საქართველოს მეცნიერებათა ეროვნული აკადემიის მოამბე, ტ. 4, №1, 2010
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Structural Mechanics

On One Nonlinear Dynamic Iterative Method of Structure Analysis

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ABSTRACT. The paper discusses one method of structure analysis, given in European Standard EC-8, part 3, the essence of which can be formulated as follows: the response spectrum analysis of structure to full seismic action without the reduction coefficient is conducted, and verification of structure elements in the field of the obtained displacements is carried out to see how much they comply with this field of displacements. We propose to continue this procedure by way of iteration, the aim of which is to make more accurate the obtained solution. The paper presents the description of this iteration process and its demonstration with simple example is given. The method can be extended to tasks of dynamics when time participates in explicit form.

Key words: *EC-8, seismic action, building nonlinear analysis, iteration procedure, dynamic tasks.*

If we take a view of history, we can note two significant points of inadequate response to real circumstances by the earthquake engineering ideology. One was in the second half of the 20th century, when instrumental data were accumulated confirming that at earthquakes accelerations of higher level develop on the earth's surface than believed earlier. At that time the earthquake engineering ideology failed (nor was it able) to switch actively to nonlinear models of structure analysis, and it introduced the conception based on the so-called reduction coefficient. The second point refers to the beginning of the 21st century, when the unreliability of this conception became clear. Though the necessity of nonlinear analysis was verbally declared, first very cautiously, then more strongly, the so-called "Pushover" method assumed the leading function, even in normative documents, thus an insufficiently verified, not clearly formulated model of structure nonlinear analysis was adopted [1].

Georgia has the oldest tradition of earthquake engineering regulation. After the strongest Leninakan earthquake - (Armenia 1926), in the Institute of Structures in Tbilisi codes were elaborated that can be considered as first in the world, based on the dynamical theory of structure analysis. Our Institute led earthquake engineering in different directions throughout the Soviet Union for decades.

Today Georgia is leaving the Soviet sphere of engineering regulation. We attentively watch the trends in the sphere of earthquake engineering, particularly those in Europe, as far as we consider harmonization with its engineering regulation sphere to be our main guiding line.

In this connection we wish to touch upon a European document of recent years - Eurocode 8: Design of structures for earthquake resistance, part 3: Assessment and retrofitting of buildings, especially to one model of structure analysis, described in this document. The essence of this model can be explained in the following way: the response spectrum analysis of structure to full seismic action is conducted without reduction coefficient. In the field of displacements $U_1(x)$ obtained in this way, the verification of structure elements is carried out. If ductile elements have the capacity with certain reserve to be "stretched" over these displacements, and the brittle ones at these displacements do not exceed the limit of strength, then an inference about the structure reliability can be made. We propose to continue this procedure. We consider that the field of displacements $U_1(x)$, obtained as a result of calculation of the body of nonlinear behaviour, in particular, structure, by means of a linear analytical model, can be viewed as the first approximation of an iteration process. In "verification" of structure elements in this field of displacements, it will revealed that the nonlinear structures comply to this field of displacements only at the expense of the effect of additional "fictitious" external forces $P_1(x)$. The value of these fictitious external forces $P_1(x)$ themselves is determined during the verification process. It turns out that the field of displacements $U_1(x)$ represents the result of two effects. One is the seismic effect, and the other - additional, known fictitious external forces $P_1(x)$.

It is natural to attempt to exclude the field of displacements caused by fictitious $P_1(x)$ forces. To this end the structure must be calculated under the effect of fictitious external forces $P_1(x)$. This must be done by way of calculation, using the linear model, and the second approximation of the field of displacements $U_2(x)$ will be obtained. Again, as a result of verification, a new approximation of fictitious external forces $P_2(x)$ will be obtained, and so on. Finally, as a result of such an iteration process, the field of displacements will be obtained due only to the seismic effect, which constitutes our goal.

To illustrate the above stated, a simplified example is given below. Two bars (*ab*) and (*bc*) are connected to each other. The point (*a*) is embedded, and the point (*c*)

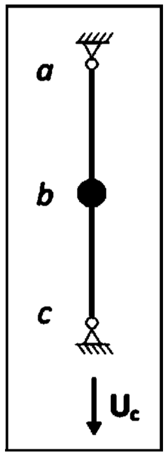


Fig. 1. Two bars (*ab*) and (*bc*) with different mechanical properties are connected to each other and suffer tension due to the displacement of support (*c*).

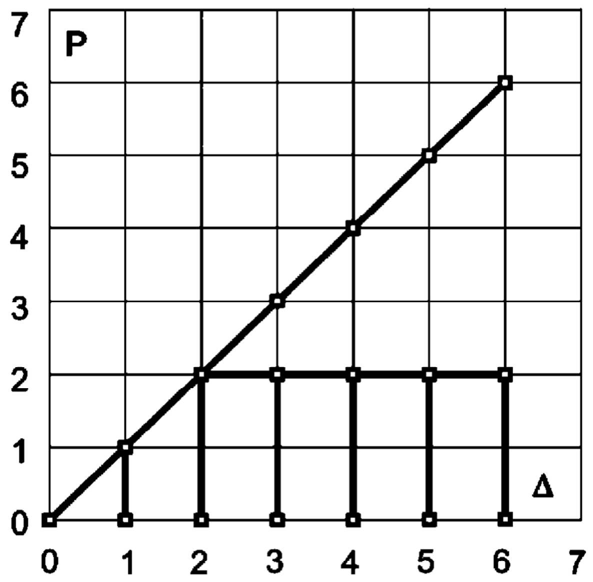
is displaced downwards and it stretches the bar (Fig.1)

The bars are characterized by different mechanical properties (Fig.2). The bar (*ab*) has the capacity of plastic compliance, and the second - (*bc*) is the brittle element of high strength (Fig.2).

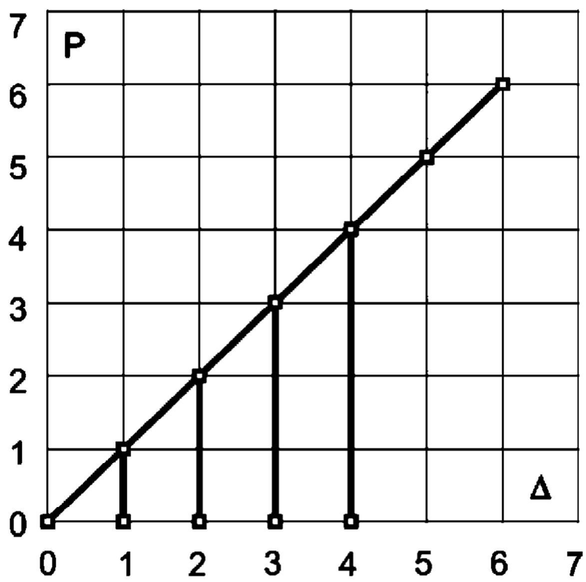
Here it must be noted that, if the strength of the ductile element (*ab*) exceeds that of the brittle element, the total bar can not reveal ductility, it will collapse brittle.

Fig. 3 shows the displacement of point (*b*) at displacements of point (*c*) one after another at distances U_c=2Δ, 4Δ, 6Δ and 8Δ, considering the real mechanical properties of the bars.

We consider it necessary to give the following explanation. In Fig.3 the Δ elongations are shown in hy-



a)



b)

Fig. 2. a) Dependence between the increase (Δ) of bar (*ab)* and corresponding force P. b) The same holds for the bar (*bc*).

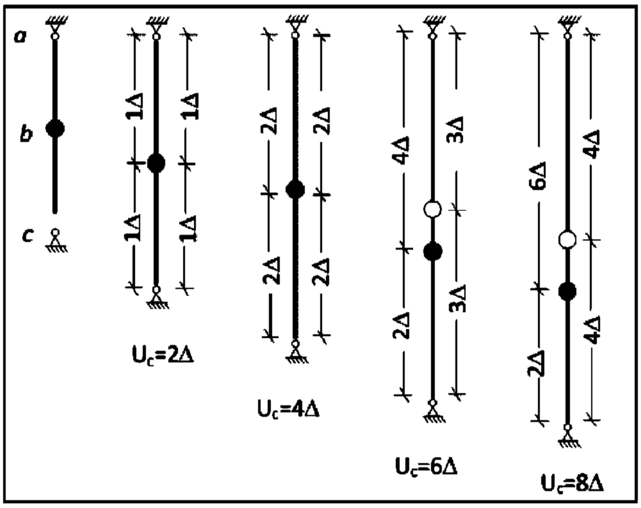


Fig. 3. Displacement of point (*b*) at displacements of point (*c*) one after another at distances $U_c=2\Delta$, 4Δ, 6Δ and 8Δ.
• – considering the real mechanical properties of the bars,
o – according to results of linear analysis.

same degree as the lengths of the bars have. In reality, we have to imply, that Δ elongations are smaller as compared to the sizes of bars, allowing us to neglect the elongations of the bars in the calculation under the effect of external forces, which will be carried out further.

Fig. 3 shows that at the two initial stages of tension, when the point (*c*) is displaced at distances 2Δ and 4Δ, the total bar works in linear regime. The result of linear analysis confirms the same – in the figures the empty bulbs are simply not shown, since their locations coincide with the black ones .

However, by now, at displacement $U_c= 6\Delta$, the displacement of point (*b*), obtained by linear calculation, cannot reflect the picture of real nonlinear deformation , the black symbol does not coincide with the empty symbol. If we stopped at this stage, we could say on the basis of verification that at tension at value 3Δ, neither the strength of the lower bar, nor the ductility of the upper one are exhausted. At the same time we could evaluate how close the lower bar is to the limit strength, and the upper element to limit ductility. We propose to consider this stage as the first one of the iteration process and to continue it as a result of which the picture of total bar deformation and the reserves of strength and ductility will be evaluated more adequately.

Let us to examine in more detail this condition of the bar (see Fig. 4). The Figure shows the result obtained by linear analysis, i.e. the upper as well as the lower bar is stretched out at value 3Δ. Now let us try to apply our real bars to these tensions. To stretch the lower bar at value 3Δ, we have to apply the forces 3P to it, but a force higher than 2P cannot be applied to the upper bar. The mentioned bars with applied forces are shown in Figure 4.c. If we connect these bars to each other (Fig.4.d), we will see that they comply with the displacements obtained as a result of calculation, however, at point (*b*) the external force P_1=1P will appear. I.e., it can be stated that as a result of calculation by the linear method, the displacement of point (*b*) is obtained, which represents the result of two effects, one being the forced displacement of the point (*c*) of value $U_c=6\Delta$, and the second - displacement due to the extra P_1=1P force. This force has moved up the point (*b*), which is seen well in Figure 3.

The next stage of the attempt is to remove the field of displacements caused by the effect of the extra P_1=1P force. To this end, this force with opposite sign should be applied to point (*b*) of the bar. As far as we have stated that we are not capable of carrying out a nonlinear analysis of the bar, we again conduct calculation by linear model as a result of which we get a new displacement 0.5Δ of point (*b*) and the corresponding new fictitious extra P force, and so on. Finally, at point (*b*) the force $Q = 1+0.5+0.25+0.125+\ldots \cong 2$ will be applied, as a result of the action of which, the displacement of point (*b*), obtained by the iteration process, will coincide with the real displacement of point (*b*).

The strength and ductility of the structure, in our case of the total bar can now be evaluated considering this circumstance. The strength reserve will prove higher and the ductility reserve – less as compared to that given by the first stage of iteration. The picture of deformation, caused by displacement 8Δ of the point (*c*) , can be analyzed in the same way.

This is the essence of the iteration procedure proposed by us. Evaluating the method, given in EC-8, Part 3., having in view this procedure, it can be considered

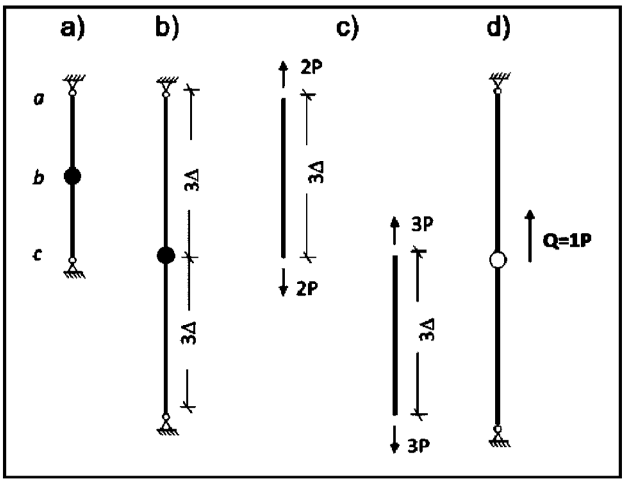


Fig. 4. As a result of verification of real bars at displacements obtained by linear calculation, the fictitious external force Q=1P arises.

as the first stage of the iteration procedure. We are pleased with this method and propose its logical continuation by way of addition of an iteration process.

Our discussion and case study dealt with a static task in which the inertia forces do not take part. The same can be extended to tasks of dynamics quite naturally, in particular, the seismic effect can be represented in the form of an accelerogram or seismogram, and the model of the behaviour of the structure can be a linear model involving time. In this case, the field of displacements $U_1(x,t)$, depending on time (t), can be considered as the first stage of the iteration process as well. Here the first approximation of fictitious external forces $P_1(x,t)$ is determined and applied to the structure with an opposite sign, as a result, the second approximation of the field of displacements $U_2(x,t)$ is determined, and so on.

სამშენებლო მექანიკა

ნაგებობათა არაწრფივი დინამიკური გაანგარიშების ერთი იტერაციული მეთოდის შესახებ

გ. გაბრიჩიძე

აკადემიის წევრი, კ. ზავრიევის სამშენებლო მექანიკის და სეისმომედეგობის ინსტიტუტი, თბილისი

სტატიაში განხილულია ნაგებობათა არაწრფივი დინამიკური გაანგარიშების მეთოდი, რომელიც შემოთავაზებულია სეისმომედეგი მშენებლობის რეგულირების ევროპულ დოკუმენტ EC-8,3-ში. მისი არსი შეიძლება ასე ჩამოვაყალიბოთ: ნაგებობა გაიანგარიშება სპექტრალური მეთოდით სრულ სეისმურ დატვირთვაზე, რედუქციის კოეფიციენტის გარეშე. ამ გზით მიღებულ გადაადგილებათა ველზე მოწმდება ნაგებობის ელემენტები – რამდენად შეძლებენ ისინი მოერგონ ამ გადაადგილებებს ისე, რომ არ გადააჭარბონ სიმტკიცის ზღვარს, და არ ამოწურონ დამყოლობის უნარი.

სტატიაში შემოთავაზებულია ამ მიდგომის გაგრძელება იტერაციული პროცესის დამატებით და მეთოდის გავრცელება დინამიკურ ამოცანებზე, რომლებიც ცხადი სქემით შეიცავენ დროს.

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