ON THE RUMANIAN GENERAL DESIGN SPECIFICATIONS

FOR CIVIL AND INDUSTRIAL BUILDINGS IN SEISMIC

AREAS. EXAMPLES

by Em. Țițaru[x] and Al. Cișmigiu[xx].

Over an important area of Rumania, including the coun-
try's capital, earthquakes were recorded from time to time,
the intensity of which attained the degrees 8...9 in the
M.C.S. (Mercalli-Cancani-Sieberg) scale. The most severe
earthquakes are rare and have a persistent hypocentrum loca-
ted on the external part of the curvature of the Carpathian
mountains at a depth of about 1oo...2oo km. Concern for pre-
paration of aseismic design rules became actual only after
the great earthquake of 1o.11.194o; this earthquake, whose
magnitude according to the Richter scale was $M = 7,4$, caused
important damage even in Bucharest, at an approximate distan-
ce of 170 km from the focus.

The preparation of aseismic structural design specifica-
tions remains even nowadays a difficult problem because de-
sign methods and especially the values of design parameters
cannot be rigorously established. Our point of view was to
give the practicing design engineer necessary indications in
order to solve with maximum efficiency the cases of usual
practice; such indications are based on an up-to-date know-
ledge of theoretical achievements as well as on design rules
consecrated by existing experience in the construction and
behaviour of aseismic structures. Thus some design data were
established on basis of practice, of engineering judgement
and of economic considerations.

1. DETERMINATION OF THE HORIZONTAL SEISMIC FORCES
FOR THE STRUCTURE AS A WHOLE

1.1. In order to determine the seismic forces the theo-
ry of the "Response Spectrum" was adopted, because this theo-
ry was found to be at the present time the best fundamented
both theoretically and experimentally answering at the same

[x] Em. Țițaru, Chief Design Engineer, IPCMC, Design Office
    for Industrial and Structural Engineering, Bucharest;

[xx] Al. Cișmigiu, Chief Design Engineer, IPB, Design Office
    for the City of Bucharest.
    Adress:
            str. Dionisie Lupu 4, Bucharest Rumania.

time to practical questions, (4),(2),(3),(4).

1.2. The equation adopted for the determination of the basic conventional shear force is:

$$Q_B = K.\gamma.\psi.\varepsilon.P = \alpha.\psi.\varepsilon.P = c.P \tag{1}$$

The factors introduced in this equation include all the elements of the response spectrum theory which influence the values of the seismic forces.

The parameter k establishes the seismic intensity of the site and depends on the characteristics of the focus, on the properties of the soil and on the distance to the focus. To the parameter γ are ascertained values in direct proportion to the acceleration spectrum S_a. We consider that this factor expresses the highly complex soil-structure interaction.

The parameter ψ introduces the effect of internal damping in the structure.

The parameter ε is a factor by which the system with many degrees of freedom is made equivalent with a one mass system.

P is the resultant of all gravity permanent and live loads acting on the structure above the level from which deformation under action of seismic loads exists. The force P includes the weight of foundation if the deformability of the soil is considered in the determination of the dynamic elastic line of the building.

In the practical design equation, the product of the factors K and γ is represented by the dynamic factor α.

1.3. Establishment of the factor k.

At the present time, it has become clear that the Mercalli scale and its various alternative proposals cannot serve as a basis for an engineering scale of forces. The establishment of a scale for Rumania has demanded an appreciation of an international scale of seismic intensities, corresponding to 4 zones:

Zone I: danger of moderate damage if buildings are built without respect of aseismic rules.

Zone II: danger of important damage.

Zone III: danger of very great damage.

Zone IV: danger of extremely grave damage. zones of maximum intensity for which adoption of general aseismic measures may be allowed.

The parameter k is typical for the relative intensity of the zones. It was accepted that k should vary according to a geometric sum. The ratio of this sum was determined on the base of a medium value of the basic shear force factor, accepted in engineering practice for the case of rigid masonry buildings sited on a medium ground; thus:

c = o,oo5 for zone I;
c = o,25 for zone IV.
It followed that r = $\sqrt[3]{5}$ and the k values: k_I = 1,o;
k_{II} = 1,7; k_{III} = 2,9; k_{IV} = 5,o.

In a modern conception, the seismic zoning should be a "spectral zoning" aiming to give the design engineer the response spectrum determined by means of direct records made at the considered site. This being at present impossible, a practical solution is obtained by operations of "macro-zoning" and "micro-zoning", that is by combining general features with local ones. By "macro-zoning", a territory is divided into zones of probable seismic intensity, corresponding to the proposed scale. For appreciation of the intensity in the epi-central zone, use of Table 1 is recommended, which takes into account the depth of the focus and the magnitude of the earthquakes.

It is proposed that the seismic features of the soil should be considered in the seismic zoning as follows:
- in "macro-zoning", features of predominant basic layers should be considered (Table 1);
- in "micro-zoning" should be considered the characteristics of local surface layers.

In practice, the category of ground, to which the surface layer(on which the building is founded) belongs, is established by micro-zoning. This is of a qualitative nature. The quantitative aspect is introduced by the values of the factor , which differ according to the nature of layers. Thus the soil-structure interaction effect is pointed out for various categories of layers in the response spectra.

1.4. Determination of the parameter δ.
This parameter is introduced in order to establish the standard response spectra corresponding to various soil conditions, in the case of a certain reference value "n" of the damping factor. The authors consider that existant experimental results as well as the behaviour of buildings in various soil conditions corroborate the existence of intervals of variation of predominant periods depending on the rigidity of the ground, as well as the possibility of a technical appreciation of the those conditions on the acceleration values.

We specially refer to the studies of the Japanese scientists Takahashi, Kanai, Kawasumi, (2),(3), to the behaviour of buildings in Mexico City during the earthquake of July 1957, to the Seattle spectra (Washington, USA) determined in 13 April 1949, (4), on soft grounds as compared with typical spectra for medium grounds, etc.

Grounds were classified in classes for which the following values of predominant periods were appreciated: very hard soils: o,15 s, hard soils: o,1...o,2 s, intermediate

soils: o,2...o,4 s, soft soils: o,3...o,6 s, very soft soils: o,6...1,2 sec.

The object of "micro-zoning" is to establish the categories of grounds, depending on surface layer stratification; this proceeds according to indications given in Tables 2 and 3.

The authors consider that the establishment of standard response spectra to be introduced in design specifications should follow this principle, the form and values of response spectra should result as medium forms and medium values of a great number of direct records of accelerograms on various categories of grounds, when making a survey programme of spectral zoning of a territory. For intermediate grounds the medium forms established by G.W.Housner (5) for Californian earthquake were adopted. These medium forms may satisfactorily be remplaced in the o,1...o,5 s interval by a straight line and in the o,5...3,interval by an equilateral hyperbole. For the remaining 4 categories of grounds, the response spectra were empirically drafted, always keeping in mind the general form of medium ground spectra, the behaviour of buildings in various ground conditions and the intervals of variation of predominant periods (fig.1). The forms of these spectra were considered to hold good for all four zones of seismic intensity outlined above. Any change in form of the spectrum as the distance from the focal zone varies was not considered, since, in the danger zone in which the earthquake may lead to damage, the predominant periods of the ground may be supposed to be constant, the variable factor being the ground acceleration (included in the parameter k).

In the interval $T = o...o,1$ 0,15 seconds, it is proposed that the acceleration values should equal the double maximum value. This seems to be true, since for $T = O$ the system must have the accelerations of the ground and very rigid buildings also posess a very small internal damping.

As in the process of seismic load determination it is possible to introduce the influence of internal damping, the equivalence of structures with many degrees of freedom with a one-mass system and the influence of higher modes of vibration, the spectrum variation was drafted up to 3 seconds, that is for the interval for which records exist.

The authors are thus persuaded that by a complex use of "macro-zoning", "micro-zoning" and of spectral forms differing by ground nature, light may be thrown on the following important practical issues:

a. distinguishing of seismicity of sites with different predominant basic layers,

b. distinguishing in a small zone with the same basic layer of the seismicity of sites with different surface layers.

We think it is unjustified to follow the practice of some

specifications which introduce the effect of soil on the spectra by changing only the intensity, without changing the form.

1.5. Establishment of the parameter ψ .

The influence of internal damping was not introduced by a system of spectral S_a curves, differing with the factor "n", but an independent parameter ψ was proposed. This parameter is defined as a ratio between ordinates of two spectral curves $(S_v$ and $S_a)$ for two different values of the factor "n", corresponding to the same period T.

Computing the " ψ " values for various "n" for the medium response spectra of intermediate soils, it results that these values are fairly constant for the entire scale of periods T. Thus it is possible to work in practice with a single stable "basic" curve computed for a "basic" factor "n", instead of considering a system of spectral curves corresponding to various "n". To pass from one spectral curve to another, corresponding to another "n" value, the basic curve is multiplied by factor "ψ". These results were considered valid for the spectral curves other ground categories too. The basic value of "n" was chosen to be n = 20% n_{cr}, because this value corresponds to brick masonry buildings. With the medium values thus computed, the following analytical expression was established for the factor   (fig.2):

$$\psi = \frac{2.92}{\sqrt{n}} \qquad (2)$$

It was considered that the factor "n" is not dependant on the vibration modes. The factor "n" is especially conditioned by the following important influences: the nature of building materials, the predominant strain, the intensity of the state of stress on the building as a whole, as shown in the "stress-strain" relationship, as regards great elasto-plastic or plastic strains. In table 4 the proposed values for ψ are given, depending on the following medium values admitted for "n": steel 2...6%, concrete and reinforced concrete 7...14%, brick masonry 15...25%, structural timber 10...20%.

In spite of the fact that present data are of a qualitative more than of a quantitative nature, such a proposal means to guide the design engineer in making a full and active use of the damping properties of various materials and structures and in the rational design of earthquake-resistant structures.

1.6. Determination of the equivalence factor ε ; the distribution law of seismic forces; the influence of higher modes.

The equivalence factor ε of a system with many degrees of freedom with a one-mass system may be determined as

$$\varepsilon = \frac{\left[\sum_{i}^{n} P_i \cdot u_i\right]^2}{\left[\sum_{1} P_i\right]\left[\sum_{1}^{2} P_i \cdot u_i^2\right]} = \frac{\left[\sum_{1}^{n} P_i \cdot \xi_i\right]^2}{\left[\sum_{1} P_i\right]\left[\sum_{1}^{2} P_i \xi_i^2\right]} \quad (3) \quad who \; \xi_i = \frac{u_i}{u_n}$$

It may be shown that $\sum_{1}^{n} \varepsilon \to 1$

The conditions of equivalence in which this equation was established were: the identity of periods and the equality of energies. Considering the expressions of the inertial forces $F_i = \frac{4\pi^2}{T^2} \cdot m_i \cdot u_i$ a simple development leads to a distribution law of the basic shear Q_B along the height of the buildings:

$$S_i = \frac{P_i \cdot u_i}{\sum_{1}^{2} P_i u_i} \cdot Q_B = \frac{P_i \cdot \xi_i}{\sum_{1}^{2} P_i \xi_i} \cdot Q_B = \chi_i \cdot Q_B \quad (4)$$

Eq. (5) may be used in a direct determination of the seismic level shears S_i (without a preliminary computation of Q_B):

$$S_i = \left[\alpha \cdot y \cdot \frac{\sum_{1}^{n} P_i \cdot u_i}{\sum_{1}^{2} P_i \cdot u_i^2}\right) u_i\right] \cdot P_i = \left[\left(\alpha \cdot y \cdot \frac{\sum_{1}^{n} P_i \cdot \xi_i}{\sum_{1}^{2} P_i \cdot \xi_i^2}\right) \xi_i\right] P_i = C_i \cdot P_i \quad (5)$$

The real displacement of the level (i) may be computed from eq. (6):

$$u_{i,real} = \frac{g}{4\pi^2} \cdot T^2 c_i = 24.87 \; T^2 c_i \quad (6)$$

The equivalence factor ε (eq. 3) and the distribution law make possible a quick determination of seismic forces for higher vibration modes of structures with many degrees of freedom, using the response spectra for the one-mass system (fig. 1), if the form of the dynamic elastic lines corresponding to the considered modes are previously computed.

In order to determine seismic forces for important prominences placed on buildings, the maximum values of forces which appear in higher vibration modes may be chosen and especially those which appear in the first vibration modes of the prominences.

2. DETERMINATION OF VERTICAL SEISMIC FORCES

The authors consider that the response spectrum theory may hold good for the vertical seismic action too. As the vertical rigidity of buildings is usually extremely great, the assumption was made that the vertical vibration period of the building is greatly influenced by the deformability of the ground. This leads practically to short periods, situated on the constant part of the spectrum, that is to maximum values for accelerations. In the special case of big span cantilevers and roofs, if the vertical flexibility of the structure becomes important, the periods increase and the influence of higher modes may be considered. In analysis, equations (1), (3), (4) hold good, but for the factor y, for

which values amplified from 3 to 4 times were proposed.

3. DETERMINATION OF SEISMIC PRESSURES EXERTED BY LIQUIDS AND FRIABLE MASSES IN SILOS

The seismic pressures p_0 exerted by liquids stored in reservoirs are considered to be constant along the height of the vessel and are determined by eq.(7)(fig. 3):

$$p_0 = \omega \cdot \alpha_{max} \cdot \delta \cdot H \qquad (7)$$

The seismic pressures of friable masses in silos result from the increase in active thrust due to the diminution of friction angles, to which the resultant of inertial forces corresponding to the stored mass is added.

x

x   x

With the values of ε and T computed for the 1-st mode, the ratio $\rho_1 = \frac{\varepsilon_1}{T_1}$ may be evaluated by means of which the necessity of analysing higher modes may be appreciated. The ratio ρ may serve as a criterion of dynamic rigidity, more satisfactory than the classifications made on basis of the period of the first mode only. A curve $\varepsilon = \varepsilon(T)$ (fig.4) may be traced for the ε, T values of various modes; this curve may help to forecast approximately the ε, T values for higher modes, using the ε, T, values computed for the lower modes; this may lead to important advantages in practical analysis.

4. ASEISMIC STRUCTURAL ANALYSIS

4.1. Principles

The chapter of aseismic structural analysis was developed on the following principles:

1. In problems of **strength of materials** (general sectional stresses, sectional stresses on structural members, unit stresses), **rigidity** (relative level rigidity, general rigidity, rigidity of structural members), **stability** (mechanical and elastic, - local and general), **vibrations** (determination of periods and of vibration modes), consideration must always be given to the spatial co-operation of structural members.

2. The behaviour of the building must be considered not only in the stage in which the prescribed forces may act, but for all phases, ranging from 0 seismic load up to the collapse load, and following step by step the process of development of elastic and plastic strain, during the dynamic adaptation phenomena.

3. An increase in strength and stability of the spatial structure must be achieved by alloting some strength reserves in members and sections of decisive rôle.

4.2. Basic principle of the dynamic-spatial analysis

The authors developed a general method for the dynamic

and spatial structural analysis of buildings stressed by
earthquake loads. This method comprises all the steps needed
in an exhaustive analysis, such as:
  1. the determination of dynamic elastic lines;
  2. the determination of periods;
  3. the determination of the basic shear;
  4. the distribution of the seismic forces along the
height of the building;
  5. the determination of seismic forces acting on the
principal resistant members of the spatial structure (frames,
diaphragms, special bars, etc.);
  6. the determination of sectional stresses in various
sections of the structural members.

The method may be applied for any vibration mode. The
fundamental idea of the method lies in the fact that it co-
ordinates in a comprehensive method the spatial behaviour,
the active use of principles outlined in 4.1, of the respon-
se spectrum theory (eq.1, 4, 5) and of free vibration analy-
sis (periods and forms).

The consideration of spatial co-operation for the deter-
mination of displacements in the structure as well as for the
determination of the loads acting on every resistant member
(frames, vertical and horizontal diaphragms, special bars)
was achieved by making use of the generalised method of the
centre of rigidity, the basic notion of which is the relative
level rigidity. This may be defined as the shear force at le-
vel (i), corresponding to a unit relative displacement: (6)
$R_i = \frac{Q_i}{\Delta_i}$ (fig.5). The quantity R_i is not a constant for the
considered member at the considered level (i), since it de-
pends on the general elastic line of the member (variation
of rigidity and the distribution of forces along height); R_i
may even change of sign ($\pm$) along the height of the building.
It must be stressed that the expression for R_i has
significance only if the state of strain is caused by bending
with shear.

In the special case when on a certain length pure ben-
ding ($Q_i = 0$) exists, an indetermination of the form $\frac{0}{0}$ appears,
which may be solved by correcting the real shear force dia-
gram by a supplementary force acting at the top of the member.
This force, of small intensity compared with the other forces
(thus slightly influencing the displacements) does not allow
the shear to vanish along the height of the considered member.

From the practical point of view, the sensibility of R_i
(that is the variation of R_i with the variation of the load
diagram of the considered member) may be considered as follows:
  1. In the case of short cantilever diaphragms, when ben-
ding strain may be negligible, R_i is invariable and becomes
a geometric constant;
  2. in the case of frames in which displacements due to
shears are predominant, the sensibility of R_i is small;
  3. in the case of "bars" in which bending strains are

dominant or cannot be neglected, R_i has an extreme sensibili-
ty to small variations in the loading diagram.
If the principal inertial axes of some vertical members are
not parallel to two orthogonal axes, it will be necessary to
determine for each member the oblique relative level rigidi-
ties and the principal axes of rigidity at level (i), (7), al-
ways keeping in mind the above-mentioned definition of R_i. By
the generalised expression of the relative level rigidity and
its properties in the form above (dependency on, and sensibi-
lity in, the variation of rigidities and masses along the
height of the member, the property of being alloted a sign,
possibility of solving indeterminations on portions stressed
in pure bending, the consideration of oblique rigidities) a
basic criterion is obtained for the establishment of the com-
patibility of strains along the height of the member and in
the plane of the horizontal diaphragms. This criterion per-
mits the solving of the problems of establishment of the elas-
tic lines and the distribution of forces for any loading sys-
tem acting on a spatial structure; that is the solving of the
problem of the higher vibration modes. The method may easily
be suited to step-by-step procedures.
    Starting from the method proposed by L. Grinter for the
analysis of frames acted on by lateral forces (8), the au-
thors have outlined in Table 5 the steps necessary to obtain
the relative level rigidities, the elastic lines, and the
solution for lateral forces, in a practical form suitable to
step-by-step procedures. This table may also be used in the
determination of periods and vibration forms of independent
frames.
    In the analysis of diaphragms with apertures, the au-
thors use the method of forces, with a grouping of redundants
(fig. 6) and certain simplifications due to practical consi-
derations.
    In the method outlined above, the spatial aseismic struc-
ture is treated as a "complex bar" supported by a deformable
medium; this was achieved by a generalisation of the notion
of "simple bar".
    Regarding the dynamic analysis (the determination of
forms and periods of vibrations), the energy method with a
step-by-step procedure is adopted, for all vibration modes.
    The dynamic elastic line corresponding to a certain vi-
bration mode is the elastic line of "the complex bar" under
action of inertial forces in direct proportion to the seismic
forces computed by eq.4. Thus the sum of the conventional
inertial forces (with which the vibration form was determined)
results in a basic shear Q'_B , "m" times bigger than Q_B (eq.1).
This result permits the coupling of the dynamic analysis for
the determination of vibration forms and periods with the de-
termination of seismic forces on the base of the response spec-
trum theory (chapter 1).
    The step-by-step procedure for the 1-st mode.

I-st Approximation. Step I_1. Determine in a preliminary approximation the relative level rigidities $R_{i,1,0}$ for each vertical resistant member, for a horizontal loading diagram in proportion with the gravity level forces P_i.

Step I_2. The loading of the spatial structure with the horizontal inertial forces $F_{i,1,I} = P_i$ and the determination of sectional level shears $Q_{i,3,I}$.

Step I_3. The distribution of $Q_{i,1,I}$ to the resistant members, on the base of the relative level rigidities $R_{i,1,0}$.

Step I_4. Determination of a new approximation of the relative level rigidities, $R_{i,1,I}$ on the base of forces obtained in step I_3.

Step I_5. Determination of relative level displacements $\Delta_{i,1,I}$, of the absolute displacements $U_{c,1,I}$ and of those relative of the elastic axis the spatial structure with the eq.:

$$\Delta_{i,1,I} = \frac{Q_{i,1,I}}{\Sigma' R_{i,1,I}} \;;\quad U_{i,1,I} = U_{i-1,1,I} + \Delta_{i,1,I} \;;\quad \xi_{i,1,I} = \frac{U_{i,1,I}}{U_{n,1,I}}$$

Step I_6. The determination of the periods in the approximation I:

$$T_{1,I} = 0{,}2 \sqrt{\frac{\sum_1^n P_i \cdot \xi_{i,1,I}^2}{\sum_1^n P_i \cdot \xi_{i,1,I}}} \cdot \sqrt{U_{n,1,I}}$$

in which $U_{n,1,I}$ is the top displacement in cm.

II-nd Approximation

Step II_1. The spatial structure is loaded with the forces $F_{1,II} = P_i \cdot \xi_{i,1,I}$, and the $Q_{i,1,II}$ is drafted.

Step II_2. The distribution of $Q_{i,1,II}$ to the vertical resistant members, depending on $R_{i,1,I}$.

Step II_3. The determination of a new approximation of the relative level rigidities $R_{i,1,II}$.

Step II_4. The determination of $\Delta_{i,1,II}$; $U_{c,1,II}$ and $\xi_{i,1,II}$ on basis of eq. determined in step I_5.

Step II_5. The determination of the period in the approximation II.

$$T_{1,II} = 0{,}2 \sqrt{\frac{\Sigma P_i \cdot \xi_{i,1,II}^2}{\Sigma P_i \cdot \xi_{i,1,II}}} \cdot \sqrt{U_{n,1,II}}$$

Approximation nr. N

Step N_1. The steps are to be continued till the relative elastic line is the approximation N-1 and N fairly coincide:

$$\xi_{i,1,N} \sim \xi_{i,1,N-1}$$

Step N_2. The period and the equivalent factor for the 1-st mode are:

$$T_{1,N} = 0{,}2 \sqrt{U_{n,1,N}} \;;\quad \varepsilon_{1,N} = \frac{\left[\sum_1^n P_i \cdot \xi_{i,1,N}\right]^2}{\left[\sum_1^n P_i \cdot \xi_{i,1,N}^2\right][\Sigma P_i]}$$

Step N_3. The shear at the base is computed for the conventional forces:

$$F_{i,1,N} = P_i \cdot \xi_{i,1,N-1}$$

$$Q'_{B,1,N} = \sum_1^n P_i \cdot \xi_{i,1,N-1}$$

Step N_4. The basic seismic shear force for the 1-st mode $Q_{B,1,N}$ is computed with eq.1, for $T_{1,N}$.

Step N_5. The factor $m = Q_{B,1,N}/Q'_{B,1,N}$ is to be determined.

Step N_6. Determination of seismic level forces with the eq.:

$$Si = m \cdot F_{i,1,N} = m \cdot P_i \cdot \{c_{1,N-1}$$

It is thus sufficient to amplify by factor "m" the shear force diagrams obtained by distribution of sectional shears in approximation N, for each vertical resistant member.

Higher modes of vibration

The succession of steps is, in principles, the same as for the 1-st mode. As a first approximation for the form of mode j , the statical elastic line under gravity loads P_i may be taken, considering j -1 intermediate supports at the nodes of the considered mode, the position of which are chosen by experience.

It is known that the tentative analysis of higher forms of vibration by applying simply step-by-step procedures is bound to be unsuccessful, because it always leads to the form of the first mode. But, if the orthogonalizing condition of forms and the "purifiying" conditions are repeated at every step, the forms of lower modes are eliminated and the chosen form is obtained. Lack of space does not permit a more developed discussion.

As shown above, the frequent case of a structure with two rectangular axes of symmetry (for which a longitudinal and a transverse analysis is made) may be solved correctly. The method may be extended for the case of assymetric structures if the displacements (rotations and linear displacements in a plane perpendicular to the direction of earthquake) of the axis of the structure are small compared with the displacements of the real spatial axis projected in the plane of seismic action.

4.3. On dynamic adaptation.

The authors recommended in the Draft Specifications the consideration of the property of adaptation of structures to seismic action. We think that the notion of "adaptation", set forth by A.Caquot (9) for the case of a system of forces of constant form and value, may be extended for the case of dynamic seismic action. This is justified by the limited duration of alternative seismic stresses and by the experience of damage processes in buildings, showing a stress redistribution. The conception of analysis based on dynamic adaptation is best suited for aseismic design as it permits a correct evaluation of the safety of structures, an improvement in general behaviour and an economically advantageous design. Making use of the property of adaptation, the effects of diminution of seismic forces and of stress redis-

tribution between members of the spatial structure may be considered, according to behaviour schemes of the structures in limit stages of stress and strain. To apply actively seismic adaptability, particular judgement is needed in directing the process of plastification (positions and succession of plastic hinges and zones). The section in which plastic strain occurs for one or more sectional stresses (bending moment, shear force, torsional moment, tensile or compressive normal force) should be designed to resist safely to the rest of sectional stresses. A way for future investigation in aseismic structural analysis, outlining the capacity for seismic adaptation, is set forth by G.W.Housner's proposal (9). The condition of stability of a limit stage is: the sum of elastic and plastic strain energies for which a structure is capable in the considered limit stage should exceed the total seismic kinetic energy E_c allotted to the structure. The parameters set forth in chapter 1 permit evaluation of E_c in eq.(8):

$$E_c = \frac{1}{2} \cdot M \, Sv^2 = \frac{P}{2g} \cdot \left(\frac{\alpha \cdot \psi \cdot T}{2\pi} \right)^2 \qquad (8)$$

For more than 10 years, limit design specifications are in use in our country for the design of sections in the ultimate stage and for the analysis of some structural members in which stress redistribution due to plastic strain is allowed for, in the case of statical forces. At the present time, studies and investigations are made for the generalisation of limit analysis and design to all building materials. Such Specifications allow for the practical use of seismic adaptability and of the energy analysis outlined above. In our Draft Specifications, use of the property of dynamic adaptability on basis of assumptions of probable damage is especially recommended in the case of buildings with masonry panels and those with load-bearing walls.

4.4. Creation of strength reserves.
Creation of strength reserves is recommended in the following cases:
a. when uncertainties subsist in the appreciation of spatial co-operation of some members of the structure or regarding stress flow;
b. when reserves exist in section due to constructive arrangements, which may become seismically effective with no supplementary uneconomic expense of materials and labour;
c. when uncertainties subsist in seismic load determination.

4.5. The aseismic analysis of multi-storey buildings.

The advantages of the method proposed above are especially pointed out in the aseismic analysis of high multi-storey structures performed by the response spectrum theory. Regarding the damping, the function $\psi = \psi(n)$ allows for a fairly correct appreciation on basis of experimental data, of the damping factor and its influence on the spectrum. It is thus possible to distinguish between various high buildings, regarding the building materials used in the structure and cladding as well as the structural solution itself (frames, diaphragms, rigid or flexible structures, etc.).

The function $\mathcal{E} = \mathcal{E}(P_i, \mu_i)$ allows for the exact determination of the equivalence factor for any vibration mode, any mass distribution and any variation of rigidity. The law of distribution of seismic forces (eq.4) is quite general and takes exactly into account the form of the elastic line.

The method of dynamic and spatial analysis takes also into account the spatial co-operation of all members resistant to lateral action (including the consideration of the rigidity of resistant masonry panels). The higher vibration mode can be completely and exactly determined by the energy method using step-by-step procedures, with a "purification" of the influence of lower modes at each stage. Thus, the use of response spectre up to 3 seconds appear to be justified.

5. ASEISMIC CONFORMATION

The chapter regarding "Aseismic conformation" was developed in such a manner as to give the design engineer the necessary elements for a rational and active organisation of strength rigidity and stability in the designed structure.

Such elements put forward the complex interaction between choice of structure and of building materials and the optimum stress flow under lateral earthquake action.

The designer's creative thought, rather not acquainted with the seismic action, is systematically directed to the pursuit and understanding of the behaviour of the structure as an integrated whole, from "O" seismic load up to the collapse load, in order to set forth any adaptation possibilities by material and by structure, to achieve thus an optimum solution.

This chapter develops the following main problems:

1 - General rules of aseismic building conformation
2 - Arrangement of masses and resistant members.
3 - Arrangement of aseismic joints
4 - Conformation of foundations
5 - Conformation of vertical resistant members (frames and diaphragms)
6 - Conformation of transverse ties between vertical resistant members.
7 - Aseismic conformation of non-structural members (principal and secondary).

8 - Aseismic conformation of important prominences
9 - Aseismic conformation of buildings made of various mate-
      rials (timber, steel, reinforced concrete cast "in situ"
      and precast masonry).
10 - Aseismic conformation of various types of structural (in-
      dustrial halls, multi-storey buildings).

A somewhat larger development was given to buidings made
of precast parts, because the Economic Development Plan of the
Rumanian Republic lay particular stress on the industrialisa-
tion of the constructional activity by prefabrication (12).
Regarding the answer to the question "Which structures
are to preffered in seismic zones, rigid or flexible ones ?"
the authors'point of view, based on their engineering practica
is outlined below:
      a. flexible structures should be preffered in the case
of light structures, in which danger of damage in wall panell-
ling is excluded, due to a correct arrangement (wall panelling
should not be dragged along in the deformation of the struc-
ture);
      b. rigid structures are of advantage in the case of buil-
dings made of traditional materials and in the case of monumen-
tal buildings.

6. EXAMPLES
The application of the Draft Specifications as well as
studies and design of aseismic buildings carried out by the
authors have contributed towards the development in the Ruma-
nian Peoples'Republic of a modern conception in aseismic de-
sign.
In fig. 7,8,9,1o,11,12,13 some typical structures built
in Bucharest are shown, which were directly designed by the
authors or for which the authors served as Consulting Engi-
neers for aseismic design. The designs were carried out in
the principal Design Offices in Bucharest:
      I.P.C.M.C. - Design Office for Structural Engineering;
      I.P.B.   - Design Office for the City of Bucharest;
      I.S.C.A.S. - Design Office for Studies in Construction
                    Town Planning and Architecture.

FIGURE CAPTIONS
Fig.1. Response spectra for various ground categories and
      various seismic zones.
Fig.2. Diagram of the damping function ψ
Fig.3. The ω diagram.
Fig.4. Relation curve between the equivalence factor ε and
      the natural vibration periods T for various modes.
Fig.5. The relative level rigidity.
Fig.6. The analysis scheme of diaphragms with apertures.
Fig.7. Typical 8 storey block, with rare aseismic reinfor-
      ced concrete walls cast "in situ", of "cellular" type
(Co-workers: D.Badea and T.Popp, Struct.Engrs.)

Fig. 8. Ibidem (Co-workers: D.Badea and Gr. Eliescu, Struct. Engrs.).

Fig. 9. 8 storey block with flexible ground-floor, in reinforced cast "in situ" concrete (co-workers: T.Popp and D.Moscuna, Struct. Engrs.).

Fig. 10. Typical 8 storey block in precast reinforced concrete, "big panel" system (Co-workers S.Cristescu, N.Mihalache, A.Sorbală, Struct.Engrs.).

Fig. 11. 14 storey block, of "cellular" type with aseismic reinforced concrete cast "in situ" walls (Co-workers, D.Badea, Gr. Eliescu Struct.Engrs.).

Fig. 12. Tower buildings 18 storeys high in reinforced cast "in situ" concrete. The complex structure is an example of a complete application of the analysis method described in chapter 4.2. (Chief Design,Enginer P.Vernescu, Struct.Eng. co-worker Z.Măduţă, Struct. Eng.).

Fig. 13. Tower building 18 storeys high. Project, Structure composed of a central reinforced concrete "tower" cast "in situ" and lift slabs. (Co-worker, I.Berleanu, Struct.Eng.).

REFERENCES

(1)- METODI RASCIOTA ZDANII I SOORUJENII NA SEISMOSTOIKOS-TI Moskva 1958
  - Rasciot ghibkih soorujenii na seismiceschie vozdeistvia. Korcinski I.L.
  - Opredelenie seismiceskih vozdeistvii na soorujenia s uciotom vîssih form sobstwenîh kolebanii. Urazbaev M.T., Leiderman Iu.R., Rasskazovskii V.T.
  - Uproscenii sposob rasciota soorujenii na seismostoikosti Nazarov A.G., Karapetian B.K.
  - Spectrî deistvia seismiceskih colebanii na soorujenia. Medvedev S.V.

(2)- The "SMAC" strong mation accelerograph and other latest instruments for measuring earthquakes and building vibrations. by Ryntaro Takahasi Proceedings of the World Conference on Earthquake Engineering - California 1956

(3) - Seismic characteristics of ground by Kiyoshi Kanai, Ryntaro Takahasi, and Hirosi Kawasumi. Proceeding of the World Conference on Earthquake Engineering - California 1956

(4) - Spectrum Analysis of strong-Mation Earthquakes. by Housner, 6.W., Martel, R.R. and Alford, J.L. Bull. of the seismological Society of America, Vol. 43, NO2. April 1953

(5) - {DISCUSSION by E.Housner, G.W. Proceedings ASCE vol.84 Nov. 1958

(6)- Seismic analysis of reinforced concrete buildings by Kiyoshi Muto Proceedings of the World Conference on

Earthquake Engineering-California 1956

(7) - A.C.I. 1951. Lyn.

(8)-Theory of modern steel structures vol.II by L.E.
Grinter
Mac Millan Co., New York, 1949.

(9)-Leçons sur la résistance des matériaux, Paris,1931-
1932. A. Caquot

(10)-Limit Design of Structures to resist Earthquakes. by
G.W.Housner. Proceedings of the World Conference on
Earthquake Engineering California 1956

(11)-The spatial dynamic aseismic analysis of structures,
by Em.Titaru and Al.Cișmigiu, Industria Constructii-
lor și Materialelor de Construcții (ASIT) Bucharest
Dec. 1959(in rumanian Laguage).

(12)-Über die rumänischen Bestimmungen für erdbebensichere
Bauten aus Stahlbetonfertigteilen by Em.Titaru and
Al.Cișmigiu. Die Montagebauweise mit Stahlbetonfer-
tigteilen im Industrie und Wohnungsbau. II Interna-
tionaler Kongress. Dresda 1957.

FIG. 1a — VERY HARD GROUND

FIG. 1b — HARD GROUND

Fig. 1c. — FIRM GROUND

Fig. 1d. — SOFT GROUND

RUMANIAN DRAFT DESIGN.
DETERMINATION OF THE SEISMIC
FORCES
STRUCTURES WITH SEVERAL FREEDOM DEGREES

FOR ANY VIBRATION MODE
THE BASE SHEAR

$$① \quad Q_B = (K.\chi).\psi.\varepsilon.P$$
$$= (\alpha.\psi.\varepsilon).P$$
$$= C.P.$$

$$P = \sum_1^n P_i' = P_1 + \dots + P_n$$

$$②\ S_i'\ \begin{vmatrix} \text{METHOD I} & S_i' = \dfrac{P_i' U_i'}{\sum P_i' U_i'} & Q_B = \dfrac{P_i' \xi_i'}{\sum P_i' \xi_i'} Q_B = \chi_i.Q_B \\ \text{METHOD II} & S_i' = \left[\alpha.\psi \dfrac{\sum_1^n P_i' \xi_i'}{\sum_1^n P_i' \xi_i'}\right]\xi_i' \Big] & P_i' = C_i.P_i' \end{vmatrix}$$

$$U_i' \text{ real} = 24,87\ T^2\ C_i'$$

③ VALUES OF K

ZONE		K
0	MINOR DAMAGE	0
I	MODERATE	1,00
II	IMPORTANTE	1,70
III	VERY GREAT	2,90
IV	EXTREMLY GRAVE	5,00

⑤ THE DAMPING FUNCTION
$$\psi = \frac{292}{\sqrt{n}} \ \text{(damping factor)}$$

④ THE RESPONSE SPECTRUM FOR VARIOUS GROUNDS TYPES AND FOR $n = 0.20\ n_{cr.}$

⑥ THE EQUIVALENCE FACTOR WITH A ONE MASS SYSTEM
$$\varepsilon = \frac{\left[\sum_1^n P_i\,\xi_i\right]^2}{\left[\sum_1^n P_i'\right]\left[\sum P_i\,\xi_i^2\right]}$$

Fig. 1f.

FOR $T < 0,1$ SEC $\alpha = (2\dots3)\alpha$ max.
$n_{BASE} = 20\% \ n_{cr}$.

α max = 16,7 ZONE IV — $\dfrac{25,0}{T}$

α max = 10,0 ZONE III — $\dfrac{15,0}{T}$

α max = 6,0 ZONE II — $\dfrac{9,0}{T}$

α max = 3,0 ZONE I — $\dfrac{3,0}{T}$

Fig. 1e   VERY SOFT GROUND

CALCULATED CURVE
$\psi = \dfrac{2,92}{\sqrt{n}}$

Fig. 2 $\psi = \psi(n)$

$P_0 = \omega.\alpha_{max}.T.H$

Fig. 3.

$$\sum_1^n \varepsilon \to 1,000$$
$$\varepsilon_1 + \varepsilon_2$$
$$\sum_1^n \varepsilon \to 1,000$$
$$\varepsilon_1 + \varepsilon_2 + \varepsilon_3 + \varepsilon_4 = 0,938$$

MASONRY CHIMNEY

Fig. 4 $\varepsilon = \varepsilon(T)$

Q_R Q_i Q_2

SHEAR   DEFLECTION

$R_i = +\dfrac{Q_i}{\Delta_i}$

$R_2 = -\dfrac{Q_2}{\Delta_2}$

FIG. 5

FIG. 6

2194

FIG. 7

RESULTS

x - x	y - y
$Tx = 0.875$ $\alpha = 0.0444$	$Ty = 0.71$ sec. $\alpha = 0.0423$
$\varepsilon = 0.657$ $\Psi = 1.3$	$\varepsilon = 0.772$ $\Psi = 1.3$
$Q_B^x = 0.039\,P = 134^t$	$Q_B^y = 0.0425\,P = 149^t$
$Q_B^m = 1437^t$	$Q_B^m = 1863^t$
$m_x = \dfrac{Q_B^x}{Q_B^m} = 0.093$	$m_y = \dfrac{Q_B^y}{Q_B^m} = 0.08$

D_3 D_3

D_1 D_1

D_2 D_4 D_2

Fig. 8 a

APPROX. I

	D1	D2	D3	D4	ξ
	124,5	118,5	70,5	97	1,000
	77	53,5	71	48	0,883
	70	59,5	41,5	48	0,763
	68,5	60	43	49	0,646
	68,5	59,5	44	53	0,529
	61	60,5	48	50	0,411
	53,5	40	54	67	0,297
	52	47,5	63,5	74	0,190
	24		86,5		0,090

APPROX. II

	D1	D2	D3	D4	
	141,8	105	53,2	75,1	1,000
	28	42,8	10,7	30,7	0,887
	64,7	35	25,9	26,1	0,765
	52,6	30,2	24,6	22,8	0,641
	37,1	24,6	22,3	21,4	0,521
	25,1	16,7	19,7	16	0,404
	12,7	6,7	20,6	27,1	0,290
	12,3	7,4	29,1	30,8	0,183
	32,7		27,4		0,086

APPROX. III

	D1	D2	D3	D4	
	153,5	97,4	41,5	64	1,000
	70,6	39,4	13	26,4	0,886
	73,1	33,3	23,3	22,8	0,763
	59,1	29,2	22,2	20,2	0,638
	43,1	24,5	20,2	20,3	0,517
	24,9	16,7	19,2	16,6	0,401
	10,7	11,7	20,6	28,9	0,286
	21,8	11,3	31,4	39,8	0,162
	48,2		33,7		0,064

$m_y = 0.08$

SEISMIC FORCES: S_i

	S_i^{D1}	S_i^{D2}	S_i^{D3}	S_i^{D4}	
	124	7,8	3,3	5,1	(MODE 1)
	2,3	3,1	1,1	2,1	
	5,7	2,7	1,8	1,9	TRANSVERSAL
	4,0	2,3	1,8	1,6	
	3,5	1,9	1,6	1,6	CALCULLATION
	1,9	1,3	1,5	1,3	
	0,9	1	1,7	2,3	ACCORDING
	1,7	0,9	2,5	3,3	
	3,9		2,7		To 4.2

Fig. 8 B

$Tx = 1.20$ sec.

$Ty = 1.08$ sec.

FIG. 9a

Fig. 9 b I.P.B.

Arh. E COSMATU

Fig. 10

$$T_x = 0.5 \text{ sec.}$$
$$\alpha = \frac{0.03}{T} = 0.06$$
$$\Psi = 1.25$$
$$\varepsilon = 0.77$$
$$Q_B^x = 0.058 \, D$$

$$T_y = 1.20 \text{ sec.}$$
$$\alpha = \frac{0.03}{T} = 0.025$$
$$\Psi = 1.35$$
$$\varepsilon = 0.75$$
$$Q_B^y = 0.025 \, D$$

FiG. 11a

Fig. 11b I.P.B.

Arh. V. NITULESCU

20.55 m

1-1

2-2

frame

Fig. 12a

THE DYNAMIC SPATIAL ANALYSIS

17	3.23	4.34	35.9
16	6.46	8.50	22.2
15	8.15	10.3	15.2 Frame
14	8.95	11.3	11.8
13	9.53		12.00
12	8.15	11.3	12.50
11	6.90	10.20	12.70
10	6.10	10.25	13.00 diaphragm C
9	5.40	10.50	12.90
8	4.72	10.74	13.00
7	4.15	10.83	13.30
6	3.55	11.03	13.30
5	2.85	10.88	13.60 diaphragm b
4	2.35	10.88	13.50
	0.99	11.01	13.40
2	7.80	10.74	12.00
1	6.40	10.10	17.00
0			

APPROXIMATION II
APPROXIMATION I

12b   12c   12d

Fig. 12b THE RELATIVE LEVEL RIGIDITIES FOR FRAME.
Fig. 12c THE RELATIVE LEVEL RIGIDITIES FOR DIAPHR. b
Fig. 12d THE DISTRIBUTION OF SHEARS FORCES.

ASEISMIC STRUCTURE WITH LIFT SLABS (PROJECT)

19.30

34.0 m.

Fig. 13a

Fig. 12e

Fig. 13b

2197

Table 1

Zone	Surface focus		Intermediate focus	
	H=5...10 km	H=20 km	H=40...60 km	H=100...200 km
I	4,5	4,5...5,3	5,3...5,9	6,0... 6,9
II	4,5... 5,3	5,3...5,9	6,0...6,9	7,0... 7,7
III	5,3... 5,9	6,0...6,9	7,0...7,7	7,75... 8,5
IV	6,0... 6,9	7,0...7,7	7,7...8,5	8,5
IV	7,0... 7,7	7,75..8,5	8,5	

Note. In zones of deposits, it is appreciated that the intensity may increase by 1...2 steps, as compared with rocky zones at the same distance from the epicentral zones.

Table 4

Material of construction	Shear strain due to shear stress predominant	Strain due to normal stress predominant	
	Rigid diaphragms	Frames, trusses, flexible diaphragms	Slender flexible structures, chimney-stacks, radio masts, columns
Brick masonry	1,00	-	1,35
Concrete and R.C.	1,20	1,35	1,50
Steel	1,35	1,50	1,75
Timber	1,00	1,35	1,35
Massives structures	2,00	-	-

Note. In cases in which the procentual critical damping "n" can be appreciated from well established data, the factor should be computed by

Table 2

Classification of ground layers

Type	Description
F_1	Formations extending on large areas, of early tertiary or pro-tertiary age, composed of rocky eruptive sedimentalyer metamorphic layers, - massive, compact, - with very high allowable pressures ($\sigma < 10$ kg/cm^2).
F_2	Early tertiary formations or dilluvial deposits such as – marl rocks, – cemented sands, – sand - gravels, compact, of very small compressibility and high allowable pressures ($4 < \sigma < 10$ kg/cm^2), regardless of humidity ratio.
F_3	Dilluvial or alluvial formations of medium allowable pressures ($2 < \sigma < 5$ kg/cm^2) such as: – sandy grounds (of medium compressibility or compact), regardless of humidity; – clayey grounds (clays, sandy clays, clayey sands) dry or humid, with a consistency ratio $I_c > 0,75$, of medium compressibility; – sandy gravels – big-pore grounds dry or humid, of eolian or other origin
F_4	a. Alluvial formations, dry or humid, compressible, with low allowable pressures ($0,8$ kg/cm$^2 < \sigma < 2$ kg/cm^2) consisting of: – fine or dusty sandy soils, with or without gravel interstices; – clayey soils of plastic consistency; ($I_c = 0,50..$ $..0,75$)(dusts and clys) b. Ibidem, under water level.
F_5	Very recent fine alluvial formations, saturated, very compressible, unconsolidated with small allowable pressures ($\sigma < 0,8$ kg/cm^2) consisting of: – fine dusty fluid sands; – soft clayey grounds, fluid; – marshy soils (bogs) typical for Delta grounds for lake bottoms and for the inferiour part of certain siver beds; – macroporous grounds, inundated.

Notes. 1. "Alluvial formations" are formations made of boggy deposits, of sand, gravel and rocks, carried and deposited by water transport during and after the quaternary age.
2. "Dilluvial formations" are unconsolidated rocky formations on slopes or at base of slopes, deposited during and after the quaternary age, by accumulation of material destroyed by

atmospheric factors, under action of gravity and torrential waters.

3. Magnitudes of allowable pressures are only informative.

Table 3

Categories of grounds

Category	Stratification and thickness of the various types of formations		
I Very hard	F_1	F_2, F_3 / F_1 <3m.	
II hard	F_2 / F_1 >5m.	F_3 / F_1, F_2 <5m.	
III firm	F_3 / F_2 >5m.	F_{4a} / F_2, F_3 <5m.	
IV soft	F_{4a} / F_2, F_3 >5m.	F_{4b} / F_2, F_3 3÷10m	$F_{4a}+F_{4b}$ 10÷20m
V very soft	F_5, F_{4b} / F_2, F_3 >10m	$F_{4a}+F_{4b}$ / F_2, F_3 >20m	

Notes. 1. The thickness of the surface layer should be measured from the surface of the ground downwards.

2. The surface layer should be considered only if a thickness of minimum 2 m of the layer exists under the foundation level.

I

n — 21.5
43.0
i+1
43.0
i
43.0
i-1
43.0
4
43.0
3
43.0
2
43.0
1

21.5
64.5
107.5
150.5
193.5
236.5
279.5
322.5

$P'_{i,I} = G'_i$ $T'_{i,I}$

II

21.5
41.7
38.5
33.5
27.5
20.5
12.4
4.9

21.5
63.2
101.7
135.2
162.7
183.2
195.6
200.5

$P'_i \xi'_{i,\xi}$ $T'_{i,II}$

III

21.5
44.5
37.5
31.5
25.1
17.2
9.9
3.75

21.5
63.0
100.5
132.0
157.1
174.3
184.2
188.0

$P'_i \xi'_{i,III}$ $T'_{i,III}$

TABLE 5d

TABLE 5c

$u_{n,I}$ $u_{n,I}$ $u_{n,II}$

$u_{i,I}$

$u_{i-1,I}$

$\Delta_{i,s}$ $u_{i,I}$

$u_{i,II}$

$u_{i,III}$

$R_{i,I} = \dfrac{\Delta_{i,I}}{T'_{i,I}}$

$R_{i,II} = \dfrac{\Delta_{i,II}}{T'_{i,II}}$

$R_{i,III} = \dfrac{\Delta_{i,III}}{T'_{i,III}}$

$\xi_{i,I} = \dfrac{u_{i,I}}{u_{n,I}}$

K_{Di}

K_{CDi} K_{Ci}

K_{BCi} K_{Bi}

K_{ABi} K_{Ai}

A   B   C   D

n
i+1
i
i-1
4
3
2
1

Table 5.a

	hi	Li	Ki	Ki+1	Ji	fi	zi	fi-1	ni	Pi	zi-1	mi	ni
	m	dm3	dm3	dm3	dm3								
	The height of every Level	The same with (14)	The same with (3)	The corresp. values of (3)	(3)+(4)	(2)/(5)	2×(6)	The corresp. values of (6)	(7)×(8)	The same with (6)	The corresp. values of (7)	(10)+(11)	The same with (9)
	1	2	3	4	5	6	7	8	9	10	11	12	13
n													
n-1													
n-2													

	Pi	ri sup	Ei inf	The approximation	Fi	Ti	hi	mi	Ei	Mi inf	ri	Ni sup	Mi+1
					tone	tone	m	tm.		tm.		tm	
	(12)+(13)	(13)/(14)	(12)/(14)		The horiz. force applied at the level	The shear force at the level	The same with (1)	(18)×(19)	The same with (16)	(20)×(21)	The same with (15)	(20)×(23)	The corresp. values of (22)
	14	15	16		17	18	19	20	21	22	23	24	25
n				1 2 3 4									
n-1				1 2 3 4									
n-2				1 2 3 4									

	Mi	Mi	Ki	Pi	Mi	Li	fi	Mi-1	Pi	φi	hi	δi	Δi
	tm.	tm.	dm3	tm/bm3	tm.	dm3	tm/dm3	tm/dm3	tm/dm3	tm/bm3	m.	tm/dm3	cm.
	(24)+(25)	The same with (20)	The same with (3)	(27)/(28)	The same with (26)	The same with (2)	(30)/(31)	The corresp. values of (32)	The same with (29)	(32)+(33)+(34)	The same with (19)	(35)+(36)	(37)/A
	26	27	28	29	30	31	32	33	34	35	36	37	38
n													
n-1													
n-2													

Table 5.6

	Ti	Ri	Ui	Ui		Li	Ki	φi	ψi	COLUMN A	beam AB	COLUMN B	beam BC	COLUMN C
	tone	tone/cm	cm.	cm										
	The same with (18)	The relative levels rigid (20)/(38)	Ui-1+Δi	the deformation of the cracked structure	KABi+Kbi/Kcbi	2×(63)	Kai+Kai+Kcbi							
	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55

2202