BULETINUL INSTITUTULUI POLITEHNIC DIN IAŞI
Tomul LI (LV), Fasc. 3–4, 2005
CONSTRUCŢII. ARHITECTURĂ

NON-LINEAR SEISMIC ANALYSIS METHOD OF CONCRETE FRAME STRUCTURE

BY
V. FILIP and FELIX SCHÄRF

The proposed computation method principle utilized in the studied problem, which consist in solving the dynamic equilibrium differential equation system, written in the incremental form, using a hysteretic model of computing in analytical form, is presented.

1. General Considerations

It is well known that the actual design conception, described in the most of the technical codes, provide a direct evaluation of the behavior of structures in the non-linear domain, despite of the fact that during severe earthquakes take place deformations greater than the elastic limit, providing important structural energy dissipation capabilities. The obtained progress in the elaboration of some computing techniques concerning the structural design, leads, taking into consideration the non-linear behavior of the structural elements, including the stiffness of these variation of the mentioned elements during the earthquake, to the possibility of classification of the structural damages as structural or non-structural. Also, can be specified the structural areas that suffer excessive ductile damages and can provide data regarding the choosing of appropriate materials, in order to assemble the structural elements such as the necessary ductility is fulfilled.

As in the P100/1992 standard, there is no problem in adapting this method to the existing one introducing structural types, but for the structures that can't be included in the common type, because of their dimension, structural characteristics, special serviceability conditions, etc., these ones require the use of analysis methods based on advanced computing concepts.

2. Application Possibilities of the Proposed Analysis Method

2.1. The Principle of the Analysis Method

The proposed method consists in solving the dynamic equilibrium differential equation system, written in the incremental form, using numerical integration and a hysteretic model of computing in analytical form. From the initial state of the structure to final one sufficient large number of linearized steps are performed.

The method requires a large number of steps, and the precision of the results depends on the adopted hysteretic model and on the chosen time step (Δt).

The method involves the application of two distinct computing steps:

a) The first step, when the structural design is made according to the seismic standard P100/1992, defining the seismic actions as conventional lateral forces.

b) The second step, when the analysis is made in the non-linear domain and there are used gravity and seismic forces.

Because the stiffness degradation in every loading – unloading cycle of the frame elements, it is possible that the elements collapse takes place before the structure transforms into a mechanism, taking into consideration the hypothesis that the collapse appears in the same time when the collapse bending occurs in an element section. When justified, it is taken into consideration the hypothesis that the structural collapse occurs when the lateral displacements are larger than some pre-established values or when the displacements can produce important damages to the non-structural elements.

2.2. The Advantages of the Proposed Analysis Method

a) The analysis method allows to monitoring the structural elements' behavior from effort value point of view, as of the ductility during the earthquake, offering a clear image of the way the structure behaves.

b) Since the deformation capacity of reinforced concrete structures is limited, the method allows the verification of the elements' ductility, expressed by the ductility factor as the ratio between the collapse curve, ϕ_r, and the yielding curve, ϕ_c.

c) The definition of the stiffness matrix of a bar element through the flexibility matrix permits the analysis not only including lumped plastic hinges, but also distributes plastic hinges on a certain length of the element.

d) In this analysis method, the general stiffness matrices of the structure remain unchanged I shape from a step to another in the computation, despite of the fact that the stiffness or flexibility matrix of a bar element changes with every step of the analysis.

e) The method permits the conceiving of a general computing program available for any registered or adapted time history.

3. The Proposed Analysis Method

The differential equation system for the dynamic equilibrium is written in the incremental form as

$$(1) \qquad [m]\left\{\Delta\ddot{u}(t)\right\} + [C]\left\{\Delta\dot{u}(t)\right\} + [k_L(t)\{u(t)\}] = -[m]\{1\}\ddot{u}_g(t),$$

where the damping matrix of the structure, $[C]$, can be expressed in accordance with the linear combination between mass matrix, $[M]$, and stiffness matrix, $[k_L(t)]$, or in accordance to one of the two,

$$[C] = \beta[M] + \alpha[k_L(t)]; \quad [C] = \beta[M], \quad [C] = \alpha[k_L(t)]. \tag{2}$$

In the proposed analysis method it is used the expression $C = \beta[M]$, the value of β corresponding to the critical damping factor of the first mode of vibration even if this factor decreases in the higher modes.

The lateral stiffness matrix of the structure, $[k_L(t)]$, is a time function, its values modifying with each step of the analysis.

The solution of the differential equations system is obtained using the parameter β method (Newmark method). According to this method, the differential equations system is solved step-by-step using the equations,

$$\left\{\dot{u}\right\}_{i+1} = \frac{\Delta t}{2}\left\{\ddot{u}\right\}_i + \left\{\dot{u}\right\}_i + \frac{\Delta t}{2}\left\{\ddot{u}\right\}_{i+1}. \tag{3}$$

$$\left\{u\right\}_{i+1} = \{u\}_i + \frac{(\Delta t)^2}{2}\left\{\ddot{u}\right\}_i + \Delta t\left\{\dot{u}\right\}_i + \beta(\Delta t)^2\left\{\ddot{u}\right\}_{i+1} - \beta(\Delta t)^2\left\{\ddot{u}\right\}_i. \tag{4}$$

where β is a coefficient having values in terms of the form of variation of the time history variation in Δt time step. Good results are obtained considering a linear variation of the acceleration, when β has the value $1/6$. In this case the differential equations system is

$$\Delta\left\{\dot{u}\right\} = -\frac{\Delta t}{2}\left\{\ddot{u}\right\}_i - 3\left\{\dot{u}\right\}_i + \{\Delta u\}\frac{3}{\Delta t}, \tag{5}$$

$$\Delta\left\{\ddot{u}\right\} = -3\left\{\ddot{u}\right\}_i - \frac{6}{\Delta t}\left\{\dot{u}\right\}_i + \{\Delta u\}\frac{6}{\Delta t^2}. \tag{5}$$

The dynamic equilibrium equation system is

$$[k_L]^*\{\Delta u\} = \{\Delta P\}^* \tag{7}$$

or

$$\{\Delta u\} = [k_L]^{*-1}\{\Delta P\}^*, \tag{7'}$$

where

$$[k_L]^* = \frac{6}{\Delta t}[m] + \frac{3}{\Delta t}[C] + [k_L], \tag{8}$$

$$\{\Delta P\}^* = -[m]\{1\}\Delta\ddot{u}_g(t) + [m]\left(3\left\{\ddot{u}\right\} + \left\{\dot{u}\right\}\frac{6}{\Delta t}\right) + [C]\left(\left\{\ddot{u}\right\}\frac{\Delta t}{2} + 3\left\{\dot{u}\right\}\right). \tag{8'}$$

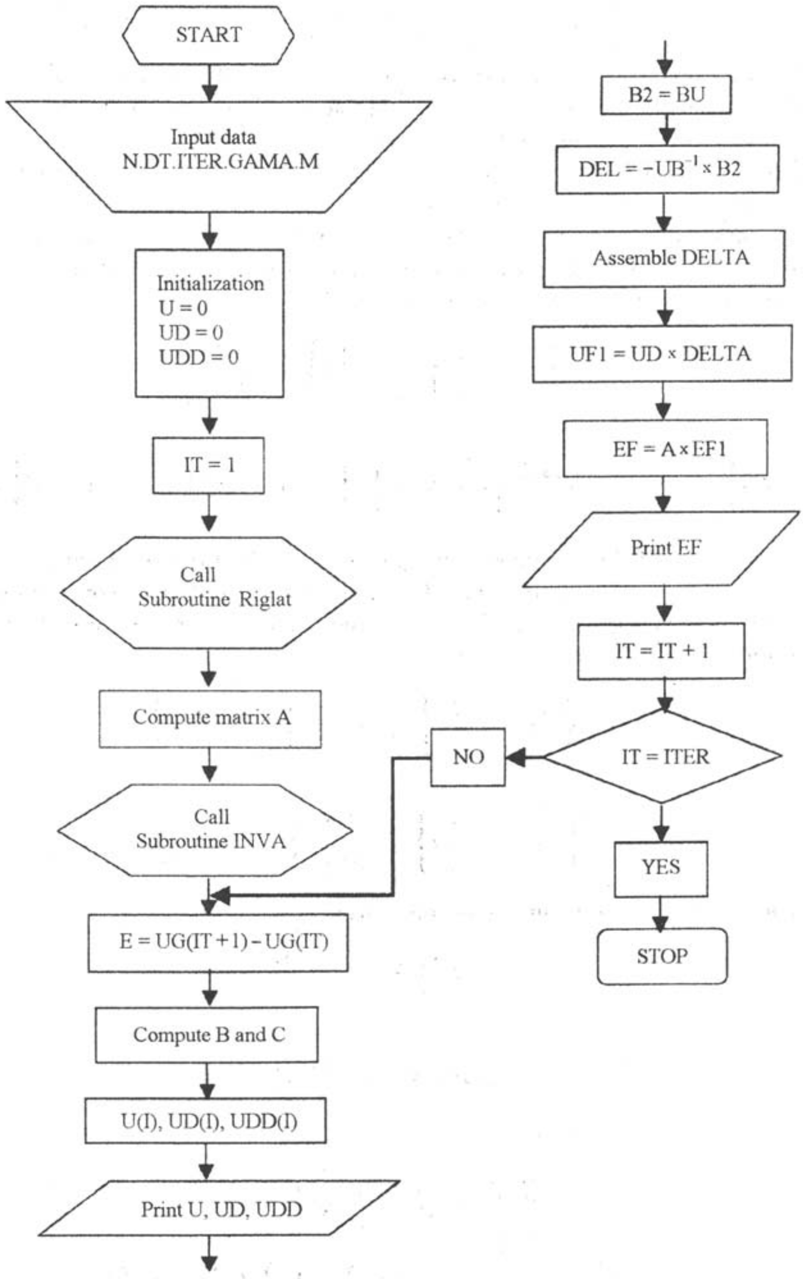
START
Input data
N.DT.ITER.GAMA.M
Initialization
U = 0
UD = 0
UDD = 0
IT = 1
Call
Subroutine Riglat
Compute matrix A
Call
Subroutine INVA
E = UG(IT + 1) - UG(IT)
Compute B and C
U(I), UD(I), UDD(I)
Print U, UD, UDD
B2 = BU
DEL = -UB^{-1} x B2
Assemble DELTA
UF1 = UD x DELTA
EF = A x EF1
Print EF
IT = IT + 1
IT = ITER
NO
YES
STOP

Fig. 1

Used notations:
N – number of levels
DT – time interval
ITER – number of iterations
GAMA – damping coefficient
M – diagonal mass matrix
UG – ground acceleration
U, UD, UDD – displacements, velocities and accelerations
DELTA – column vector of displacements
EF – efforts in the bar elements
Sub. RIGLAT – assembling matrix []
Sub. INVA – inverse of matrix []

Regarding the selection of the time step, Δt, in the reference material, it must be mentioned that it is less important the selection of a time step in order to characterize the precision of registering an earthquake, because some high frequency movements don't influence in a decisive manner the important modes of vibration and so the interpolation method can act as a filter. It is also mentioned that a time step of 0.01 s give good enough results. If the integration is made by successive iterations, the time step length can be given from the convergence and stability conditions.

The time step length, Δt, in the case of using the method of the first approximation, depends on the error that occurs by equivalation of the infinitesimal flexibility matrix with the incremental flexibility matrix.

The principal steps described in the algorithm in Fig. 1, are:

1° Chose of the maximum allowable time step, Δt_i, (where i is the incremental step of the analysis).

2° Reading the j ordinate of the ground acceleration for every time step, Δt_i.

3° Setting the initial condition for integration; at time $t = 0$ the structure will be under the action of the efforts resulted from the vertical loadings and the corresponding displacements, and the ground acceleration is considered null.

4° The assembling of the incremental stiffness matrices of the bar elements, according to the proposed hysteretic model.

5° Determination of the lateral stiffness matrix for every frame of the structure.

6° Determination of matrix $[K_L]^*$ and of vector $\{\Delta P\}^*$.

7° Determination of the vectors $\{\Delta u\}$, $\left\{\dot{u}\right\}$ and $\left\{\ddot{u}\right\}$.

8° Determination of the displacement vectors $\{x\}$, $\{\theta\}$.

9° Determination of the efforts in the frame elements, $\{S\} = [K]\{\Delta\}$.

10° For every bar element it is checked if the bending moment is larger than the collapse moment; it is computed the ductility factor. Also it is checked if it isn't over-passing the maximum time.

4. Recommendations Concerning the Design and Structural Assembling of Ductile Reinforced Concrete Frames

a) The element section will always have the shear capacity greater or equal than the bending capacity, such that the cross reinforcement will be determined taking into account the appearance of plastic hinges in the ends of the beams.

b) The check of the bending capacity is made by the yielding strength of the steel and not only by the stresses from compression or deformation of the concrete.

c) The concrete should be confined in the areas with large effort concentration and girder-column joints in order to avoid the collapse due to shear and to enhance the ductility to rotation.

d) It is assured the prolongation and the sufficient anchorage of the reinforcement.

5. Recommendations for Beam

a) The beam dimensions – the width must follow some minimum limits.

b) It is recommended a minimum reinforcement of the beams, depending on the yielding limit of the steel, expressed by the positive coefficient of the reinforcement.

c) At the supports (at joints) the positive plastic moment of the beams should not overpass half of the negative moment.

d) The beam reinforcement necessary to take over the negative moments in the adjacent section to the columns will have the following reinforcement coefficient limits: 0.5%...2% when it is used steel OB38, 0.4%...1.5% when it is used steel PC52.

e) It is necessary to follow some recommendations regarding the reinforcement anchorage near the columns. The ϕ_d values vary in accordance with the diameter and the size of the reinforcement and concrete.

f) Because the reinforcement ensures a large ductility for the sections, it is necessary the lengthening of the negative moment reinforcement (at least 1/3) on some distance from the bays ($L_e/4$).

g) Also, the fourth part of the reinforcement necessary to take over the negative moment from every edge of the beam will be continued on the upper part of the beam. The bay was denoted with L_e and the negative reinforcement with A_N.

h) At the middle of the beams, the transverse reinforcement will take over the shear force produced by the vertical loadings and from the plastic moments at the ends produced by the lateral forces. The minimum dimension of the transverse reinforcement is of 8 mm; the maximum distance is minimum $h/2$ and 20 cm. In the support area of the beams can be placed stirrups on a length at least equal to 1/4 from the beams bay.

6. Recommendations for Columns

a) The width must be greater than 24 cm or 1/20 of the free height; the side's ratio mustn't be bigger than 2.5.

b) In cases when the column's section is enlarged for some construction reasons, such as from computation doesn't results the necessary reinforcement, the minimum reinforcement coefficient over the whole concrete section will be equal to 0.5% for OB38 and to 0.4% for PC52 steel.

c) For a joint it is recommended that the plastic moment capacity I columns (dead load and live-axial load) shall be greater than the sum of plastic moment capacities of the beams. If this condition isn't fulfilled, the sum of the moment capacities for the confined bulbs of the columns is sufficient to resist to the axial and moment designed load.

d) The transverse reinforcing of the column in the central area must take over the shear force obtained from vertical loads and the appearance of plastic hinges.

e) The minimum distance between stirrups, on the central area of the column must be chosen as the minimum between 20 cm and $0.5h$.

f) At the ends, on a minimum distance equal to the maximum value between the greatest dimension of the column (b, h), 1/6 from the free height L1 or 60 cm, the concrete is confined or stirrup, at distances less than 10 cm.

g) It will be assured the continuity of the longitudinal reinforcements and of the stirrup for the columns over the height of intersections with the beams.

h) It is recommended that there is no beak from the but-joining of the longitudinal reinforcements of the column, lengthening the over-passing area.

7. The Hysteretic Model of Analysis

As described above, the method allows the input of any hysteretic model of analysis (Figs. 2,...,4). Of course, the more complicated is the hysteretic model, the more difficult the computation is, but the results are more reliable.

In the reference literature are presented simplified hysteretic models. It is worth recalling the bilinear hysteretic model (Fig. 2), the hysteretic model with decreasing stiffness (Fig. 3) and the hysteretic model Ramberg – Osgood with decreasing stiffness (Fig. 4).

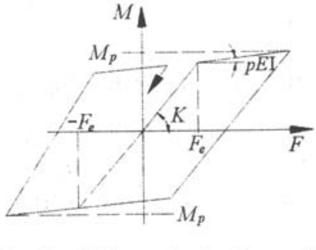

Fig. 2.– Bilinear hysteretic model.

The use of the models with broken branches make easier the computational method because the establishing of the stiffness matrix can be made directly and the change is made in sudden leaps/steps from one value to another.

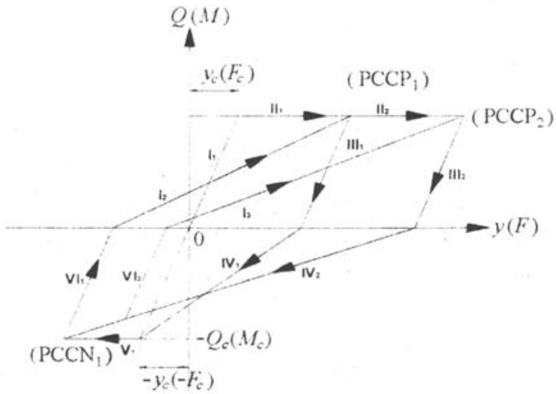

Fig. 3.- Hysteretic model with decreasing stiffness.

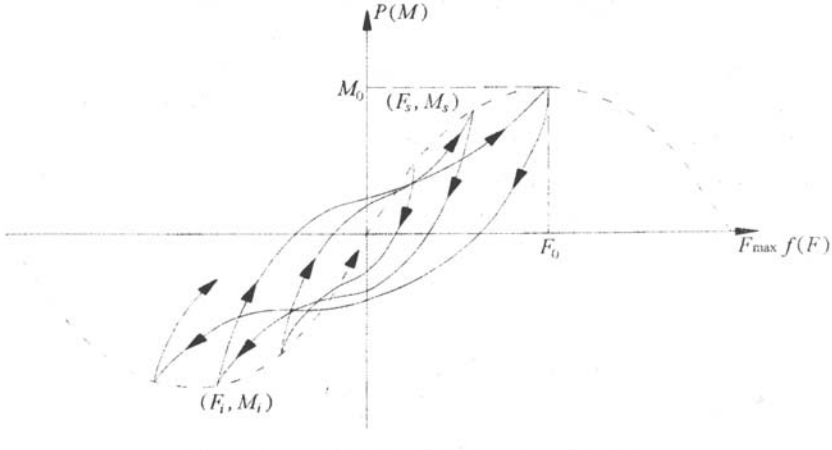

Fig. 4.- Hysteretic model Ramberd - Osgood.

Received, November 8, 2005

"Gh.Asachi" Technical University, Jassy,
Department of Structural Mechanics

REFERENCES

1. Negoiţă Al., Schärf F., *Cursuri Postuniversitare*. Inst. Politehnic, Iaşi, 1976.
2. Negoiţă Al., *Aplicaţii ale Ingineriei Seismice*. Vol. **I**, Edit. Tehnică, Bucureşti, 1988.
3. Negoiţă Al., *Aplicaţii ale Ingineriei Seismice*. Vol. **II**, Edit. Tehnică, Bucureşti, 1990.

METODĂ DE ANALIZĂ SEISMICĂ NELINIARĂ A STRUCTURILOR DIN CADRE DE BETON ARMAT

(Rezumat)

Se descrie principiul metodei de calcul a problemei studiate, care constă în soluţionarea sistemului de ecuaţii diferenţiale de echilibru dinamic, scris sub formă incrementală, utilizând un model histeretic de calcul sub formă analitică.