ASEISMIC PROVISIONS FOR THE FEDERAL DISTRICT, MEXICO

By

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Introduction

Before the earthquake of 28 July 1957 Mexican engineers knew that the Federal District Building Code was obsolete, that it was systematically violated, and that there was more need for research on structural responses to earthquakes. (1) But no one realized the tragic consequences that the inadequacy and ineffectiveness of the code and our ignorance in matters of aseismic design were to bring about. ·

Despite the aftermath, a true perspective must deem the lesson learned at a low cost indeed. Reaction was immediate; authorities took effective steps; a unique opportunity arose for gaining experience and even some insight into structural behavior; and the event attracted some of the best engineers from other parts of the world, foremost among whom were R.W. Binder, G.W. Housner, and the late Murray Erick, who awakened consciousness of the pertinent variables in aseismic design.

Emergency Regulations

One week after 28 July, the Building Department appointed a committee to write an emergency code. Regulations were drafted in three weeks and immediately enforced. They were later revised. Essentially unchanged, they were enacted in a presidential decree 12 September 1957. The chapter on aseismic design of the code extant at the time was abrogated.

The 1942 code (1) called for base shear coefficients of between zero and 0.10 depending only on the importance of the building, with 33 percent increase in allowable stresses. Office and apartment buildings should be designed for 0.025 g. Acceleration should be assumed uniform throughout the building. Structures up to 16 m (52.5 ft) tall needed not be designed against earthquake forces. The code included no provisions on torsion, none on masonry walls and partitions, and did not leave room for dynamic analysis. It was often applied in conjunction with the portal method of analysis with no regard for relative rigidities.

The earthquake brought to the foreground several points: (2,3,4)

1. Motions with recurrence period of 50 years caused far greater
    accelerations than the code called for, specially in the upper
    stories.

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2. The Valley of Mexico exhibited prevailing ground periods. Flexible structures on soft soil and rigid structures on hard ground were more vulnerable. This was particularly true of the former, because of the prevalence of distant epicenters and hence long periods.

3. Due regard should be given the question of relative rigidities, their relation with torsion and the contribution of masonry walls and partitions.

4. Great precision could not be expected in calculation of rigidities and torsional effects, particularly when masonry walls and partitions act with the rest of the structure.

The emergency regulations reflected these points. Aseismic design was required for all except the most unimportant buildings, independently of their height. Base shear coefficients were stepped up to between 0.05 and 0.10 for ordinary buildings, depending on the type of structure and its location in the Valley of Mexico (whether on the lake, transition, or hilly zone), and doubling these values for buildings of exceptional importance.

A linear variation of horizontal accelerations was adopted, from zero at the base of the structure to a maximum at the roof. Parapet walls and appurtenances were assigned an acceleration of 0.5 g. Setbacks received no special consideration.

It was specified that the computed eccentricity in torsion be increased by an "accidental" value, equal to $\pm$ 0.05 of the floor dimension measured at right angles to the acceleration. This applied to all buildings except warehouses, for which the coefficient 0.05 was increased up to 0.07 to cover unfavorable distributions of load. Besides, in no case should the design eccentricity be less than half of the maximum found in stories below the one under consideration, nor the torque be less than half of the maximum found in all stories above the one being analyzed.

It was felt that the prevalence of long periods in the lake zone precluded reductions in overturning moments. No provision was left in to allow such reductions. Shortly it became apparent that design complying with the regulations resulted in very costly structures. A round-table discussion was held at the Building Department with some consulting engineers. Reduction coefficients of up to 40 percent were adopted.

Thought was given to the possibility of requiring that all masonry be reinforced. Economic considerations discarded this, since unreinforced masonry costs one tenth of what it does in the U. S. while reinforcing steel can be obtained for about the same price as in that country.

Unreinforced masonry was to continue in general use. It was clear from the 1957 earthquake that such walls were prone to break leaving

the rest of the structure unaided to resist further motion, be it due
to wind, the some quake, or aftershocks. A check on the conditions of
the unaided frame was felt mandatory. Accordingly, the regulations
required two analyses: one taking in consideration unreinforced walls,
and the second assuming that these elements had failed and using a lower
base shear coefficient.

In conjunction with conventional design (essentially 1940 American
practice) the regulations allowed doubling most working stresses under
combined static seismic loads. For concrete in diagonal tension, only
a 33 percent increase was permitted; structural grade reinforcement could
be designed at 90 percent of its yield point; and other grades at 80
percent of their nominal limits of elasticity. The regulations also
permitted plastic analysis and design with a load factor of 1.5 in flexure
and 1.2 in axial compression.

It was felt that with high base shear coefficients and high working
stresses a well balanced design would be attained. For example, points
of counterflexure shifted thus over a wider range than with the conven-
tional small lateral loads and 33 percent overstress. Also, cross brac-
ing did not turn out too flimsy in relation to the columns. (2)

The 33 percent overstress was retained for diagonal tension in
concrete since the revision was to cover no more than aseismic design
and it was realized that concrete members were being underdesigned in
shear.

Drift was limited to 0.002 of story height except for lightweight
steel roofs in factory constructions, for which no limitation was stated.

Buildings over 45 m (140 ft) tall required dynamic analysis, the
details of which the regulations left unspecified.

The 1959 Revision

Early in 1959 the Building Department reorganized its Committee on
the Building Code. It was the general consensus that the code merited
thorough revision rather than mere polishing. Several subcommittees
were appointed to study specialized topics, including one on aseismic
design.

Most engineers who had applied the emergency regulations felt that
ordinary buildings were being penalized in excess because some vulner-
able atypical structures had been subjected to very high accelerations
in 1957. Many who had becried Fate, resonance, and the earthquake's
violence as responsible for the havoc had an opportunity to take a close
look at the damaged structures when repairing them. In most ordinary
buildings they had to ascribe damage to poor construction practices
and oversimplification in analysis and design. High seismic coefficients
were justified for atypical structures such as those resting on a single
row of columns, but design of ordinary buildings could be liberalized,
provided due attention was given the questions of relative rigidities,

of pounding with neighboring buildings, and of apparently minor structural details. This situation in aseismic design was probably decisive in leading to revision of the entire code.

There follows an annotated translation of the proposal made by the subcommittee, as it stood in December 1959. It may suffer minor changes before final approval.

1. Nomenclature. Each symbol is defined where it first appears. The most important are:

a = coefficient used in dynamic analysis (dimensionless);
c = V/W = base shear coefficient (dimensionless);
F_i = horizontal force at floor i (ton);
f'_m = least of brick or mortar compressive strengths, determined from standard tests (kg/cm^2);
H = elevation above base of structure (cm);
h = story height of wall panel (cm);
L = length of wall panel (cm);
n = number of stories of building considered (dimentsionless);
T = natural period (sec);
V = horizontal shear at base of building (ton);
W = weight of building (ton);
W_i = weight that is assumed concentrated at floor i (ton);
x = maximum computed horizontal displacement at height H (cm).

2. Zoning. According to subsoil characteristies the Federal District is divided in two zones conforming to the chapter on foundations.

The transition zone was incorporated in the lake zone together with all recent uncompacted fills in the hilly area. Despite statistical evidence favoring distinction between lake and transition zones, seismic intensity in other cities has been particularly violent near the slopes of hills and mountains. Conceivably a different earthquake from that of 1957 might make the transition zone its favorite prey. Moreover, parts of this zone were vacant of tall buildings to stand as witnesses of local intensity.

Local inexperience with loose fills suggests classifying them together with soft ground; they are so grouped in the chapter on foundations.

3. Classification of Buildings. Considering their intended use, constructions are classified in three groups:

Group A. Government, municipal, and public buildings (as pumping stations, powerhouses, telephone exchanges); those whose functions are especially important immediately following a seism (as hospitals); those containing articles of exceptional value (as museums); and those in which there is frequent conglomeration of people (as schools, stadiums, theaters, temples, terminal communication stations).

Group B. Buildings intended for private habitation or public use where conglomerations are uncommon: hotels, apartment and office buildings, industrial buildings, gas stations, restaurants, etc. Also fence walls with height exceeding 2.50 m, constructions housing costly goods or equipment, and those whose failure could endanger other buildings in this group or in group (A)

Group C. Isolated constructions not requiring the intervention of a registered engineer or architect and the failure of which, from earthquake effects, would not normally damage human beings, costly goods or equipment, or structures in the first two groups.

The following classification is based on structural characteristics.

Type 1. Structures having, at right angles with the direction analyzed, two or more elements capable of resisting horizontal shear, and whole deformations, under lateral loads in the direction in question are essentially due to flexure in structural members. It includes buildings having frames that consist of girders (or trusses or flat slabs) on steel or reinforced concrete columns, provided each frame is capable of resisting in all stories at least 50 percent of the design shear it would be called upon to resist if isolated from the rest of the structure, not requiring therefor the aid of walls or diagonal bracing. Floors and roofs must be sufficiently rigid and resistant to distribute the lateral forces among elements of various flexibilities.

Type 2. Structures whose deformations under lateral loads in the direction considered are due essentially to shearing stresses or axial forces in the structural members. Included are buildings supported by bearing walls only and those whose frames are incapable of resisting by themselves in each story 50 percent of the design shear. Each frame in this group must be able to resist at least 25 percent of the design shear.

Type 3. Elevated water tanks, chimney stacks, and all structures supported by a single column or row of columns perpendicularly to the direction analyzed, or whose columns are not tied at the roof and floors through elements of sufficient rigidity and strength to distribute the horizontal forces among columns of different flexibilities.

The need to differentiate groups (A) and (B) was made plain in 1957. The children's hospital and the country's largest hospital were severely damaged. Had the quake occurred at another time of day it would have been impossible to accomodate enough patients. The most conservative building code will not afford absolute protection against the most violent earthquake possible, nor is it clear that such an upper limit exists. (The most plausible bases for establishing an upper limit to the magnitude of earthquakes (5) are contradicted by the authors in their analysis of the Lisbon earthquake.) The fact that all authorities agree on seismic regionalization in terms of regional seismicity — not only of local

geology — implies that they admit designing for magnitudes smaller than the maximum possible, even if that maximum is finite. Hence one must contemplate collapse, however remote. Clearly some buildings should be protected against collapse more dearly than others. Finally, in any probabilistic approach to structures, design loads must depend on the importance of the structure, especially the most unpredictable types of load.

The division into types classifies structures in a decreasing order of their ability to absorb energy. It is influenced by the code approved by the SEAOC in October 1958, [6] as well as on observations of damage inflicted by the 1957 quake. Differences with the SEAOC code reflect personal opinions, since neither system is based on quantitative information.

Experience with the emergency regulations indicates that the double check in stability computations — with and without walls —, although desirable, was too demanding. The requirements that each frame of structures type 1 resist at least 50 percent of its own lateral load was left in as second best choice and within reach of most structural engineers. In part, also, it takes care of miscalculation of relative rigidities. The last comment applies also to the 25 percent minimum for structures of type 2.

4. Allowable Stresses. When using elastic methods of analysis and design for the combination of gravity and earthquake loads the following increases in allowable stresses are permitted with respect to those specified elsewhere in this code for gravity loads alone.

(a) In timber and structural or reinforcing steel, 50 percent.
(b) In other metals, concrete, and masonry, 33 percent.
(c) In the cohesion of soils, 100 percent.*
(d) In the angle of internal friction of soils, none.*

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* Subject to revision by the subcommittee on foundations.

Some allowable stresses for static loads are as follows. Rolled structural grade steel in tension or compression, 1400 kg/cm^2 (20 ksi); other grades, 0.6 fy (fy at 0.2 percent offset). Structural grade bars as main reinforcement or stirrups in concrete, 1265 kg/cm^2 (18 ksi); other grades, 0.5 f$_y$. Concrete in compression due to flexure, 0.45 f$_c'$; in compression due to axial load, 0.28 f$_c'$; and in diagonal tension or shear, the average stress = 0.25$\sqrt{f_c'}$ in kg/cm^2 (0.94$\sqrt{f_c'}$ in psi) plus a term depending on longitudinal reinforcement and shear span, but not to exceed 0.45$\sqrt{f_c'}$ (1.7$\sqrt{f_c'}$ in psi).

Moduli of elasticity are 2.1x10^6 kg/cm^2 (30x10^6 psi) for steel, 15,000$\sqrt{f_c'}$ in kg/cm^2 (56,800$\sqrt{f_c'}$ in psi) for concrete, and 250 f$_m'$ for masonry. The last value is based on tests with materials from the Federal District. Moduli of rigidity for concrete and masonry are 0.4 of the moduli of elasticity. Use of empirically determined moduli in conjunction with standard strength of materials is expected to give good results for most cases on the basis of available test data. [7]

5. **Base Shear Coefficients.** The base shear coefficient in the ratio of the horizontal shear V at the base of the building to the weight W of the building. Live loads assumed in the evaluation of W are those tabulated in the chapter on loads.
In static analysis of buildings belonging to group (B), the following minimum values of C shall be used.

Type of structure	Zone of high compressibility	Zone of low compressibility
1	0.06	0.04
2	0.08	0.08
3	0.15	0.10

For buildings in group (A) these values shall be multiplied by 1.3.
For those in group (C) aseismic design is not required.

The coefficient 0.06 was derived from the base shear registered in the 43-story Torre Latino Americana in 1957 (between 0.03 and 0.04 of its weight, (8)) somewhat increased to take into account the length of natural periods in this structure. The coefficient 0.15 responds to the fact that the two cases of resonance in 1957 indicated values greater than 0.10 for structures type 3. (4) The base shear coefficient of 0.10 for structures type 3 on soil of low compressibility is based on experience in California, allowing for lesser seismicity and higher working stresses in Mexico City. Other values in the Table were fit in once these had been agreed upon.

The 30 percent increase for group (A) is much smaller than the 100 percent in the emergency regulations and even smaller than the variations required by the 1942 code. The fact that such a small increase is now thought adequate comes from the feeling that the coefficients are quite high as they stand, plus experience gained in applying the emergency regulations, which led to the conclusion that greater variation would excessively increase the cost of structures. There was besides a qualitative consideration of the results of probability analyses concerning the maximum response of damped systems to earthquakes of given intensity and the statistical distribution of intensities in time.

6. **Selection of Method of Analysis.** The method of Art. 7 is acceptable when the following requirements are simultaneously met.

-   In each story at least 75 percent of vertical loads is supported by walls joined through a continuous slab.

-   There exist in the perimeter at least two bearing walls, parallel to each other or forming between them an angle not greater than 20 degrees. These are joined through the slab for a length at least equal to half that of the building measured along these walls.

-   The ratio of height to width of the building does not exceed 1.5.

The aspect ratio of the base of the building does not exceed
2.0 except if, for aseismic design, it can be assumed divided
into independent segments with maximum aspect ratio within this
limitation and each segment is earthquake resistant in accordance
with Art. 8.

In all other cases recourse must be made to the static or the dy-
namic method, save that the static method is not applicable when in any
story torsional eccentricity exceeds 20 percent of the plant dimension
measured parallel to the eccentricity.

The need for a simplified method applicable to inexpensive construc-
tion arose mostly from the fact that the 1942 code did not require
aseismic design of buildings up to four stories tall. Of the six total
collapses of 1957, three were in this range as well as many cases of
major damage; hence, the emergency regulations required aseismic design
of practically all structures. Since many houses and four-story build-
ings are designed by registered practitioners with little academic back-
ground, there was need to simplify matters in this range. Something
similar to the present article had been unofficially approved by the
Building Department shortly after issuance of the emergency regulations.

Analysis of more than 100 buildings (each with 2-16 degrees of free-
dom but no torsion) by the methods of Arts. 8 and 9, disclosed not a
single simple criterion to discern the need for dynamic analysis. In
most cases the static method gave horizontal shears greater than their
dynamic counterparts. The taller structures systematically gave dynamic
shears considerably smaller than those computed statically. Only in
systems having torsional eccentricity variable with height
were important errors on the unsafe side found when applying the static
method. Accordingly it was decided not to exact a dynamic analysis at
all, but to leave the choice up to the structural engineer, except in
cases of high torsional eccentricities.

7. Simplified Method of Analysis. Lateral displacements, torques,
and overturning moments may be ignored. It suffices to check that
the total shear acting in each story, computes as in Art. 8, does
not exceed the sum of the projected shear strengths of the bearing
walls on axes parallel to the directions in which the acceleration
is assumed to act. The check shall be made along two mutually per-
pendicular directions.

In wall panels whose ratio of story height, h, to length L, exceeds
1.33, the allowable stress shall be reduced by a factor $(1.33L/h)^2$.

For purposes of this article the shear strength of bearing walls,
already increased because it applies to lateral loading, shall be
taken as 0.5 kg/cm^2 (7 psi) for walls of light-weight brick, and
1.5 kg/cm^2 (21 psi) for those of burned clay, unless if, by means
of the tests specified in the chapter on masonry construction, the
value of f_m^* is determined. In that case the shear strength shall
be taken equal to 0.06 f_m^* allowing for no increase derived from
vertical loads.

Walls in question shall comply with the chapter on masonry. It shall be unnecessary to satisfy Art. 15 in the present chapter.

The reduction factor tends to take estimatively in consideration the decrease in rigidity with increase in the h/L ratio.

8. Static Analysis. Design shears at different elevations in a building shall be computed from the assumption of linear distribution of horizontal accelerations, varying from zero at the base of the structure to a maximum at the top, in such a manner that the ratio V/W at the base equal the value of C specified in Art. 5. Accordingly, the horizontal force at floor i is given by the expression

$$F_i = CW \frac{W_i H_i}{W_1 H_1 + W_2 H_2 + \ldots + W_n H_n}$$

Tanks on buildings, constructions not forming an integral part of the structure, and all other elements whose stability depends on local acceleration (not on shear or overturning moment) shall be designed for an acceleration not smaller than twice the one obtained from the forgoing paragraph nor than 0.15 times gravity. The requirement applies to cantilever walls, signs, ornaments, windows, partitions, curtain walls, façade veneer, etc., and to horizontal diaphragms that transmit the inertia forces of the masses they carry.

For design purposes, overturning moments may be reduced, but for each frame or group of resisting elements it shall not be taken smaller than the design shear at the elevation considered times its distance to the center of gravity of all the corresponding masses above.

Torsional eccentricity must be computed in all stories. The design eccentricity shall be taken equal to 1.5 times the computed value* plus or minus 0.05 times the maximum plant dimension at the elevation considered (accidental eccentricity), measured at right angles with the shear (Fig. 1). For each structural member, that sign of the accidental eccentricity shall be taken which gives the most unfavorable condition. If uniformity of live loads and rigidities can be assured, the accidental eccentricity may be lowered slightly.

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*Subject to revision in the light of further dynamic analyses before submission for final approval.

Provisions for appurtenances are less stringent than what other codes require. Besides, it was felt that local accelerations should be taken in consideration. The minimum of 0.15 g corresponds to type 3 in the table of Art. 5, for a parapet resting on the foundation can be regarded as a structure of this type.

The dynamic analysis of several buildings referred to above did not disclose the need to include special provisions for setbacks in buildings up to 16 stories tall. For much taller structures, dynamically computed shears would be smaller than those found in static analysis.

Explicit consideration of natural periods in this method was not deemed appropriate. Empirical formulas for the fundamental period can well err over 100 percent; and if this period is to be computed by trustworthy methods, a little more effort will permit application of the dynamic method.

Allowable reduction in overturning moment is conservative. It amounts to about 25 percent at the base of the building (near 30 percent in extreme cases), varying to zero at the uppermost story. The latter condition is imposed by equilibrium. This conservative stand in connection with overturning was based partly on the fact that the code's design criteria for columns and for distribution of soil pressure (Art. 19d) are somewhat liberal and partly on the predominance of distant-focus (hence long-period) earthquakes in the Federal District. (9)

Accidental torsion is specified much in the same manner as in the emergency regulations (an _additional_ rather than a minimum eccentricity as in the SEAOC code). However, no distinction is made between warehouses and other buildings since live load distribution contributes far less to accidental eccentricity than chance variations in rigidity and errors in its computation.(9) The 1.5 factor is an approximation to Housner's dynamic magnification of torsion in single-story buildings.(10)

Emergency regulation requirements for torsion in terms of that computed in stories other than the one considered are qualitatively correct. (9) They were dropped for the sake of simplicity, since dynamic analyses showed that these effects were quite small when the maximum eccentricity does not exceed 0.20 of the plant dimension.

9. Dynamic Analysis. Any dynamic method of analysis is permissible if it is consistent with the nature of earthquakes and behavior of ground and structure and fulfills the restrictions in this article.

If the structure is analyzed computing its natural modes of vibrations, it shall be assumed that the ground suddenly undergoes a constant acceleration equal to aC times gravity. Here C are the coefficients of Art. 5 and a the values obtained from the following expressions, in which T represents the natural period in seconds.

(a) In the zone of low compressibility, for $T \leqslant 0.5$, $a = 1.0$, and for $T \geqslant 0.5$, $a = 0.5/T$ (Fig. 2a).

(b) In the zone of high compressibility, if $T \leqslant 1.0$, $a = 0.5 (1+T)$, if $1.0 \leqslant T \leqslant 2.5$, $a = 1.0$, and if $T \geqslant 2.5$, $a = 2.5/T$ (Fig. 2b).

With basis on quantitative experimental information a different relation between a and T may be adopted if approved by the Building Department.

When using this method design shall be based on the square root of the sum of squares of the generalized forces (shears, overturning moments, bending moments, etc.) corresponding to the various modes in the direction considered. Natural modes whose combined effect does not modify seismic design shears by more than 10 percent may be neglected. In this phase of analysis it is also permissible to neglect the dynamic effect of statically computed torques amounting to eccentricities not greater than 0.2 times the plant dimention measured parallel to the eccentricity. Effects of these as well as of the accidental eccentricity shall be taken in consideration in accordance with Art. 8.

At no elevation shall strength be provided for less than 60 percent of the horizontal design shear obtained from static analysis.

Requirements of the static method concerning walls, ornaments, signs, etc. shall be complied with in design, although it is not necessary to double accelerations computed dynamically.

The subcommittee is conscious of limitations in the method proposed. Yet, experience with the emergency regulations pointed the need for setting rules of dynamic analysis. The wording does not exclude the use of methods more rational than the one specified.

The curve giving a vs. T on firm ground is based on analyses of acceleration spectra, with 10 percent of critical damping, from earthquakes recorded in Western United States. [11] Results of the analyses have been modified due to the prevalence of earthquakes with distant focus. Except for the value of T at which the horizontal line intersects the hyperbola, the curve conincides with those in the San Francisco and Russian codes.

The shape of the curve in Fig. 2b intends to account for long prevailing periods and damping of short-period waves in the compressible mantles.

Combination of natural modes by the root mean square criterion is based on probability analyses. [12]

The limitation of 60 percent of the static-method shears is safer and just as simple as a cutoff in the base shear coefficient for very long natural periods.

The chance given to deal statically with small eccentricities seems artificial. It is necessary since dynamic analysis of buildings with three degrees of freedom per floor is far from simple, and it would not be justified to require explicit consideration of small torques when great accuracy is not expected in the determination of the center of torsion.

10. **Drift Limitations.** The maximum relative displacement between consecutive floors shall not exceed 0.002 times the corresponding difference in elevations, save where elements that do not form an integral part of the structure are attached thereto in a manner such that they are not damaged by its deformations. The value 0.002 may then be increased to 0.003 in the zone of high compressibility and to 0.004 in the rest of the Federal District; in floors and roofs that do not usually support live load, no limitation is imposed.

In computing horizontal displacements every element which forms an integral part of the structure shall be taken in consideration.

Drift limitations may seem overly conservative in the light of experience in other cities. Actually, on soft ground they are intended to minimize panic[9] and in all cases to minimize property damage, for to an extent it should be one of the code's purposes to protect the interests of those who buy a buildings, not only the lives of those who inhabit it.

11. **Special Provisions in Façades.** In interior and exterior facades, window panes shall be placed in their frames leaving a play equal to at least twice the relative horizontal displacement. This measure may be omitted when the window frames are so attached to the structure that its deformations can not affect them.

12. **Awnings.** Buildings taller than 10 m (33 ft) faced with materials that may fracture, or having glass mounted on frames fixed to the structure in a manner incapable of preventing damage from structural deformations, shall possess an awning cantilevering at least 1.5 m all along the facade. This protection is not required if the facade is set 1.5 m or more back of the property line and there is no open pass to pedestrians in this area.

13. **Elements not Forming an Integral Part of the Structure.** Regarded as such and as not contributing to structural strength and rigidity are those parts of a building which are made of brittle materials, such as glass and plaster, and partitions less than 10 cm (4 in.) thick.

Metal and wooden partitions and those made of highly deformable materials, such as glass-reinforced plastics, when not possessing panels of brittle materials, require no special provisions in their junction with the structure from the point of view of lateral displacements. All other nonstructural elements must be fixed to the structure taking such measures that they not be damaged by the latter's deformations. These precautions may consist in fixing the element at one floor, leaving it free at the opposite end, or placing it on one floor and guiding it by means of a channel at the opposite end, etc. Leeways shall be left consistent with the displacements of the structure, and stability verified in accordance with the accelerations of Art. 8.

14. Structural Elements. Elements in a building shall be regarded as structural when, being of metal, timber, concrete, or masonry, their parts are tied to each other and to the rest of the structure.

Masonry walls and partitions forming part of the structure must satisfy the following requirements.

(a) They shall be no thinner than 10 cm (4 in.).

(b) At least in one direction they shall possess or be tied to metal or reinforced concrete members (girders, bond beams, slabs, columns, pilasters, etc.) whose spacing does not exceed 25 times the wall thickness when $f'_m = 50$ kg/cm^2 (711 psi). If $f'_m = 50$ kg/cm^2 the spacing, in centimeters, shall not exceed $f'_m/2$, where f'_m is expressed in kg/cm^2. (This requirement may be omitted in reinforced masonry.)

(c) Blocks, brick, and tile shall have a compressive strength not smaller than 15 kg/cm^2 (213 psi).

(d) Mortar shall have a compressive strength not smaller than 30 kg/cm^2 (427 psi).

This and the following article are supplementary to the chapter on masonry. Therein specific rules are given for bond beams and pilasters, calling for a minimum of four longitudinal bars to give an area not smaller than $0.05 \, f_y/f'_c$ times the cross section area of the member or times the square of the wall panels whose height to width ratio exceeds 1.5, around openings whose maximum dimensions is less than four times the wall thickness, and in reinforced masonry work.

When applying the simplified method of analysis compliance with the requirements in that chapter presumably makes it unnecessary to compute stresses in the wall-reinforcing elements.

15. Bond Beams and Pilasters. Reinforcement of bond beams and pilasters shall resist in tension the components of stresses in the wall or partition assuming cracks at 45 degrees with the horizontal. Stresses due to dead load may be taken into account. Bond of the reinforcement shall checked. Continuity shall be attained in the transmission of forces taken by the reinforcement. Provisions shall be made to resist the resultant of tension where bars are bent. Bars shall be anchored by means of standard hooks at free edges of panels.

16. Clear Spacing at Property Lines and in Construction Joints. Every new building shall be constructed away from neighboring pieces of land at least 3 cm (1.2 in.), but not less than x + 0.006 H in the zone of high compressibility nor less than x + 0.004 H elsewhere. Here x = maximum lateral displacement computed at elevation H. When using the simplified method of analysis take x = 0.002 H.

The exterior surface of every bearing wall in the perimeter of the building shall be vertical.

At construction joints the same criterion applies as for property lines unless special measures are taken to prevent damage from pounding.

Emergency regulations called for "special studies" to prevent pounding. The requisite had practically nil effects in design practice. Hence the subcommittee decided to be more explicit.

The possibility of specifying the clear space as a function of the existence of neighboring structures was dismissed because it might cause legal difficulties. The spacing approved intends to account for computed seismic displacements plus uncomputed rocking and, to a degree, tilting due to differential settlement.

17. Repair of Damage Structures. When a building suffers minor earthquake damage either in its structure of finishing, it shall be repaired restoring at least its original resistnace. If the facades have been damage, the corresponding requirements in Arts. 8, 11, and 12 shall be fulfilled. If the damage is such as to endanger structural stability, the structure shall be reinforced complying with all articles in the present chapter which pertain to structural stability.

Repair of earthquake damage shall in all cases be preceded by a permit from the Building Department. If repairs have been made without it, the Building Department may condemn the repaired members so as to effect repairs according to an approved design.

Actually a permit is required for all repairs whatever the cause. It is appropriate to insist in connection with earthquake damage since it has been Mexico's experience that inadequate repairs run rampant immediately following a macroseism. It was found impracticable to be more explicit in this article.

18. Precautions during Construction. All work shall be so executed that, during the construction stage, the building be capable of resisting the lateral forces that correspond to the base shear coefficient specified in Art. 5. In secondary members it suffices to take measures preventing damage from local failure.

19. General Criteria of Stress Analysis. Simultaneous action of earthquake and wind need not be considered. Structural analysis of buildings subjected to the combined effects of gravity and earthquake shall satisfy at least the requirements specified below, with the exceptions pertinent to the simplified method.

(a) Effects of lateral forces shall be analyzed taking in consideration the horizontal displacements and rotations of all structural members, as well as their continuity and their rigidities.

(b) Flexural deformations of walls and those due to axial load in columns shall be taken in consideration whenever they appreciably affect design stresses and displacements.

(c) In steel structures embedded in reinforced concrete it is permissible to consider the combined section for computing stresses and rigidities if connectors are provided, capable of preventing slip between both materials, and the requirements in the chapter on compound structures are met.

(d) If analysis indicates tensions between foundation and ground, it shall be assumed that part of the foundation is exempt from contact pressures. Equilibrium of the total computed forces and moments must be satisfied in the rest of the contact area and the allowable stresses on the soil shall not be exceeded. Stability of the foundation and superstructure shall be checked under these conditions. If there are elements such as piles or caissons capable of taking tension, they shall be considered in the analysis.

(e) In all cases a revision is required of the compatibility of the displacements of slabs and all other members; slabs must be capable of resisting the induced stresses.

(f) In design of frames containing wall panels it may be assumed that shears in the panels are in equilibrium with axial forces in the frame members, neglecting tensions in the wall.

Points specified herein refer principally to items that are often forgotten in standard design practice. Paragraph (b) is vague; however, too many variables are involved, so no rule of thumb could be set up to decide when flexural deformations in shear walls and the like are important.

Paragraph (c) contradicts the practice and observations of many engineers, who are willing to consider combined action of steel and concrete even when connectors are not provided. Absence of quantitative information on the subject led to this conservative requirement.

One possible objection to (d) concerns the apparently low factor of safety that it provides against overturning. However, the proposed code allows little reduction in overturning moments. Moreover, actual overturning of structures is not anticipated.

Paragraph (f) is admittedly inexact. Yet no better assumption could be found in the literature. The problem of interaction between frames and the wall panels they encase is complicated by the presence of high localized stresses in the frames, in zones partially confined by the walls.

20. **Instruments.** Permit for occupancy of structures covering an area in excess of 10,00 m^2 (one million sq ft) or whose height exceeds 45 m (148 ft) necessitates proof before the Building Department to the

effect that deformeters and oscillographs are installed and that these instruments are capable of accurately recording strong motions.

21. Changes to the Present Chapter. Any requirement in this chapter may modified when analytical or experimental information becomes available which makes such changes possible. (The rest of this article concerns administrative requirements for said modifications.)

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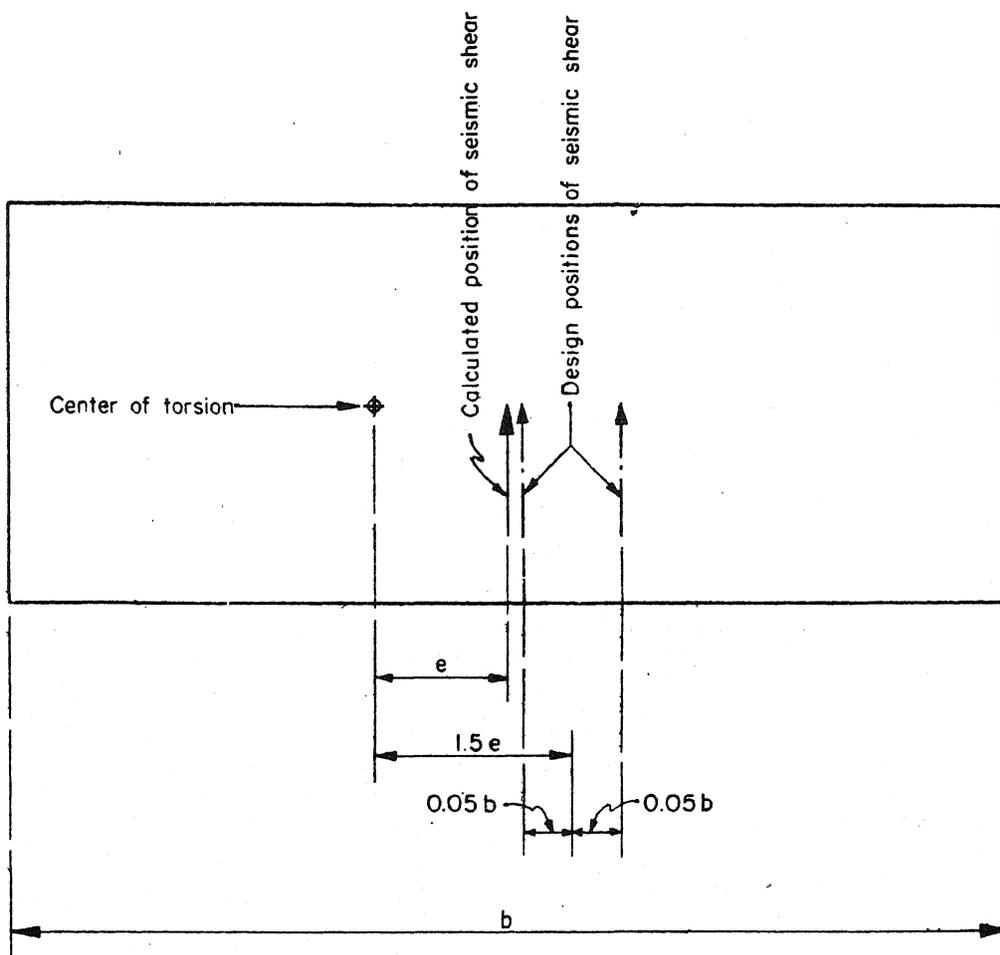
Center of torsion

Calculated position of seismic shear

Design positions of seismic shear
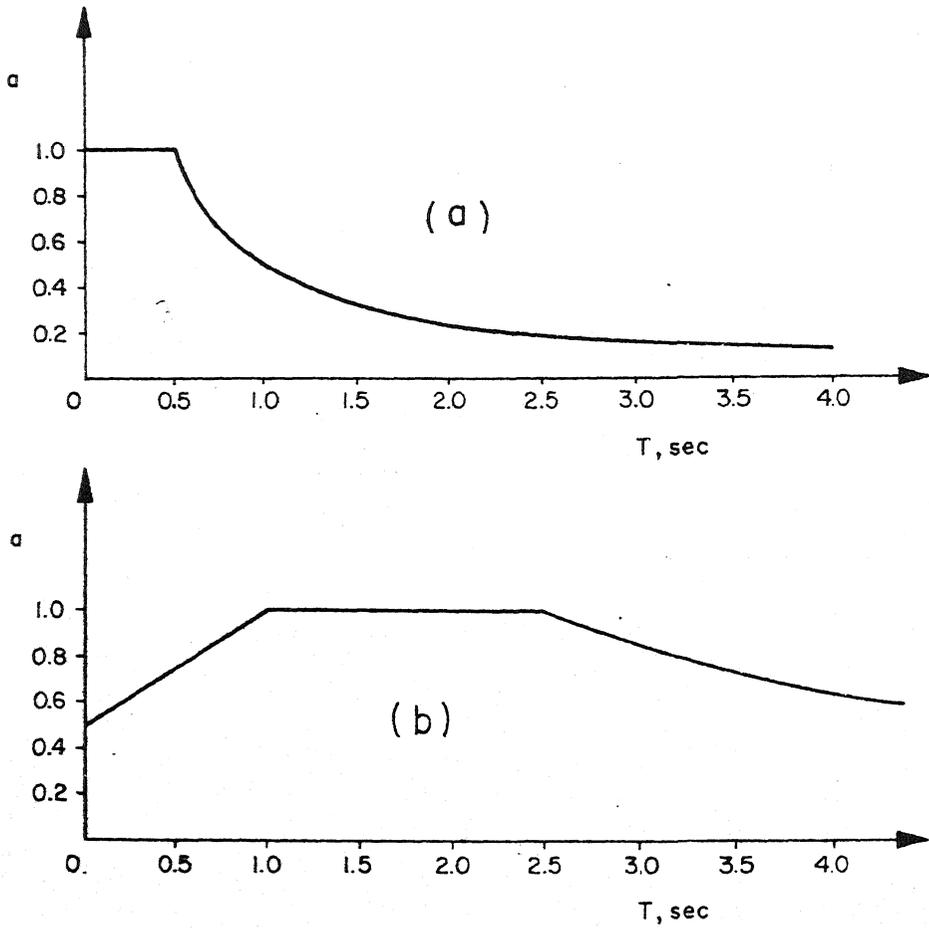
e

1.5 e

0.05 b

0.05 b

b

FIG. 1. TREATMENT OF TORSION IN STATIC METHOD

FIG. 2. DESIGN ACCELERATION SPECTRA