods of analysis for the determination of seismic forces. These two methods are distinguished by the conventional names of "static" and "dynamic" methods.

The "dynamic" methos is allowed with no restriction as to the kind of structure to which it is applicable. Its application is mandatory for certain structures as mentioned below.

The "static" methos is of quite general application, but it is not allowed for certain structures to be mentioned presently.

In all cases buildings have to be analyzed and designed to resist earthquake forces acting in two orthogonal directions. The analysis is conducted independently for each of these two directions.

"Static" method.-

According to this method of analysis earthquake effects are represented as a set of static horizontal forces acting simultaneously at each story level, all in the same direction and sense. Analysis is done independently for each of two orthogonal directios.

The application of the "static" method is not permitted for buildings more than fifteen stories or 45 m. high with irregular distribution of masses and for stiffness along the height.

Base shear :

Base shear, Q_o is given by

$$Q_0 = K_1 \; K_2 \; C \; P \qquad (1)$$

where

K_1 : dimensionless coefficient related to building destination or use (see Table I).

K_2 : dimensionless coefficient related to structural characteristics of building (see Table II).

C . : Seismic coefficient (dimensionless) expressed by the formulae :

$$C = 0.10 \quad for \; T \leqslant T_0 \qquad (2)$$

$$C = 0.10 \frac{2TT_0}{T^2 + T_0^2} \quad for \; T > T_0 \qquad (3)$$

T is the fundamental period of the building when oscillating translationally in the direction considered in the analysis.

T_o is a parameter depending on foundation conditions at the site of the building.

The manual of recommended practice will contain information on the computation of the fundamental period and the selection of the value of T_o.

P is the weight above base level. This includes the total weight of the building above base level plus a fraction on the live loads. This fractio shall not be less than 0.50 for buildings where a great number of people usually congregate or where the maximum live load is highly probable (as museums, libraries, cinemas, archives, warehouses, industrial buildings, etc.). For other buildings the fraction of the live load included in P shall not be less than 0.25. Live loads on roofs may not be included in the computation of P.

The Code prescribes two lower bounds for base shear when the "static" method is employed :

a) Q_o shall not be less than 0.06 P.
b) C shall not be less than 0.06

The first of these limitations is also mandatory for the "dynamic" method. It was the opinion of the Committee that there was no need to precribe any limitation whatsoever as to maximum building height, because a minimum design base shear of 0.06 P imposes a practical and economical limitation on height.

Distribution of seismic forces along the height:

Once the base shear is known, the seismic forces at each story-level are computed by the formulae :

$$F_k = \frac{P_k A_k}{\sum_{j=1}^{n} P_j A_j} Q_o \qquad (4)$$

$$A_k = \sqrt{1 - \frac{Z_{k-1}}{H}} - \sqrt{1 - \frac{Z_k}{H}} \qquad (5)$$

where

F_K : horizontal force applied at the level of the K^{th} story (F_o=0)

P_K : weight at level of K^{th} story (dead load plus fraction of live load as specified above).

Z_K : elevation of K^{th} floor above base level.

H : total height of building above base level.

j,k : ordinal number of story counted from the base level upwards; for the base level this ordinal number shall be zero.

n : total number of stories above base level.

For buildings not exceeding five stories ($n \leqslant 5$) nor exceeding 16 m. in height ($H \leqslant 16$ m.), the values of A_K may be taken as

$$A_K = Z_K \tag{6}$$

Formula (4) is of general application, with no limitation as to building height or number of stories. This formula, for the case of a uniform distribution of masses along the height, corresponds to a parabolic shear diagram, with the vertex of the parabola at the top of the building, as first suggested by TUNG and NEWMARK [5].

Formula (4) and (5) is based on modal analyses of a substantial number of structures. Shear-buildings 2-, 3-, 4-, 10- and 20- stories high were included in these analyses; for some of these, foundation compliance was taken into account. The response of a uniform cantilever beam was also considered, both for the cases of pure shear deformation and pure bending deformation, with and without foundation rotation. Some of the results of these analyses have been published elsewhere [6] [7]. Further analyses were done, at the request of the Committee, to include extreme variations between stories, both of story masses and story stiffnesses.

The parabola mentioned above was obtained as an envelope of the shear diagrams for the same base shear for all structures considered. It was found that the form of the envelope is practically independent both of the fundamental period of the structures considered, and of the shape of the acceleration response spectrum used.

The spectra used for modal analyses were four : a) the two spectra prescribed by the Mexico City Code, one for "hard" soils and the other one for "soft" soils, b) the acceleration spectrum of the SEAOC Code (1960), c) a spectrum defined by

$$S_a = A = \text{constant for } 0 \leqslant T \leqslant 0.3 \text{ secs}$$

$$S_a = \frac{0.3\,A}{T} \quad \text{for } T > 0.3 \text{ secs}$$

For some of the structures included in the analyses, the responses to the El Centro (1940, May 18), Olympia (1949, April 13) and Taft (1952, July 21) earthquake records were computed, and the shears were included in the graphs drawn to obtain the envelopes.

Two methods for mode superposition were employed : a) by adding the absolute values of maximum modal shears at each story level, and b) by computing the square-root of the sum of the squares of maximum modal shears at each level. In the case of finite-degree of freedom structures, all modes were taken into account. For the two cantilever beams mentioned above, the first ten modes were considered. It was found that the shape of the envelopes is, for all practical purposes, independent of the method used in mode superposition.

The results obtained through formulae (4) and (5) have been compared by one of the authors of the present paper with the results of modal analysis in the case of four reinforced concrete buildings in Santiago, 17 to 24 stories high. All these buildings have shear walls; one of them has in addition a reinforced concrete moment resisting frame. In the computation of the stiffness-matrix of each of these buildings, both the shear and bending deformation of the shear walls was taken into account. It was found that the agreement between the results of modal analysis and those obtained through formulae (4) and (5) is satisfactory for design purposes.

Formula (6) is allowed for low buildings. This kind of distribution of the horizontal forces along the height coincides with the triangulan distribution of some foreign codes. Its use was allowed because of its simplicity.

Overturning moment :

It was found that the reduction of overturning moment arising from the fact that forces corresponding to different modes do not act in the same sense at all stories, is not substantial. For this reason a small reduction is allowed for buildings of more than three stories above base level. The overturning moment at each story level shall be the one computed from the forces F_K given by equation (4) multiplied by a reduction factor J defined as

$$J = 0.8 + 0.2 \frac{Z}{H} \tag{7}$$

where Z is the story elevation above base level and H is the total height of the building. For buildings less than four stories high, J = 1.

Horizontal distribution of seismic forces.-

In buildings with rigid horizontal diaphragms at all story levels, the shear Q_K at the $K^{th.}$ story shall be distributed between the lateral-forces- resisting elements in such a way to satisfy the equations of equilibrium and the assumption of rigidity of the diaphragms. Approximate methods of analysis, as given in the manual of recommended practice, are acceptable for this purpose. When computing the reactive forces that equilibrate Q_K, it shall be assumed that the shear Q is applied at a point (elastic center) such that only translation of the story occurs.

The stiffness and strength of the horizontal diaphragms shall be verified to show if the assumption of rigidity is tenable within the margin of error usually accepted in structural design. If this condition is not fulfilled, the flexibility of the diaphragms shall be taken into account when performing the distribution of the horizontal forces between the resisting elements.

If axial deformations of columns of rigid frames arising from seismic forces are not negligible, the effect of this deformation shall be taken into account when the distribution is done.

At each story a torsional moment given by

$$M_{t,K} = 1.5\, Q_K\, (e_K \pm 0.033\, b_K) \tag{8}$$

shall be considered. Here

$M_{t,K}$ = torsional moment at $k^{th.}$ story.

e_K = distance between elastic center of $k^{th.}$ floor and the line of action of Q_K.

b_K = dimension in plan of the $k^{th.}$ floor measured perpendicularly to O_K.

The reactive forces arising from this torsional moment in each lateral-force-resisting element shall be added to the reactive forces arising from the distribution of Q_K.

Every building shall be so designed that for each lateral-force-resisting element the reactive force arising from the torsional moment given by equation (8) does not exceed the reactive force arising through the distribution of Q_K. This prescription introduce a limitation on the horizontal distribution of stiffness, so that buildings with extreme excentricities are excluded by it. If for a given element the reactive force arising through torsion and the one obtained by the distribution of O_K have opposite senses, the said element is to be designed to resist the last said force minus half the first.

"Dynamic" method of analysis.-

As it has been pointed previously, the "dynamic" method is applicable to all kind of buildings, regardless of height, mass distribution and stiffness distribution.

Modal analysis :

Modal analysis is allowed, using an acceleration response spectrum given by

$$\frac{a}{g} = 0.10\ k_1\ k_2 \quad for\ T \leqslant T_0 \tag{9}$$

$$\frac{a}{g} = 0.10\ k_1\ k_2 \cdot \frac{2\,T\,T_0}{T^2\,T_0^2} \quad for\ T > T_0 \tag{10}$$

where T is the natural period of the mode considered and T_0 is the same parameter introduced in connection with the "static" method. A justification of this spectrum can be found in reference (8).

The computation of the masses of each story is done as in the "static" method. In the determination of the natural modes it is allowed to disregard the coupling of the modes due to damping. Formulae (9) and (10) already include the effect of damping; no further reductions of seismic forces through the consideration of damping effects is allowed.

Mode superposition is done according to the following equation

$$S = \frac{1}{2} \left\{ \sum_{i=1}^{r} \left| S_i \right| + \sqrt{\sum_{i=1}^{r} S_i^2} \right\}, (r \geqslant 3) \quad (11)$$

where S is the design seismic "force", in a general sense (shear, bending moment, etc.), at any given point of the structure, and S_i is the maximum seismic "force" corresponding to the $i^{th.}$ mode at the same point.

As implied in equation (11), it is mandatory to take into consideration at least the three lower modes.

It is known that the maximum response of multi-degree-of-freedom structures to real earthquakes lies in between the response obtained by adding the absolute values of maximum modal responses, and the one obtained by computing the square-root of the sum of the squares of maximum modal responses (9). For structures with a small number of degrees of freedom, the true maximum response approches the absolute value way of adding modal responses. For structures with a large number of degrees of freedom (greater than ten, say), the true response, for some cases studied, was found to approach closer to the quadratic way of superposing the modal responses, but the difference was high enough, specially at the upper stories, to justify the adoption of equation (11) that seems to be on the conservative side for this kind of structures.

If the base shear obtained through equation (11) is less than 0.06 P, all the computed magnitudes ("forces") are to be magnified proportionally as to comply with the requirement that the base shear should be 0,06 P as a minimum.

Other methods of analysis :

Other methods of "dynamic" analysis are allowed, as for example, computing the response of the structure to records of real earthquakes, or the use of artificially generated time series that simulate earthquakes. In all cases the minimum requirements mentioned above as to base shear is mandatory.

Other regulations contained in the Code.-

Separations.-

The new code includes prescriptions on separations between adjacent buildings. The experience obtained in Chilean earthquakes shows that considerable damage has been caused by the omissión of such regulations in the ordinance.

Some prescriptions apply to the expansion joints between parts of the same buildings. Special consideration is given to the constructive aspects of such joints, in order to prohibit generalized practice that consists in filling that joint space with boards and covering it with mortar.

The codes calls attention on the design of buildings with irregular horizontal cross-section, for instance H-, L-, T-, or V-shaped. It recommends that those buildings be divided by expansion joints, unless a careful design of the connecting diaphragms is made.

Relative inter-storey displacements.-

Secondary elements such as partitions and window frames usually rigidly connected to the building structure. When the building is rather flexible, considerable damage has occurred to these elements. There is a trend in order to introduce devices that connect them to the structure in a way that allows for a relative displacement between floor and ceiling.

The new Chilean Code regulates the relative inter-storey horizontal displacements, limiting them to 2% of the storey height for elements rigidly connected to the structure and to 4% of the same height in case that the forementioned devices are used. However, when the building is founded upon soft cohesive soil, the last figure is reduced to 3%.

Appendages.-

For the design of elements such as parapet walls, a horizontal force equal to half of the weight of the element is considered.

Minor structures attached to buildings, such as watertanks, chimneys, penthouses and others, must be designed for a horizontal acceleration equal to the floor slab acceleration. An exception is established for those buildings not exceeding 5 storeys or 16 meters in height and designed with a triangular distribution of base shear. For these buildings, the design horizontal acceleration for minor structures attached to them is increased by one third.

Eaves and balconies must be designed for a vertical force equal to dead load plus live load, increased by 30% and with the same allowable stresses specified for static conditions.

Allowable stresses and design procedure.-

The design of structures for lateral forces in Chile assumes that structural members remain elastic for the forces prescribed by the code. Regulations for steel structures allow for a one third increase in allowable stresses in case of wind or earthquake forces; for reinforced concrete structures, the allowable increase is about one sixth.

Nevertheless, the Committee was conscious of the fact that during destructive earthquakes the elastic limit is exceeded. This is recognized by allowing a reduction in the base shear for structures where all lateral forces are taken by moment resisting spatial frames designed as to obtain adequate ductility.

A change in actual design regulations for reinforced concrete structures is being considered, in order to introduce the anelastic design as an alternate procedure.

Other code requirements.-

Special mention is made of procedures for repairing damaged buildings. The approval of city authorities is necessary for the execution of such repair work.

For buildings exceeding 25 storeys, the owner shall provide for the installation of two-strong motion, three components accelerographs and a number of seismoscopes, one for every other storey. The Department of Geophysics, Seismology and Geodesy of the University of Chile shall be in charge of the maintenance and operation of these instruments.

TABLE I
Coefficient K_1

Use or destination of buildings.	K_1
(a)	1.2
(b)	1.0
(c)	0.8

TABLE II
Coefficient K_2

Structural characteristics of building	K_2
(d)	1.2
(e)	1.0
(f)	0.8

References.-

1.- BERTLING, H.
D Development of earthquake-proof construction in Chile. First World Conference on Earthquake Engineering, Berkeley, Cal., 1956. Paper N.20

2.- HOUSNER, G.W. (editor)
An Engineering Report on the Chilean Earthquakes of May 1960. Bulletin of the Seismological Society of America, (special issue), vol. 53, N°1, February 1963.

3.- ARIAS, A. & R. HUSID
Proyecto de Norma de Cálculo Antisísmico de Edificios. Revista del IDIEM, vol. 1, N°2, Junio 1962. Santiago-Chile, pp. 121-146.

4.- INTERNATIONAL ASSOCIATION FOR EARTHQUAKE ENGINEERING.
Earthquake-Resistant Regulations. A World List, 1963.

5.- TUNG, T.P. & N.M. NEWMARK
Shears in a Tall Building subjected to Strong Motion Earthquakes. First World Conference on Earthquake Engineering, Berkeley, Cal., 1956, Paper N. 10.

6.- HUSID, R.
Estudio Teórico sobre la Repartición Vertical de Fuerzas Sísmicas en Edificios. Civil Engineer Degree Thesis, University of Chile, 1960.

7.- ARIAS, A. & R. HUSID
Fuerzas Sísmicas en Edificios. First Argentinean Conference on Earthquake Engineering, San Juan and Mendoza, April 1962.

8.- ARIAS, A. & L. PETIT-LAURENT
Un modelo teórico para acelerogramas de temblores fuertes. Revista de IDIEM, vol. 4, N°1, May 1965.

9.- MERCHANT, H.C. & D.E. HUDSON
Mode superposition in Multi-Degree of Freedom Systems Using Earthquake Response Spectrum Data. Bulletin of the Seismological Society of America, vol. 52, N°2, pp. 405-415, April 1962.