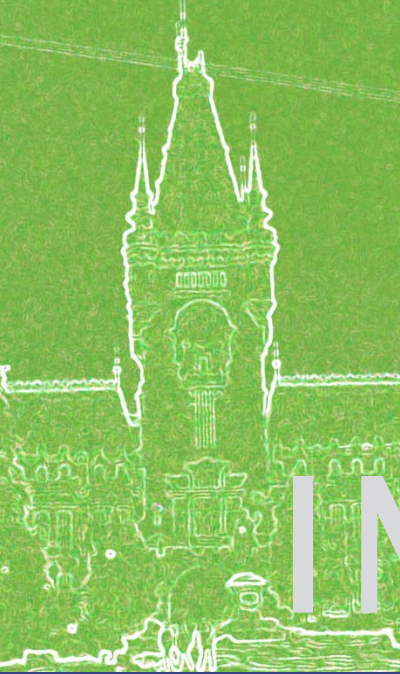


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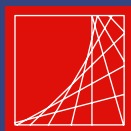
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Boundary element method numerical modeling of RC plane stress cracked plate

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Summary

In this paper different methods of distribution solution of RC cracked of the plane stress plates are shown. The differential equation of the cracked plates, using the classical variational method of Lagrange is worked out. The displacements equations with the boundary conditions and compatibility conditions in the crack are obtained. The total differential equations in the class of the two-dimensional general vector functions are shown. In this model the effect of discontinuity general deformation vector is taken into account. As the next the viscoelastic plate model has been derived by the variational method of Gurtin in the space of general function. The numerical results of an approximate method of solutions with boundary element method are shown.

Keywords: boundary element method, viscoelastic plate model, RC plane stress cracked plate

1. INTRODUCTION

The RC concrete plates are non-homogeneous. Therefore the response of so heterogeneous structures and additionally defects caused by cracks in concrete to applied actions is generally nonlinear, due to nonlinear constitutive relationships of the materials, known as mechanical nonlinearity and to second order effects of normal forces, known as geometrical nonlinearity. Regard of defects in form of cracks treated as continuous functions, which are usually based on the continuum mechanics approach, gives unsatisfied solution because of summation of assumption errors and solution errors. Therefore the proper mathematical modeling of plate is so important since the final error appears solely in solution phase.

This paper contains a mathematical model of a reinforced concrete plane stress plate formulated in terms of general functions. The physical hypothesis about discontinuous change of displacement vector, caused by cracking of extension zone in the concrete, was taken in the model. Such assumptions and first investigation for distribution model of RC beam with cracks were made by Borcz in 1963 [1].



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This paper expands distribution beam Borcz's model for RC cracked plane stress plates.

The assumption of distribution theory of Schwartz [2] affords possibilities for precise mathematical description of discontinuity of the plate. Fundamental for understanding of the next consideration are the general distributions of Dirac's- δ with given density on the curve $\Lambda \in \mathbb{R}^2$ and following properties:

$$\langle \mathbf{y} \delta_{\Lambda}, \mathbf{j} \rangle = \int_{\Lambda} \mathbf{y}(\mathbf{x}) \mathbf{j}(\mathbf{x}) d\Lambda, \text{ where } \mathbf{x} = (x_1, x_2). \quad (1)$$

$$\langle D^{\alpha}(\chi(\mathbf{x})\delta_{\Lambda}), \varphi(\mathbf{x}) \rangle = (-1)^{|\alpha|} \int_{\Lambda} \chi(\mathbf{x}) D^{\alpha} \varphi(\mathbf{x}) d\Lambda, \text{ where } \alpha = (\alpha_1 + \alpha_2), |\alpha| = \alpha_1 + \alpha_2. \quad (2)$$

Here the functions $\psi(\mathbf{x})$, $\varphi(\mathbf{x})$ and $\chi(\mathbf{x})$ are continuous functions on the curve Λ in the space \mathbb{R}^2 .

The functionals formulated above for unitary density of function $\psi(\mathbf{x})$ and $\chi(\mathbf{x})$ have the analogue filtering property as for the general distribution of Dirac's- δ . It means that the $\psi(\mathbf{x})$ and $\chi(\mathbf{x})$ have the value of function $\varphi(\mathbf{x})$ or its derivatives respectively for the arguments \mathbf{x} belong to the curve Λ .

The arbitrary plane stress plate is considered. The plate has arbitrary homogeneous boundary conditions and is arbitrary forced, see Figure 1. The region of plate Ω is divided by the curve Λ , means the crack, in two zones Ω_1 and Ω_2 with bound $\partial\Omega_1$ and $\partial\Omega_2$. The curve Λ has two ends Λ_1 and Λ_2 . The normal external direction cosines of edge Λ of regions Ω_1 and Ω_2 have different sign. The considered model can be easily generalized to any amount of cracks Λ .

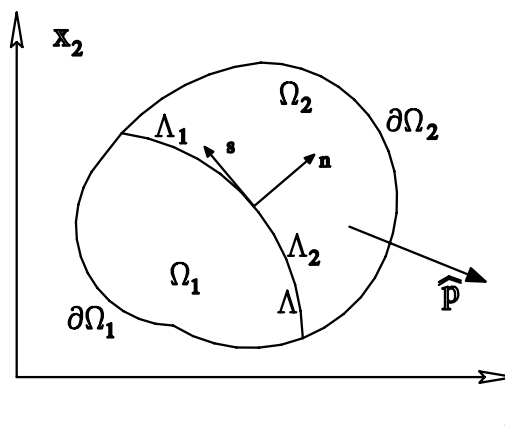


Figure 1. Scheme of plane stress plate with crack Λ



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2. DIFFERENTIAL EQUATION FOR DISPLACEMENT

The discontinuous variational problem of surface integral for displacement was solved. The equilibrium equations, constitutive law and strain equations are assumed to be represented by well-known theory of elasticity relations. We are looking for the extreme of the functional of strain energy U_s with set of permissible displacement value $\mathbf{u}(\mathbf{x})$, where $\mathbf{b}(\mathbf{x})$ means body forces:

$$J[\mathbf{u}(\mathbf{x})] = \int_{\Omega} U_s(\mathbf{u}(\mathbf{x})) d\Omega - \int_{\Omega} \mathbf{b}(\mathbf{x}) \mathbf{u}(\mathbf{x}) d\Omega - \int_{\partial\Omega} \mathbf{p}(\mathbf{x}) \mathbf{u}(\mathbf{x}) d\mathcal{A}, \quad (3)$$

where $\mathbf{u}(\mathbf{x})$ means displacement vector and $\mathbf{b}(\mathbf{x})$ body forces respectively. The searching function $\mathbf{u}(\mathbf{x})$ is in the class of function $\mathbf{u} \in C^2(\Omega/A)$ (for $\mathbf{x} \in A$ function $\mathbf{u}(\mathbf{x})$ has singularity).

Applying Green's transformation with well known relations stress-strain-displacement $\mathbf{S}-\mathbf{E}-\mathbf{u}$ we obtain differential equation of plate in plane stress:

$$\mu(\nabla^2 + \frac{3\lambda + 2\mu}{\lambda + 2\mu} \text{grad div}) \mathbf{u}(\mathbf{x}) + \mathbf{b}(\mathbf{x}) = 0, \quad (4)$$

associated with combination of elementary boundary conditions:

$$\tilde{\mathbf{P}}(\mathbf{u}(\mathbf{x})) = \mathbf{p}(\mathbf{x}), \quad \text{for } \mathbf{x} \in \Omega_1 \cup \Omega_2, \quad (5)$$

and compatibility condition in the crack:

$$[\tilde{\mathbf{P}}(\mathbf{u}(\mathbf{x}))]_{\Lambda} = 0, \quad \text{for } \mathbf{x} \in \Lambda_1 \Lambda_2, \quad (6)$$

where λ and μ are Lamé constants, $\tilde{\mathbf{P}}$ means operator of surface tension:

$$\tilde{\mathbf{P}}(\cdot) = \mu(\tilde{\nabla} + \frac{2\lambda}{\lambda + 2\mu} \mathbf{1} \text{ div})(\cdot) \mathbf{n}, \quad (7)$$

where $\mathbf{1}$ is a unitary tensor, whereas \mathbf{n} represents normal vector external to the edge $\partial\Omega$.

Here $[\cdot]_{\Lambda}$ means difference of left and right side limit of expression in square braces on the curve Λ .

Constitutive law of defect is assumed to be represented as follows (see beam analogy of Borcz [1]):

$$[\mathbf{u}(\mathbf{x})]_{\Lambda_1 \Lambda_2} = \mathbf{r}(\mathbf{x}), \quad \text{with condition } \frac{\partial \mathbf{r}}{\partial \mathbf{s}}(\Lambda_1) = \frac{\partial \mathbf{r}}{\partial \mathbf{s}}(\Lambda_2) = 0. \quad (8)$$



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Here $\mathbf{r}(\mathbf{x})$ means density of defect as a continuous function for $\mathbf{x} \in \Lambda_1 \Lambda_2$ and $[\mathbf{u}]_\Lambda = 0$ for $\mathbf{x} \notin \Lambda_1 \Lambda_2$. Equation (8) satisfies compatibility condition in the crack, where the displacement vector has a jump on a bound of crack. Here the assumption of internal crack $\Lambda_1 \Lambda_2$ was taken. This can be easily proved. Hence on the remaining part of curve Λ the condition $[\mathbf{u}]_\Lambda = 0$ yields, for $\mathbf{x} \notin \Lambda_1 \Lambda_2$. Moreover the second condition (8) in the essential way completes the definition of the defect.

It can be easily shown that using Eq. (1)-(2) and Eq. (5)-(8) differential equation (4) has the following form (see [3]).

$$\mu \left\langle \left(\nabla^2 + \frac{3\lambda + 2\mu}{\lambda + 2\mu} \text{grad div} \right) \mathbf{u}(\mathbf{x}), \boldsymbol{\varphi} \right\rangle + \langle \mathbf{b}(\mathbf{x}), \boldsymbol{\varphi} \rangle = \int_{\Omega} [\mathbf{u}(\mathbf{x}) \tilde{\mathbf{P}}(\boldsymbol{\varphi}) + (\mathbf{p}(\mathbf{x}) - \tilde{\mathbf{P}}(\mathbf{u}(\mathbf{x})) \boldsymbol{\varphi}] d\mathcal{X}\Omega + \int_{\Lambda_1 \Lambda_2} \mathbf{r}(\mathbf{x}) \tilde{\mathbf{P}}(\boldsymbol{\varphi}) d\Lambda . \quad (9)$$

Using functional way of description with distribution in form of Dirac's- δ we can write final general differential equation of RC cracked plate in plane stress, appropriate boundary and compatibility conditions in the crack respectively:

$$\mu \left(\nabla^2 + \frac{3\lambda + 2\mu}{\lambda + 2\mu} \text{grad div} \right) \mathbf{u}(\mathbf{x}) = -\tilde{\mathbf{P}}(\mathbf{r}(\mathbf{x}) \delta_\Lambda) + (\mathbf{p}(\mathbf{x}) - \tilde{\mathbf{P}}(\mathbf{u}(\mathbf{x})) \delta_{\Omega_1} + \tilde{\mathbf{P}}[(\tilde{\mathbf{u}}(\mathbf{x}) - \mathbf{u}(\mathbf{x})) \delta_{\Omega_2}] \quad (10)$$

where $\mathbf{b}(\mathbf{x})$ was taken in following form: $\mathbf{b}(\mathbf{x}) = \tilde{\mathbf{P}}(\tilde{\mathbf{u}}(\mathbf{x})) \delta_{\Omega_2}$.

The assumption of the jump of displacement vector $\mathbf{u}(\mathbf{x})$ was proved by experimental study [1]. Density of defect also known as constitutive law of crack opening, is the function of tension vector \mathbf{N} acting in the crack:

$$\mathbf{r}(\mathbf{x}) = \mathbf{r}^0(\mathbf{x}) - \mathbf{r}^1(\mathbf{x}) \mathfrak{Z}(\mathbf{N}(\mathbf{x})) \Big|_\Lambda, \quad \mathbf{x} \in \Lambda_1 \Lambda_2 . \quad (11)$$

Here \mathbf{r}^0 describes residual general deformations whereas $\mathbf{r}^1 \mathfrak{Z}(\mathbf{N})$ elastic general deformations respectively.

The zone of plate Ω_1 is connected with another one Ω_2 by means of reinforcement bars appearing in the cracks. So, the edges of the cracks are not free from tensions at the points of connections.

The components \mathbf{r} of Eq. (11) are given from RC element tests as well as from general assumption of crack theory and equilibrium conditions in the crack.

Using definition of convolution and Green function satisfying equation $\Delta \Delta \mathbf{G}(\mathbf{x}) = \delta(\mathbf{x})$ the solution of Eq. (10) is described in following form:



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$$\mathbf{u}(\mathbf{x}) = \int_{\Lambda_1 \Lambda_2} [\mathbf{r}^0(\mathbf{y}) + \mathbf{r}^1(\mathbf{y}) \mathfrak{Z}(\mathbf{N})] \tilde{\mathbf{P}}(\mathbf{G}(\mathbf{x}, \mathbf{y})) d\Lambda + \int_{\tilde{\Omega}} \{ \tilde{\mathbf{P}}(\mathbf{G}(\mathbf{x}, \mathbf{y})) [\mathbf{u}(\mathbf{y}) - \tilde{\mathbf{u}}(\mathbf{y})] - \mathbf{G}(\mathbf{x}, \mathbf{y}) [\tilde{\mathbf{P}}(\mathbf{u}(\mathbf{y})) - \mathbf{p}(\mathbf{y})] \} d\tilde{\Omega} \quad (12)$$

Here the curvilinear integrals for the edge Λ can be interpreted as a some external force modeling the defect. It can be proved [3] that for plane stress these integrals are the normal and tangential dipole forces cause the jump of displacement vector in the crack. The acting forces are self-balanced and do not cause the increments of external loading of construction.

The final solution of singular integro-differential Eq. (12) describes an accurate mathematical model of cracked RC in plane stress.

The presented solution includes the discontinuity of general deformations in the crack places and simultaneously satisfying continuity of general tension vector on both sides of the defect Λ . Such formulated model of RC cracked plate will be solved with the help of boundary element method (BEM).

3. DIFFERENTIAL EQUATION OF VISCOELASTIC PLATE

The same way as in chapter 2 was applied to describe solution of viscoelastic plane stress plate. The equilibrium equations in form: $\text{div } \mathbf{S} + \mathbf{b} = \mathbf{0}$, and geometrical

relations in form: $\mathbf{E} = \frac{1}{2}(\nabla \mathbf{u} + \nabla \mathbf{u}^T) = \tilde{\nabla} \mathbf{u}$.

The set of field equations is fulfilled in the space $\Omega \times [0, \infty)$, where $[0, \infty)$ is time interval. The initial condition of strains' tensor has to be added $\mathbf{E}(\cdot, 0) = \mathbf{E}^0$, for $t = 0$.

The physical law well known as Boltzmann type is taken in following form:

$$\mathbf{1} * \mathbf{S} = \psi_1 * \mathbf{E} + \frac{1}{2}(\psi_2 - \psi_1) * \mathbf{1} \text{ tr } \mathbf{E} - \mathbf{F} \quad , \quad \text{where:} \quad (13)$$

$$\mathbf{F} = \psi_1 * \mathbf{E}^0 + \frac{1}{2}(\psi_2 - \psi_1) * \mathbf{1} \text{ tr } \mathbf{E}^0 \quad , \quad (14)$$

with convolution rule: $f(t) * g(t) = \int_0^t f(t - \tau) g(\tau) d\tau$. Here the functions ψ_1 and ψ_2 are the functions of relaxation.



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The discontinuous viscoelastic variational problem of Gurtin type, in the some way as considered in chapter 2, was solved. The equilibrium equations, Boltzmann constitutive law, strain equations and initial condition are assumed to be represented by well-known theory of elasticity relations.

Analogue to the functional (3) we are looking for the extreme of the functional of strain energy U_v with set of permissible displacement value $\mathbf{u}(\mathbf{x})$:

$$J[\mathbf{u}(\mathbf{x}, t)] = \int_{\Omega} U_v(\mathbf{u}(\mathbf{x}, t)) d\Omega - \int_{\Omega} \mathbf{1} * \mathbf{b}(\mathbf{x}, t) * \mathbf{u}(\mathbf{x}, t) d\Omega - \int_{\partial\Omega} \mathbf{1} * \tilde{\mathbf{p}}(\mathbf{x}, t) * \mathbf{u}(\mathbf{x}, t) d\mathcal{A}, \quad (15)$$

Applying Green's transformation with material and field relations we obtain differential equation of plate in plane stress:

$$\mathbf{1} * \mathbf{S} = \psi_1 * \mathbf{E} + \frac{1}{2} (\psi_2 - \psi_1) * \mathbf{1} \operatorname{tr} \mathbf{E} - \mathbf{F}, \quad (16)$$

associated with combination of elementary boundary conditions for free and fixed edges respectively:

$$[\tilde{\mathbf{P}}(\mathbf{u}(\mathbf{x})) - \mathbf{1} * \tilde{\mathbf{p}}(\mathbf{x})]_{\partial\Omega_1} = 0 \quad \vee \quad \delta \mathbf{u}(\mathbf{x})|_{\partial\Omega_2}, \quad (17)$$

and compatibility condition in the crack:

$$[\tilde{\mathbf{N}}(\mathbf{u}(\mathbf{x}, t))]_{\Lambda} = 0, \quad \text{for } \mathbf{x} \in \Lambda_1 \Lambda_2, \quad (18)$$

where $\tilde{\mathbf{N}}$ means viscoelastic operator as the analogy of surface tension from theory of elasticity:

$$\tilde{\mathbf{P}}(\cdot) = \mu(\tilde{\nabla} + \frac{2\lambda}{\lambda + 2\mu} \mathbf{1} \operatorname{div})(\cdot) \mathbf{n}, \quad (19)$$

Taking constitutive law of cracks (8) and applying functional way of description with δ of Dirac's type, the final general differential equation of viscoelastic RC cracked plate in plane stress, appropriate boundary, compatibility and initial conditions respectively, can be written as follows:

$$[\psi_1 * \nabla^2 + \frac{1}{2} (\psi_1 + \psi_2) * \operatorname{grad} \operatorname{div}] \mathbf{u}(\mathbf{x}, t) + \mathbf{1} * \mathbf{b}(\mathbf{x}, t) - \operatorname{div} \mathbf{F} = \quad (20)$$

$$= -\tilde{\mathbf{N}}(\mathbf{r}(\mathbf{x}, t) \delta_{\Lambda}) + [\mathbf{1} * \tilde{\mathbf{p}}(\mathbf{x}, t) - \tilde{\mathbf{N}}(\mathbf{u}(\mathbf{x}, t))] \delta_{\partial\Omega_1} + \tilde{\mathbf{N}}[(\tilde{\mathbf{u}}(\mathbf{x}, t) - \mathbf{u}(\mathbf{x}, t))] \delta_{\partial\Omega_2}.$$

Here the constitutive law of crack opening is expanded as a rule valid additionally in time. Note, that the final solution (20) is similar to elastic solution (10),



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difference occurs only for relaxation function with λ and μ as a time depended function.

The solution of Eq. (20) is possible with help of elastic solution as a first approximation of viscoelastic solution. It denotes the solution of "associated" elastic problem $\mathbf{u}(\mathbf{x}, t)$ from Eq. (12) can be used to convolutions' solution of viscoelastic static problem of RC cracked plain stress as follows:

$$\mathbf{u}(\mathbf{x}, t) = \int_0^t \frac{\partial \mathbf{u}(\mathbf{x}, \tau)}{\partial \tau} \varphi(t - \tau) d\tau, \quad (21)$$

where φ is the function with the combination of relaxation and creep functions.

4. MODELING BY BOUNDARY ELEMENT METHOD

Deformation behavior depends on the history of the loading as well as nonlinearity of material properties. Hence, equations and definitions of the boundary element method in rate form according to the Brebbia [4] formulations were assumed. According to small strains' theory, total strain rate for inelastic problem can be divided into an elastic and inelastic part of total strain rate tensor respectively. Herein, the inelastic strains mean any kinds of strain field that can be considered as initial strains, that is, plastic or viscoplastic strain rate, creep strain rate, thermal strain rate and strain rate due to other causes. So now, we can write (see also [3]) the equations of considered problem with non-linear BEM formulations for fictitious traction vector \mathbf{p} and body forces \mathbf{b} , finally leading to initial stresses σ^p :

$$\dot{\mathbf{H}}\dot{\mathbf{u}} - \dot{\mathbf{A}}\dot{\mathbf{p}} = \mathbf{B}\sigma^p + \mathbf{F}\dot{\mathbf{x}} + \dot{\mathbf{Q}}(\mathbf{x}), \quad (22)$$

where: \mathbf{u} - displacement vector,
 \mathbf{X} - vector of unknown edge traction,
 \mathbf{p} - vector of fictitious traction,
 σ^p - vector of initial stresses,
 \mathbf{H}, \mathbf{A} - the same matrices as for elastic analysis,
 \mathbf{B} - matrix due to the inelastic stress integral,
 \mathbf{F} - matrix refers to the fundamental function cause by forcing traction with vector \mathbf{X} , that is, modeling density of crack opening for plane stress,
 \mathbf{Q} - matrix of bond, bond-slip relations and other displacements due to aggregate interlock and dowel action of reinforcement in the crack, related to displacement \mathbf{u} .



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5. THE INCREMENTAL COMPUTATIONS

The results' correctness depends on the choice of right type boundary or finite elements respectively and a careful discretization of the structure. The influence of the above on the problem to be studied cannot be neglected. The appropriate simulation of the load-carrying behavior of RC structure is more important than the accuracy of the numerical calculations. The question what kind of numerical methods, boundary or finite elements should be preferably chosen cannot be answered satisfactorily. Equations (22) must be solved numerically with iterative and incremental techniques. Iteration results due the fact that the right sides of equations depend directly on searching functions. In addition searching functions depend indirectly on physical law. The incremental computation is caused by rate form of Eq. (22). For iteration and incremental computations the modified Newton-Raphson method was applied.

For the computation of plate the behavior of concrete is taken as well known stress-strain Madrid parabola. The stress-strain relation of steel bars was taken as a well-known elasto-plastic relation from uniaxial tests.

The creep of concrete was taken from the Bažant's [5] rheological model. This model is most suitable for concrete structures because the parameters can be calculated only from the concrete composition (for basic and drying creep as well as shrinkage). The creep function of Bažant is shown below (f_c' means the 28 days compressive strength of concrete):

$$J(t, \tau) = \frac{1}{E_o(f_c')} [1 + \varphi_1(f_c')(\tau^{-m(f_c')} + \alpha)(t - \tau)^{n(f_c')}] \quad (23)$$

After cracking of concrete, the tensile forces in the cracked area are transmitted by bond to the reinforcement consisting of steel bars. Along the segments of broken adhesion the steel bar co-operates with the concrete through the tangential stresses distributed on the perimeter of the bar. The slip is defined as a relative displacement between reinforcement bars and surrounding concrete. The increment of tensile stresses in the steel bar was approximated by the third-degree curve. Hence, the tangential stresses and bond-slip relationships, as representation of the stiffness of the bond has been found to be in the agreement to tests of Dörr [6], (that is, the second-degree distribution along the segment l_f , where l_f means distance between cracks).

The time-dependence of bond in the loaded state exhibits a similar behavior as concrete in compression (see [7]). The presupposition similar to the linear creep theory of concrete in compression is used for bond creep with bond creep coefficient φ_b . Naturally, in accordance to τ - Δ relationship of Dörr [6], bond creep

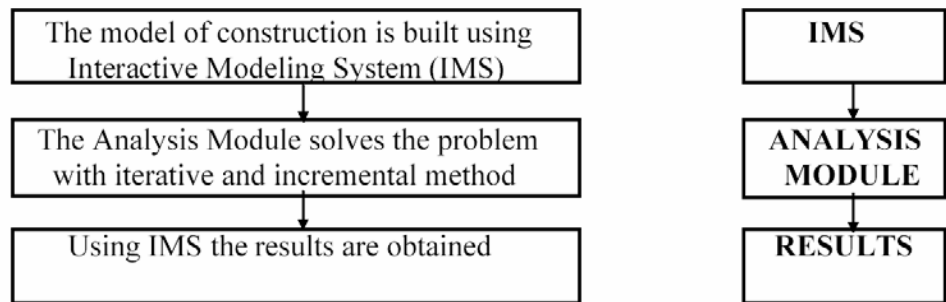


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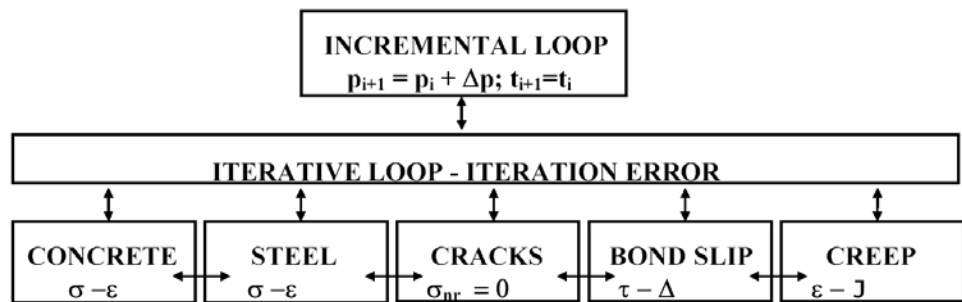
cannot be described with linear theory. The model of Rotasy [7] was applied to describe the creep of the bond in the cracked concrete.

The development of "rotating cracks" is considered as single cracks treated as the boundary element where the direction of the crack has to be assumed in accordance with the previous step of the main direction of the tensile stresses.

A program for Boundary Element Analysis named PLATE for planar structure was designed. The route through a PLATE analysis includes following procedures is shown below:



The way of applying iterative and incremental technique in the Analysis Module is shown below in the block diagram:



6. NUMERICAL EXAMPLE

The results of simply supported square panel WT3, tested by Leonhardt and Walther [8], were taken to check BEM solution for plain stresses. The panel was reinforced horizontally in different way for bottom and top part. The bottom zone (Ra₁) had 2φ8 mm bars each 6 cm fixed in 4 rows, the top zone and vertical bars (Ra₂) were 2φ5 mm each 26 cm.



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In Figure 2 the experiment received load-midspan deflection relation of the panel as compared to the results of BEM and other authors ([9] and [10]) FEM numerical calculations.

Figure 3 shows the dependence of the time and loading levels on the crack width a_f for panel WT3.

7. CONCLUSIONS

The numerical results obtained for the problems of panels indicated that the presented methods are capable to predict sufficiently and satisfactorily response of reinforced concrete planar structure.

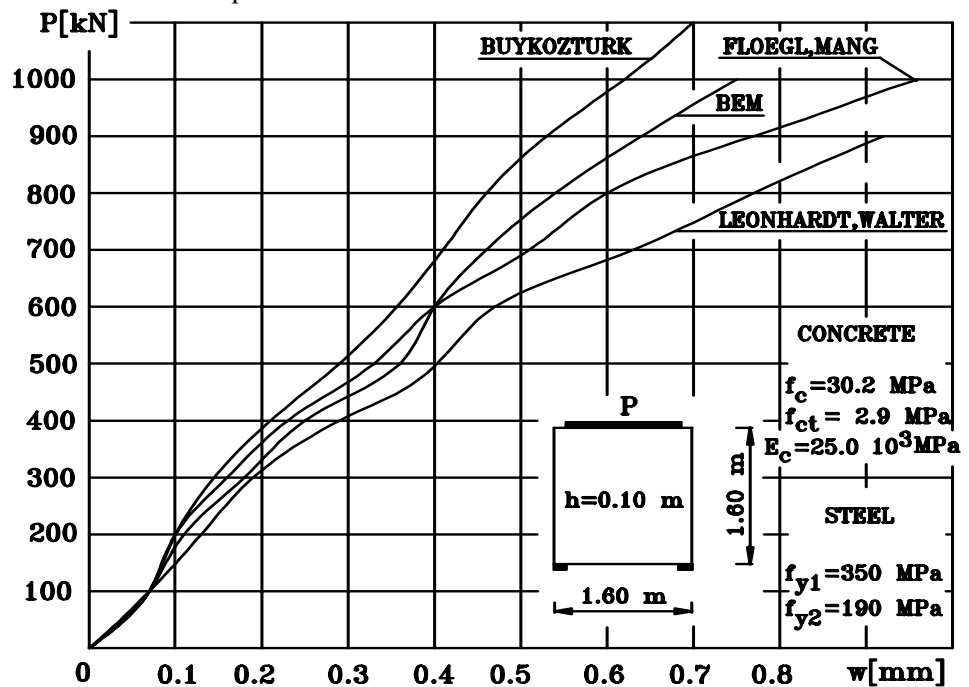


Figure 2. Comparison of calculated load-midspan deflection relations with the test result of panel WT3 [8]

For the computation of planar structure the behavior of concrete should be considered in the biaxial domain. The concrete properties are influenced by many different factors. Therefore the biaxial stress-strain relation and the failure criterion of concrete depend on the results of the tests that are performed to obtain these relations. The biaxial tests of Kupfer [11] for short time loading and proportionally increasing load proved to be the most reliable. Different authors have used these



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test results to develop analytical formulation of the failure and deformation behavior of the concrete. Link [12] developed an incremental formulation for the tangent stiffness of the concrete on the basis of Kupfer's tests. The stresses are normalized in terms of the uniaxial cylinder strength; therefore the formulation can be used for different grades of concrete. The failure criterion cannot be used as plasticity condition, because it describes a boundary for the maximum stresses and does not allow any statements about the plastic deformations.

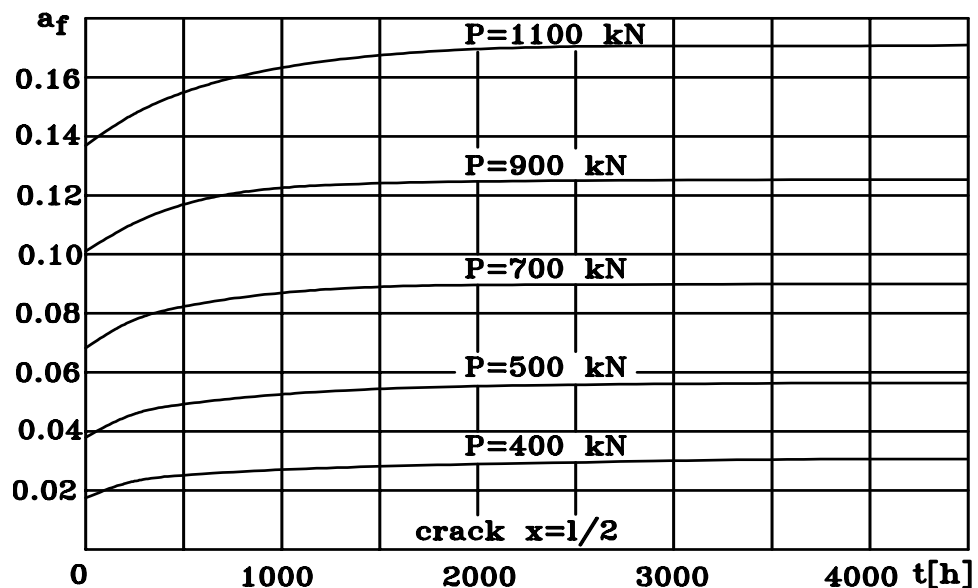


Figure 3. The dependence of the time and loading levels on the crack width a_f for $x=l/2$

The problem of crack propagation can be solved by evaluating Rice's integral along the contour of the crack top zones, where for different specimens the stress intensity factor could be found (see [13]).

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The numerical model for reinforced concrete structures and analysis using finite elements method

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Summary

The purpose of this paper is to review model for finite element techniques for non-linear crack analysis of reinforced concrete beams and slabs. The non-linear behavior of concrete and steel were described. Some calculations of "self-stress" for concrete and reinforced concrete beam was made. Current computational aspects are discussed. Several remarks for future studies are also given.

The numerical model of the concrete and reinforced concrete was described. The paper shows the results of calculations on a reinforced concrete plane stress panel with cracks. The non-linear, numerical model of calculations of reinforced concrete was assumed. Using finite elements method some calculations were made. The results of calculations like displacements, stresses and cracking are shown on diagrams. They were compared with experimental results and other finding. Some conclusions about the described model and results of calculation are shown.

1. NON-LINEAR BEHAVIOR OF CONCRETE AND STEEL

Reinforced concrete structures exhibit very complicated behavior different from steel structures. The structural system is composed of different materials, such as cement, steel bars, aggregate etc. Moreover each material shows various physical phenomena.

The non-linear behavior of entire structures can be considered to be accumulated from cracking of concrete, non-linear material properties of concrete under compression, time-dependent deformations due to creep and shrinkage, bond behavior, yielding and strain hardening of steel essentially. Progressive cracking of concrete is surely the most important component of the non-linear response of reinforced concrete structures in normal service state.

The experiments [3,4] show that under cyclic loads concrete and reinforced concrete structures response like linear-elastic materials. The linear-elastic behavior of concrete elements under cyclic loads is the base to assume that total deformations is the sum of elastic, residual, plastic and creep deformations as shown in Figure 1.



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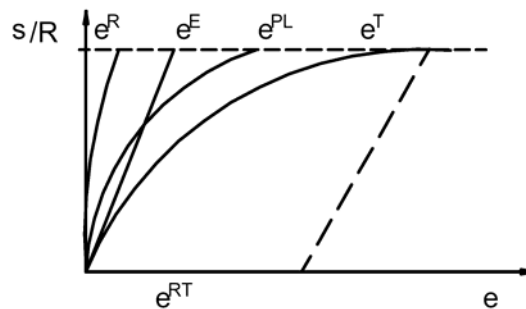


Figure 1. Stress-strain relationship for concrete

The total residual ϵ^{RT} deformation is the sum:

$$\epsilon^{RT} = \epsilon^R + \epsilon^{PL} + \epsilon^{R\phi} \quad (1)$$

where:

ϵ^{RT} - residual deformation,

ϵ^{PL} - plastic deformation,

$\epsilon^{R\phi}$ - creep deformation.

The creep deformation described using Rush [2] theory. It was assumed that residual creep deformation is 80% of total. The non-linear residual ϵ^R deformation, because of plane cross section, gives "self-stress" σ^R inside the element:

$$\int \sigma^R dF = 0, \quad (2)$$

$$\int \sigma^R z dF = 0 .$$

Using the equilibrium of forces and moments in the cross section (Eq. 2), the constants A, B and "self-stress" was calculated:

$$\sigma^R = E_b \epsilon^{ED} = E_b (A + Bz - \epsilon^R) \quad (3)$$

We can write the total stress as a sum of self-stress σ^R and easy to obtain linear elastic stress σ^E :

$$\sigma^{RT} = \sigma^R + \sigma^E \quad (4)$$



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The steel reinforcement is stressed only in one direction. The material is represented by a bilinear model which may either be elastic - perfectly plastic or strain-hardening as shown in Figure 2.

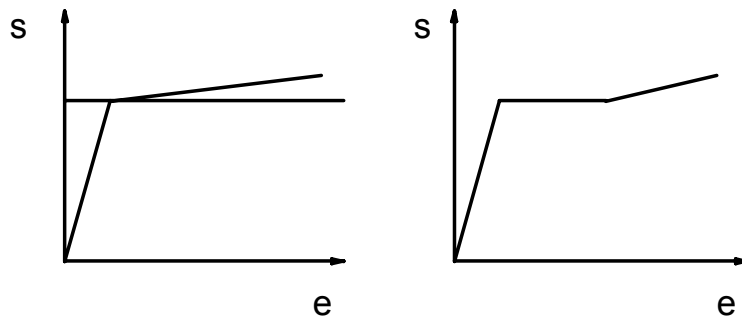


Figure 2. Stress strain model for steel

2. SELF-STRESS IN CONCRETE ELEMENT

The self-stress was calculated for bending concrete beam Figure 3.

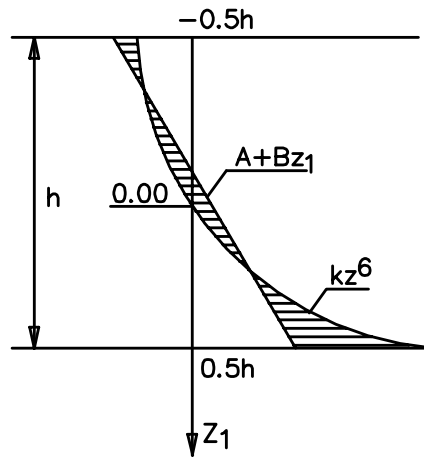


Figure 3. Self-stress in concrete beam

The course of residual strain was assumed as known and described:

$$\varepsilon^R = \begin{cases} -kz^6 & \text{for } z < 0 \\ kz^6 & \text{for } z > 0 \end{cases}, \quad (5)$$



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where: $z = \sigma / R_{bk}$ for $\sigma < 0$,
 $z = \sigma / R_{bzk}$ for $\sigma > 0$.

Using the equilibrium of forces and moments (Eq. 2) the constants A, B was calculated:

$$\int \sigma^R dF = \int [A + Bz_1 - \varepsilon^R(z_1)] b dz_1 = 0 \quad (6)$$

$$\int \sigma^R z_1 dF = \int [A + Bz_1 - \varepsilon^R(z_1)] bz_1 dz_1 = 0$$

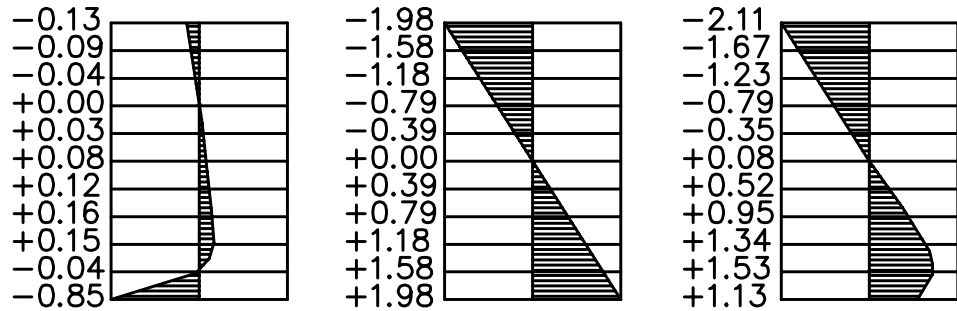


Figure 4. Results of calculation the self-stress for concrete B-15 ($f_c=15.0$ Mpa, $f_{ct}=1.40$ Mpa, $E_c=23.1 \cdot 10^3$ MPa)

$$A = 1 / F_b \int \varepsilon^R(z_1) b dz_1 ; F_b = bh \quad (7)$$

$$B = 1 / I_b \int \varepsilon^R(z_1) z_1 b dz_1 ; I_b = bh^3/12 .$$

Some calculations for different concrete was done The results of calculations of self-stress, elastic stress and total stress are shown on Figure 4.

3. SELF-STRESS IN REINFORCEMENT CONCRETE ELEMENT WITH CRACK

The self-stress in reinforcement concrete bending beam was calculate in the same way as in concrete element Figure 5.



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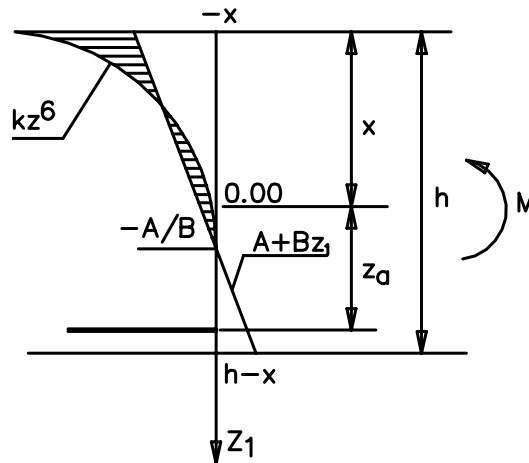


Figure 5. Self-stress in reinforced concrete beam with crack

The residual strain-stress relationship described the same equals (Eq. 4) like for concrete element. Using condition (2) the constants A, B was calculated:

$$E_b \int (A + Bz_1 - \varepsilon^R(z_1)) dF + E_a F_a (A + Bz_a - \varepsilon_a^R) = 0 , \quad (8)$$

$$E_b \int (A + Bz_1 - \varepsilon^R(z_1)) z_1 b dz_1 + E_a F_a (A + Bz_a - \varepsilon_a^R) z_a = 0 ,$$

where: ε_a^R - residual strain for steel.

Some numerical calculations for different causes were done. The results of these calculations are shown in Figure 6.

4. NON-LINEAR ANALYSIS USING F.E.M.

The Finite Elements Method was used to calculate the non-linear effects in reinforced concrete elements. The rectangular elements were used with stiffness matrix calculated by Rockey [5]. The first step is taken as the linear solution using known relationship of F.E.M. [6]:

$$[K]\{\delta\} - \{R\} = 0 , \quad (9)$$

where: [K] - stiffness matrix,

{ δ } - displacement,

{R} - external forces.



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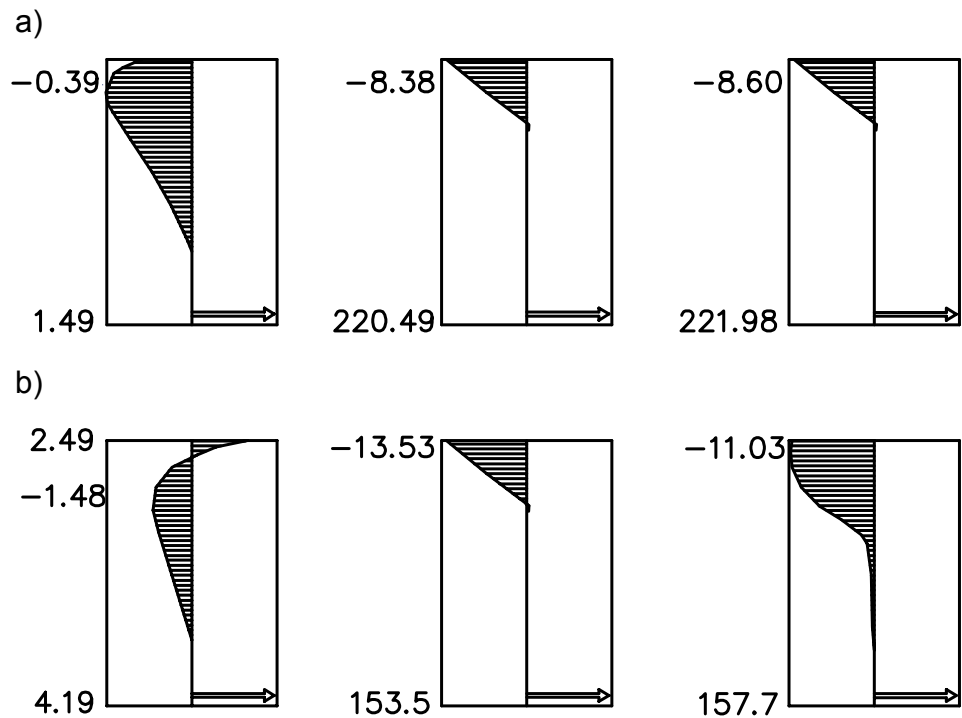


Figure 6. Stresses in cracked concrete element, concrete B-15 (see Figure4), steel $f_y = 220$ MPa. a) $\mu = 0.5\%$, $M = 29$ kNm, b) $\mu = 2\%$, $M = 75$ kNm

In Eq. 9 the linear behaviour of materials was assumed:

$$\{\sigma\} = [D](\{\varepsilon\}) - \{\varepsilon_0\} + \{\sigma_0\} \quad , \quad (10)$$

$$F(\{\sigma\}, \{\varepsilon\}) = 0 \quad . \quad (11)$$

The non-linear effects were calculated using iteration and changing of the external forces $\{R\}$. The external forces were calculated on the basis of initial strain and initial stress which described the cracks or non-linear material behavior. This method needs no necessity of changing of the stiffness matrix.

The reinforcement was described as linear elements which are added to the stiffness matrix. Before the cracks appear the concrete strain and the steel strain are equal. After cracking the steel elements take over the stresses from the concrete. This stresses are added to the external forces as initial stresses.



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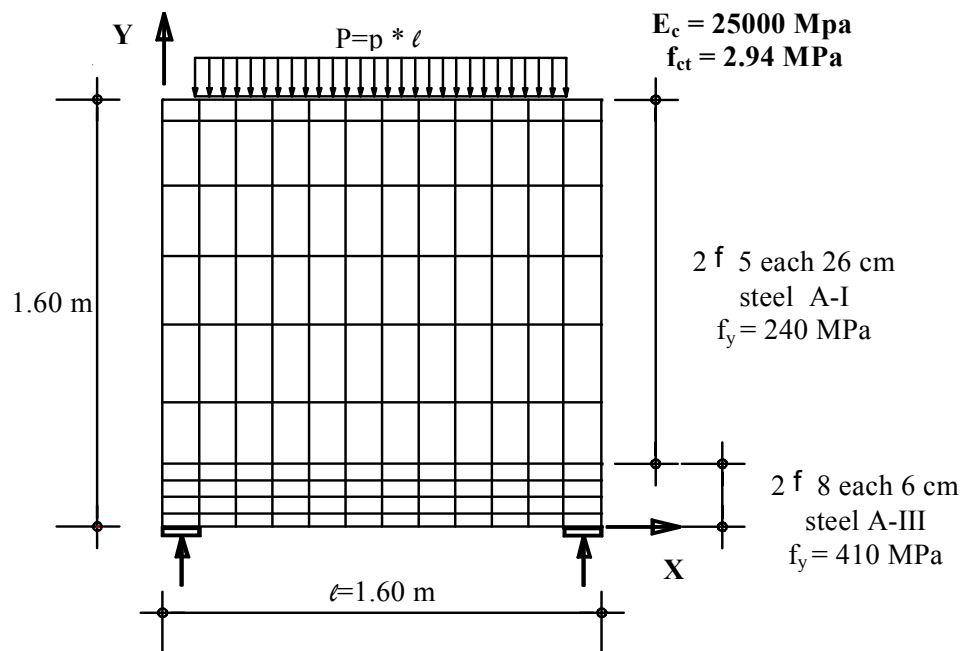


Figure 7. The schema of calculated RC panel

5. ANALYSIS OF RC DISK WHICH WAS TESTED BY LEONHARDT AND WALTHER [7]

The purpose method was examined on calculations of the RC panel (Figure 7) tested by Leonhardt and Walther [7]. The same panel was calculated by Floegl [8], Buyukozturk [9] and Lewiński [10]. So, there is the material to compare the results of analysis.

The results of numerical calculations are shown on Figures 8-11. Figure 8 shows the propagation of cracks under different loading. The first cracks was observed for $P=400$ kN. In each level of the loading the number of cracks and the width of the cracks vary as shows the figure.

In Figure 9, the relationship between loading and displacement of the panel as compared with other finding is shown. The good compatibility was notice.

Figure 10 and 11 show the comparison of stresses in steel bars calculated by authors with experimental findings and other calculations.



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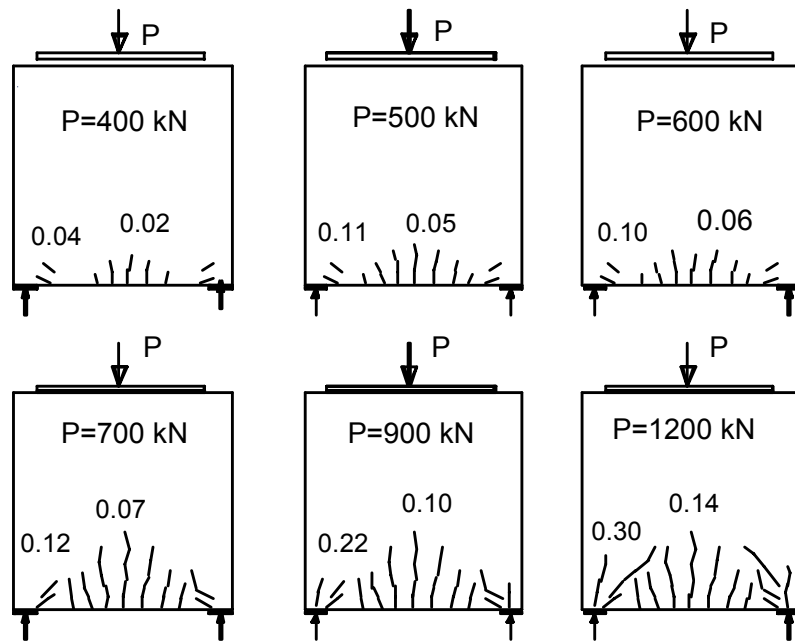


Figure 8. The propagation of the cracks under different level of loading

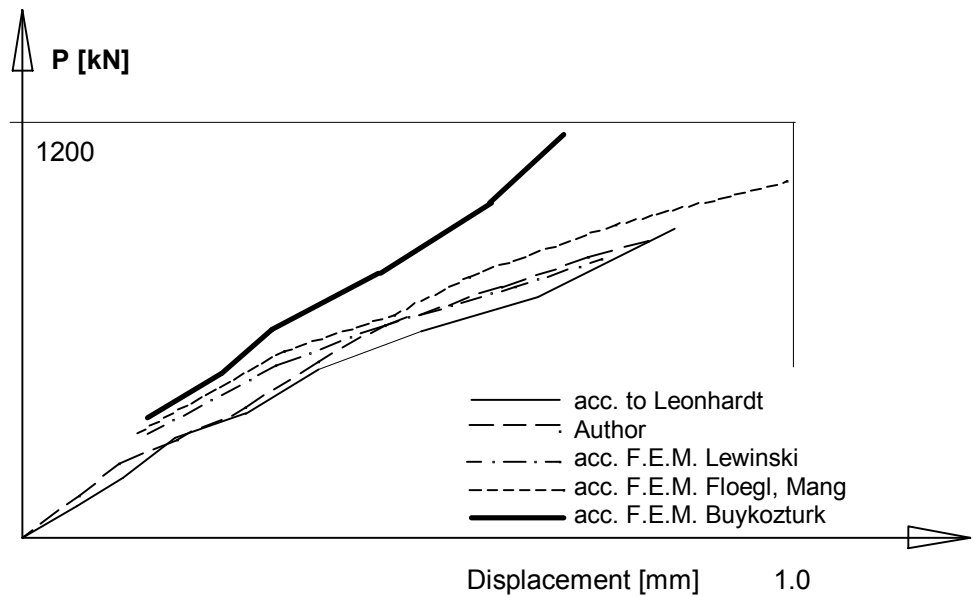


Figure 9. The relationship between loading and displacement



The numerical model for reinforced concrete structures and analysis using finite elements method

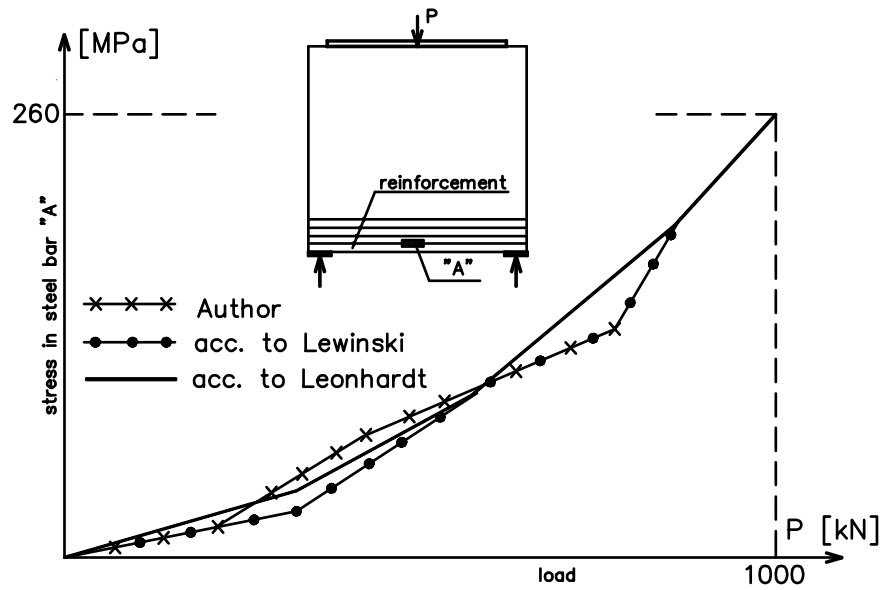


Figure 10. Comparison of the stresses in reinforcement of the panel with other finding

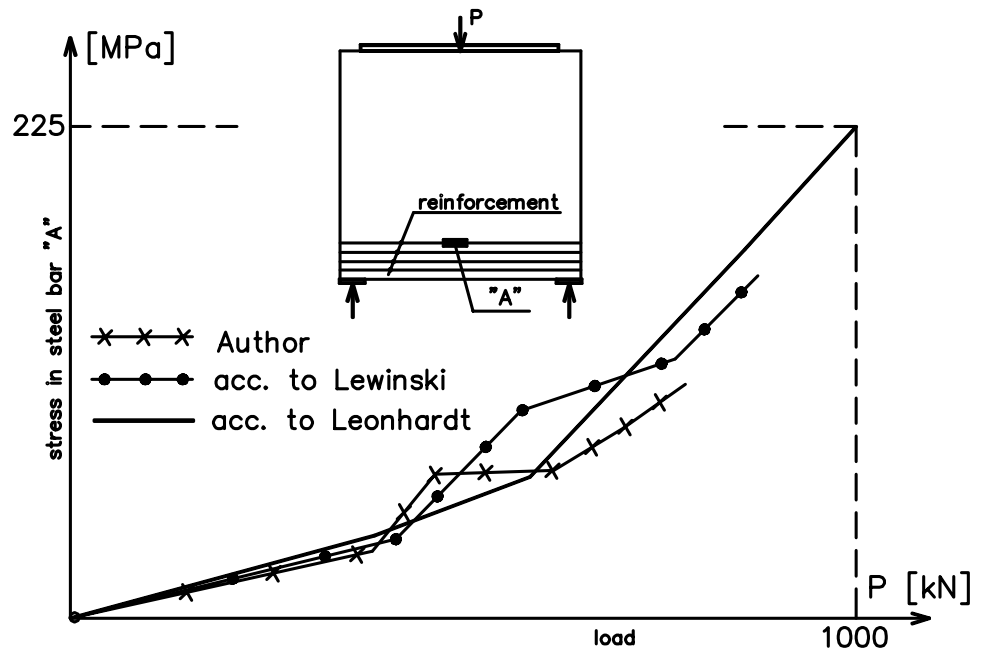


Figure 11. Comparison of the stresses in reinforcement of the panel with other finding



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4. CONCLUSIONS

The mathematical idealization of cracked reinforced concrete structures is very difficult. The presented model of non-linear behavior of reinforced concrete can be used to numerical analysis with the finite element method. It may give relatively quick solution because of no necessity to change the stiffness matrix and solve equations several times.

Descriptions of all non-linear behavior of materials, cracks, self-stresses treated as initial strains gives only different right sides of standard equals. Therefore the iteration of cracks and loads do not change the stiffness matrix. All that may preference finite element method to numerical analysis.

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Numerical modeling for homogenization of masonry structures

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Summary

In the present study, equivalent elastic properties, strength envelope and different failure patterns of masonry material are homogenized by numerically simulating responses of a representative element under different stress conditions.

The representative volume element provides a valuable dividing boundary between the discrete model and the continuum model. This paper presented a computational homogenization technique of masonry.

KEYWORDS: masonry structures, homogenization, computational simulations.

1. INTRODUCTION

In recent years growing attention has been paid by researches in structural mechanics to masonry structures with the intent to provide theoretical and numerical tools for better understanding the complex mechanical behavior of such structures. The complex mechanical behavior of masonry structures depends strongly on the composite nature of masonry material.

Masonry is constituted by blocks of natural or artificial material jointed by dry or mortar joints; the latter are the weakness – areas of such a composite material and notably affect the overall response of the assembly with a number of kinematical modes at joints such as sliding, opening – closing and dilatancy.

Generally, two different methods have been developed to perform linear and non-linear analyses of masonry structures (Szołomicki 1997). The macro-modeling approach intentionally makes no distinction between units and joints but smears the effect of joints presence through the formulation of a fictitious homogeneous and continuous material equivalent to the actual one which is discrete and composite. The alternative micro-modeling approach analyses the masonry material as a discontinuous assembly of blocks, connected each other by joints at their actual position, being simulated by appropriate constitutive models of interface (Zucchini 2002).

Another direction, presented in this paper, is a method which resorts to homogenization technique. The homogenization method that would permit to



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establish constitutive relations in terms of averaged stresses and strains from the geometry and constitutive relations of the individual components would represent a major step forward in masonry modeling.

In this paper, a typical unit of masonry is selected to serve as a representative volume element. Both the bricks and mortar joints are idealized as isotropic material having their own properties such as stiffness, strength and damage characteristics. In the homogenization process, failure of the individual material in the unit is divided into three modes: tensile failure of mortar joint, shear failure of mortar joint and brick, and compressive failure of brick. A fracturing law is associated with the tensile failure of the mortar joint, while the shear failure accounts for the variations of shear strength as a function of normal stress.

2. FORMULATION OF REPRESENTATIVE VOLUME ELEMENT

The homogenized constitutive law is determined by studying the behaviour of the representative volume element (RVE) which is the cell of periodicity in the case of periodic media (Galvanetto 1997).

Such a RVE plays in the mechanics of composite material the same role as the classical elementary volume of continuum mechanics; therefore, in general, a homogenized approach is successful if the size of the cell is small compared with that of the structure.

The representative volume element of masonry should include all the participant materials, constitute the entire structure by periodic and continuous distribution, and be minimum unit satisfying the first two conditions. Under conditions of an imposed macroscopically homogeneous stress or deformation field on the representative volume element, the average stress and strain fields are respectively:

$$\bar{\sigma}_{ij} = \frac{1}{V} \int_V \sigma_{ij} dV \quad (1)$$

$$\bar{\varepsilon}_{ij} = \frac{1}{V} \int_V \varepsilon_{ij} dV \quad (2)$$

where V is the volume of the representative volume element.

Based on the constitutive relations of the brick and the mortar materials, the equivalent stress-strain relations of the RVE are homogenised by applying various compatible displacement conditions on the RVE surfaces (Luciano 1997).



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3. DAMAGE MODEL FOR MORTAR JOINT AND BRICK

3.1. Damage of mortar joint

Most of the non-linear deformation in brick masonry, until failure, occurs only in the joints. Establishing a reliable material model for the mortar joint is very important for analysing the masonry load-bearing and deformation capacity.

The degradation of tensile strength in mortar joint can be expressed by an exponential approximation as:

$$\sigma = E_n \varepsilon_n, \quad \varepsilon_n \leq \varepsilon_0 \quad (3)$$

$$\sigma = \sigma_0 e^{-\alpha_n (\varepsilon_n - \varepsilon_{n0}) / \varepsilon_{n0}}, \quad \varepsilon_n \geq \varepsilon_{n0} \quad (4)$$

where: α_n is a material parameter, ε_{n0} – threshold strain that initiates tensile fracturing of the material, σ_0 is the elastic limit stress of the mortar joint.

The exponential decay leads to the following total fracture release energy:

$$G_f^I = \int_0^{+\infty} \sigma d\varepsilon_n. \quad (5)$$

The differential of fracture energy is obtained as:

$$dG^I = \frac{1}{2} \sigma d\varepsilon_n - \frac{1}{2} d\sigma \varepsilon_n. \quad (6)$$

Thus tensile damage can be defined as the ratio of released fracture energy to the total fracture release energy as:

$$D^I = \frac{\int_0^{\varepsilon_n} dG^I}{G_f^I}. \quad (7)$$

In the compression – shear region of mortar joints is reasonable to use the Mohr – Coulomb criterion, which can be expressed as:

$$F(\sigma, \tau) = |\tau| + \mu\sigma - c(G^{II}) = 0, \quad (8)$$

where: μ and c are frictional coefficient and cohesion, respectively; σ is a compressive stress; G^{II} is the dissipated plastic work that results from shear failure of mortar joint.



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The incremental strain vector of mortar can be divided into elastic and plastic parts:

$$d\varepsilon = d\varepsilon^e + d\varepsilon^p \quad (9)$$

To avoid excessive plastic dilatancy, a non-associated flow rule is proposed. It is expressed as:

$$Q(\sigma, \tau) = \eta(\tau) + \mu\sigma \quad (10)$$

where: η is parameter that scales the dilatancy.

The direction of plastic relative displacements is governed by the flow rule as:

$$d\varepsilon^p = d\lambda \frac{\partial Q}{\partial \sigma} \quad (11)$$

where $d\lambda$ is the plastic multiplier.

According to the traditional plastic flow rule the complete elasto-plastic incremental stress-strain relationship is presented as:

$$d\sigma = E^{ep} d\varepsilon \quad (12)$$

where:

$$E^{ep} = E^e - \frac{E^e \frac{\partial Q}{\partial \sigma} \frac{\partial F}{\partial \sigma^T} E^e}{-A + \frac{\partial F}{\partial \sigma^T} E^e \frac{\partial Q}{\partial \sigma}} \quad (13)$$

and A is a hardening parameter.

The plastic work done by the shear stress τ depends on the lateral compression σ . When the combination of (τ, σ) on the strength surface is expressed as (8), the incremental plastic work is the following:

$$dG^{II} = (|\tau| + \mu\sigma) d\varepsilon_t^p = \left\{ |\tau| + \mu\sigma \quad 0 \right\} \left\{ d\varepsilon_t^p \quad 0 \right\}^T \quad (14)$$

The damage value at compressive-shear region can be expressed as:

$$D^{II} = \frac{G^{II}}{G_f^{II}} \quad (15)$$

where:



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$$G^{\text{II}} = \int_0^{+\infty} (|\tau| + \mu\sigma) d\varepsilon_t^p \quad (16)$$

The total dissipated energy due to friction can be calculated by the relationship of the shear stress and strain with no lateral compression on the surfaces. The relationship can be written as:

$$\tau = E_t \varepsilon_t \text{ when } \varepsilon_t \leq \varepsilon_{t0} \quad (17)$$

$$\tau = E_t \varepsilon_{t0} e^{-\alpha_t(\varepsilon_t - \varepsilon_{t0})/\varepsilon_{t0}} \text{ when } \varepsilon_t \geq \varepsilon_{t0} \quad (18)$$

where α_t is a material parameter and ε_{t0} is threshold shear strain of mortar joint without lateral compression. The cohesion c is expressed as:

$$c = \left(1 - \frac{G^{\text{I}}}{G_f^{\text{I}}}\right) \left(1 - \frac{G^{\text{II}}}{G_f^{\text{II}}}\right) c_0 = (1 - D^{\text{I}})(1 - D^{\text{II}}) c_0. \quad (19)$$

3.2. Damage of bricks

Damage of bricks is composed of compressive crushing and tensile splitting due to high compression. According to the isotropic damage theory, the secant constitutive tensor can be written as:

$$\Lambda_{ijkl}(D) = \Lambda_{0ijkl}(1 - D) \quad (20)$$

where: Λ_{0ijkl} is initial stiffness of material and D is a damage.

The damage scalar consists of two parts, namely D_t due to tensile damage and D_c due to compressive damage. It is evaluated by the combination of:

$$D = A_t D_t + A_c D_c, \quad \dot{D} > 0 \quad (21)$$

where: A_t and A_c are the balancing coefficient characterizing tension and compression, respectively. Damage scalars D_t and D_c corresponding respectively to damage measured in tension and compression can be expressed again in exponential approximation as:

$$D_t = 1 - e^{-\beta_t(\tilde{\varepsilon}^+ - \varepsilon_0^+)/\varepsilon_0^+} \quad (22)$$

$$D_c = 1 - e^{-\beta_c(\tilde{\varepsilon}^- - \varepsilon_0^-)/\varepsilon_0^-} \quad (23)$$



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where $\tilde{\varepsilon}^+$ and $\tilde{\varepsilon}^-$ are the equivalent tensile and compressive strains.

4. FINITE ELEMENT IMPLEMENTATION

The material model proposed above is used into finite – difference program. The finite element analysis is based on this decoupling of the macroscopic and microscopic displacement fields. In the present analysis, the material is considered in elastic state before the directional strain of the mortar joint reaches the threshold strain.

In post-failure state, the stress will decrease with increase of uniaxial strain. The material parameters of brick can also be determined by uniaxial tensile and compressive test data. The computational models of RVEs in the present numerical analysis are shown in Figure 1. The brick and mortar are discretized individually. The brick size is 250 x 65 x 120 mm and mortar thickness is 10 mm.

In numerical simulation, vertical and horizontal displacements are applied to the RVE surfaces. The advantage of using displacement boundaries can not only avoid incompatible deformation between RVEs, but also makes it possible to obtain a complete monotonic stress-strain curve through the homogenization process.

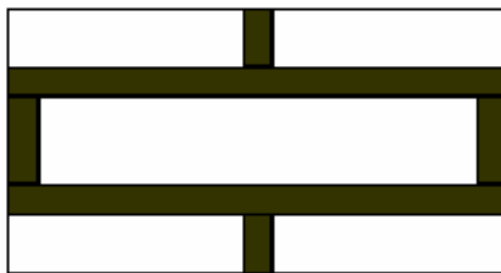


Figure 1. Representative volume element of masonry

5. NUMERICAL APPLICATION

The theory presented above has been used in the following example. The masonry panel schematically reported in Figure 2, subjected to pure shear loading is analyzed. This is characterized by the following geometrical parameters: $H = 3000$ mm, $B = 2000$ mm and material characteristics (Tables 1 and 2).



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In Figure 3, the damage and the minimum principal stress maps for the prescribed displacement $u = 5 \text{ mm}$ is plotted for analyzed panel. It can be noted that the failure mechanism is characterized by the formation, growth and propagation of inclined damage bands, as it typically occurs in structures subjected to horizontal forces. Damaging process in analysed example is concentrated in a single band.

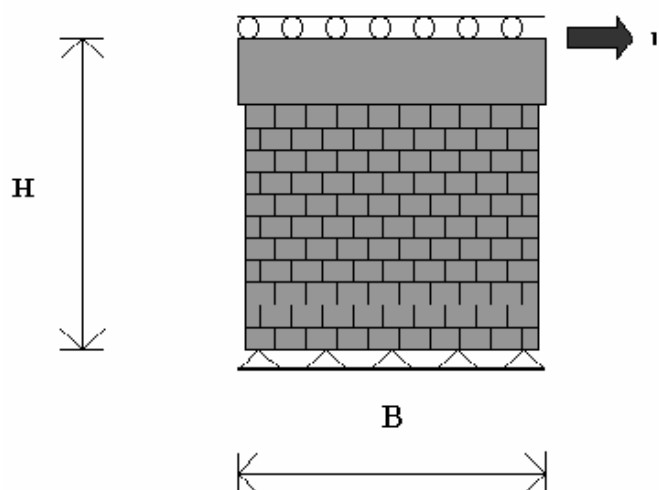


Figure 2. Analyzed masonry panel

Table 1. Material parameters for brick

Material	E MPa	G MPa	ν -	σ_t MPa	σ_c MPa	ϵ_0^+ -	ϵ_0^- -
Brick	12000	4700	0,15	2	50	0,0002	0,0004

Table 2. Material parameters for mortar

Material	E_n MPa	E_t MPa	σ_0 MPa	c_0 MPa	μ -	ϵ_{n0} -	ϵ_{t0} -
Mortar	2,0	1,0	0,9	1,1	0,8	0,0004	0,001

It can be noted that the mechanical response of wall subjected to shear loading is characterized by:

- an initial elastic response,
- a first step softening branch due to the damage propagation concentrated in place where the maximum tensile strains occur,
- a hardening phase during which the plastic evolution process becomes more significant than the damage one.



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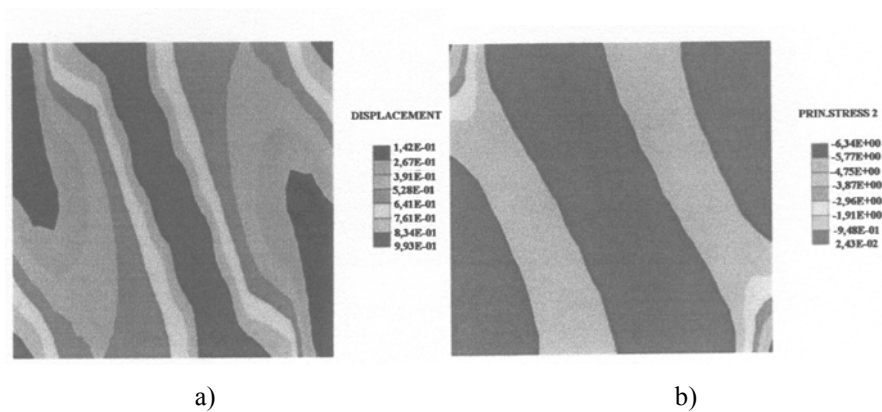


Figure 3. Numerical analysis of masonry panel: a) damage distribution and b) minimum principal stress distribution for $u = 5$ mm

6. CONCLUSION

In this paper the authors presented a numerical homogenization technique of masonry. The strength behavior and damage behavior have been obtained by numerical analysis of the representative volume element responding to various boundary conditions. They have been implemented into a continuum plastic damage model. Failure of masonry can be described into three types: tensile failure due to mortar tensile damage, shear failure of brick and mortar and compressive failure of brick. It should be noticed that the RVE employed in a general homogenization process should be smaller than the whole masonry structure. Otherwise the edge effect will affect the macro-material properties significantly, and in that case the discrete element method is preferred.

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About some important changes in applied structural optimization

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Summary

Structural optimization is special domain of employment researching many and how different problems in the field of forming structure. In times of early computer science and computational technology, when the access to “computing time” of the machine was strongly regulated (from the point of view of considerable costs) some optimization problems were very strongly simplified, so their solution could be possible without mathematical programming methods and therefore cheaper.

In times of stormy development of informatization and almost free-for-all personal computers as well as specialized software, complication of structural optimization modeling has grown considerably.

In this paper being short recapitulation of achievements made by Division of Computational Methods in Engineering Design, it refers to these earliest problems and to these very modern both dealing with applied structural optimization, what is the domain of interest of our team from over 25 years.

KEYWORDS: structural optimization, scalar optimization problem, genetic algorithm, vector optimization problem

1. INTRODUCTION

As member of the Team for Computational Methods in Engineering and Design, I have started dealing with the applied structural optimization in the end of 70-ties in XX century. In the beginning it was research concerning steel bar structures (trusses and frames) and industrial buildings (concrete beams, silos and tanks). All of these early problems mentioned above were formulated and then solved as scalar optimization questions.

Next we started researching with vector optimization problems (steel frames and trusses) and genetic algorithms (thanks to cooperation with Carlos Coello Coello and Gregorio Toscano-Pulido).

In this paper I'm trying to bring you closer how deep were the differences between these first and last problems (exactly in this year was my personal 25th anniversary of optimization research).



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2. SCALAR OPTIMIZATION PROBLEM

2.1. Simple example of the tank welded from steel

The first example of optimization I want to present (in this case example of scalar optimization, started and conducted in 1981) is a tank (the part of steel water tower), shown on the drawing below (Figure 1).

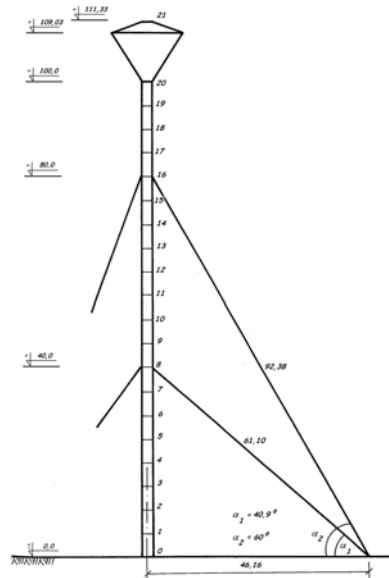


Figure 1. Steel water tower

For optimization the following lump of the tank, made from two cut off cones and one internal cylinder, has been chosen. Surface plan it in places of intersections circled wreaths stiffening. Described has resulted from capacity form highly, allocations and easy installment available methods (so called “easy” or “heavy” one).

2.2 Scalar optimization model

As criterion of optimization accept minimum of expenditure of material preliminary. Become setting up average thickness of covering above-mentioned question fetch for determination of condition of occurrence of minimum of lateral surface. Besides, it accepts following foundation and simplification:

- dimension section - they mirror middle surface,
- thickness of covering is constant (and average),



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- water fulfills only maximum bottom cone,
- we use only one design variable: corner of inclination of surface for vertical α in bottom cone (see Figure 2),
- capacity of useful tank totals 600 m^3 .

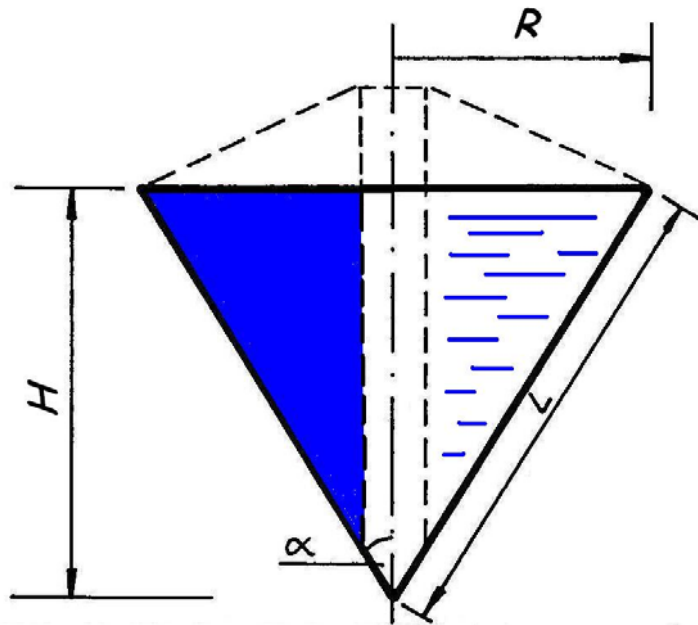


Figure 2. Steel tank – lump and design variable α

It takes into consideration, in the farthest consideration, following geometric dependences:

$$R = H \cdot \operatorname{tg}(\alpha), \quad (1)$$

$$L = H / \cos(\alpha), \quad (2)$$

Field of the lateral surface:

$$F = \pi \cdot R \cdot L = H \cdot \operatorname{tg}(\alpha) \cdot H / \cos(\alpha) = H^2 \cdot \operatorname{tg}(\alpha) / \cos(\alpha), \quad (3)$$

Capacity of the cone:

$$V = 600 = 1/3 \pi \cdot R^2 \cdot H = \dots = 1/3 \pi \cdot H^3 \cdot \operatorname{tg}^2(\alpha), \quad (4)$$

Basing on (4) in the function of the corner α , next H was indicated:

$$H = [1800 / \pi \cdot \operatorname{tg}^2(\alpha)]^{1/3}, \quad (5)$$

and it put for (1)



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$$F = A \cdot \sin^{-1}(\alpha) \cdot [\operatorname{tg}^2(\alpha)]^{1/3},$$

where $A = 1800 \cdot (\pi/1800)^{1/3} = \dots = 216,72$, (6)

Task of minimization solve existence of minimum of function alternate one researching $F(\alpha)$.

$$\min F(\alpha) \leftrightarrow F'(\alpha) = 0, \quad (7)$$

Solution illustrate on the drawing (see Figure 3). Next it verify „candidate for minimum” (α') calculating in this point value of second derivative function $F''(\alpha)$:

$$F''(\alpha') = \dots = 1,833 > 0 \quad (8)$$

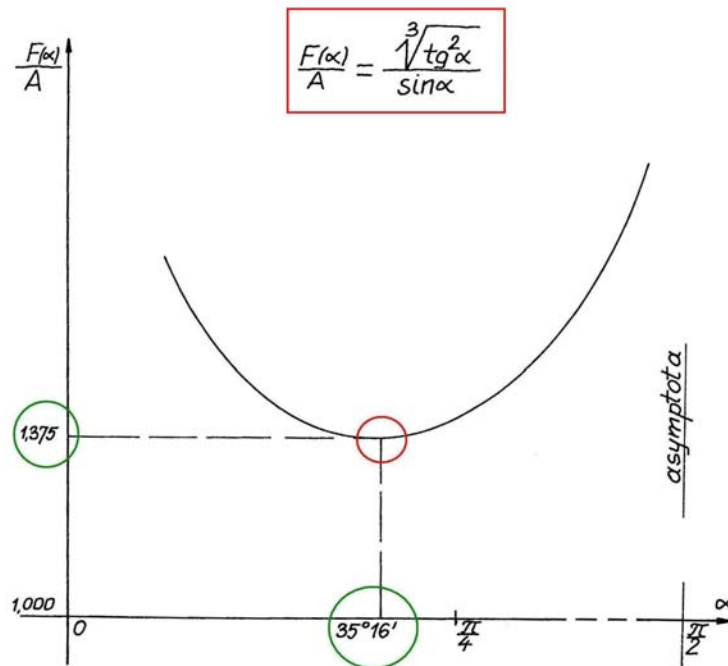


Figure 3. Solution of question of simple scalar optimization

2.3 Recapitulation and final conclusions

It exert in the first approximation, that conical tank has minimal field of lateral surface (but what behind it go, grant demanded criterion: minimum of material), when it lateral is drooping for vertical under corner creating $\alpha = 35^\circ 16''$.



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Having the corner of creating inclination α and foundations or simplifications mentioned on admission of preamble, remained dimension of the tank have been calculated from simple geometric dependences.

Fundamental geometric dimensions of the tank accepted for the farthest technical and executive design, it present on following drawing (Figure 4).

Assuring, as contact limit, the smallest surface of conical covering with aggressive environment (the water stored in this tank, so-called: industrial, with mineral small parts inclusive, about predefined temperature gone up with technological respects) we can prominently extend the constancy of maintenance of the building (water tower) in the best condition.

3. MICRO-GA AS AN EFFECTIVE SOLVER FOR MULTIOBJECTIVE OPTIMIZATION PROBLEMS

3.1. Genetic algorithms in multiobjective structural optimization

Genetic algorithms (GAs) have become very popular optimization techniques in structural optimization, but their use in multiobjective structural optimization has become less common. Additionally, only few researchers have emphasized the importance of efficiency when dealing with multiobjective optimization problems, despite the fact that its (potentially high) computational cost may become prohibitive in real-world applications.

In this paper, we present a GA with a very small population size and a reinitialization process (a micro-GA) [1] which is used for multiobjective optimization of trusses.

3.2. The micro-GA

This micro-GA approach elaborated by Toscano-Pulido [3,5] works as follows (Figure 5). It starts with a random population, it uses two memories: a replaceable (that will change during the evolutionary process) and a non-replaceable (that will not change) portion. Micro-GA uses 3 types of elitism.

The first is based on the notion that if we store the non-dominated vectors produced from each cycle of the micro-GA, we will not lose any valuable information obtained from the evolutionary process.

The second is based on the idea that if we replace the population memory by the nominal solutions (i.e., the best solutions found when nominal convergence is reached), it will gradually converge, since crossover and mutation will have a



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higher probability of reaching the true Pareto front of the problem over time. The third type of elitism is applied at certain intervals (defined by a parameter called “replacement cycle”). It takes a certain number of points from all the regions of the Pareto front generated so far and it uses them to fill the replaceable memory. Depending on the size of the replaceable memory, it chooses as many points from the Pareto front as necessary to guarantee a uniform distribution.

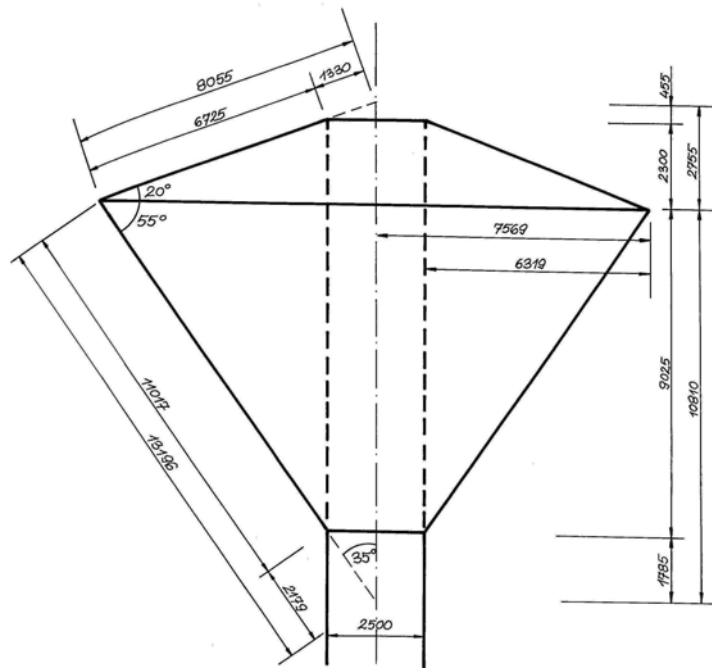


Figure 4. Conical tank accepted for the farthest technical design

This process allows us to use the best solutions generated so far as the starting point for the micro-GA, so that we can improve them (either by getting closer to the true Pareto front or by getting a better distribution along it). To keep diversity in the Pareto front, it uses an approach based on geographical location of individuals (in objective function space) similar to the adaptive grid proposed by Knowles & Corne [2]. This approach is used to decide which individuals will be stored in the external memory once it is full. Individuals in less populated regions of objective space will be preferred.

In previous work, our micro-GA has performed well (in terms of distribution along the Pareto front, and speed of convergence to the global Pareto front) with respect to other recent evolutionary multiobjective (vector) optimization approaches, while requiring a lower computational cost [3].



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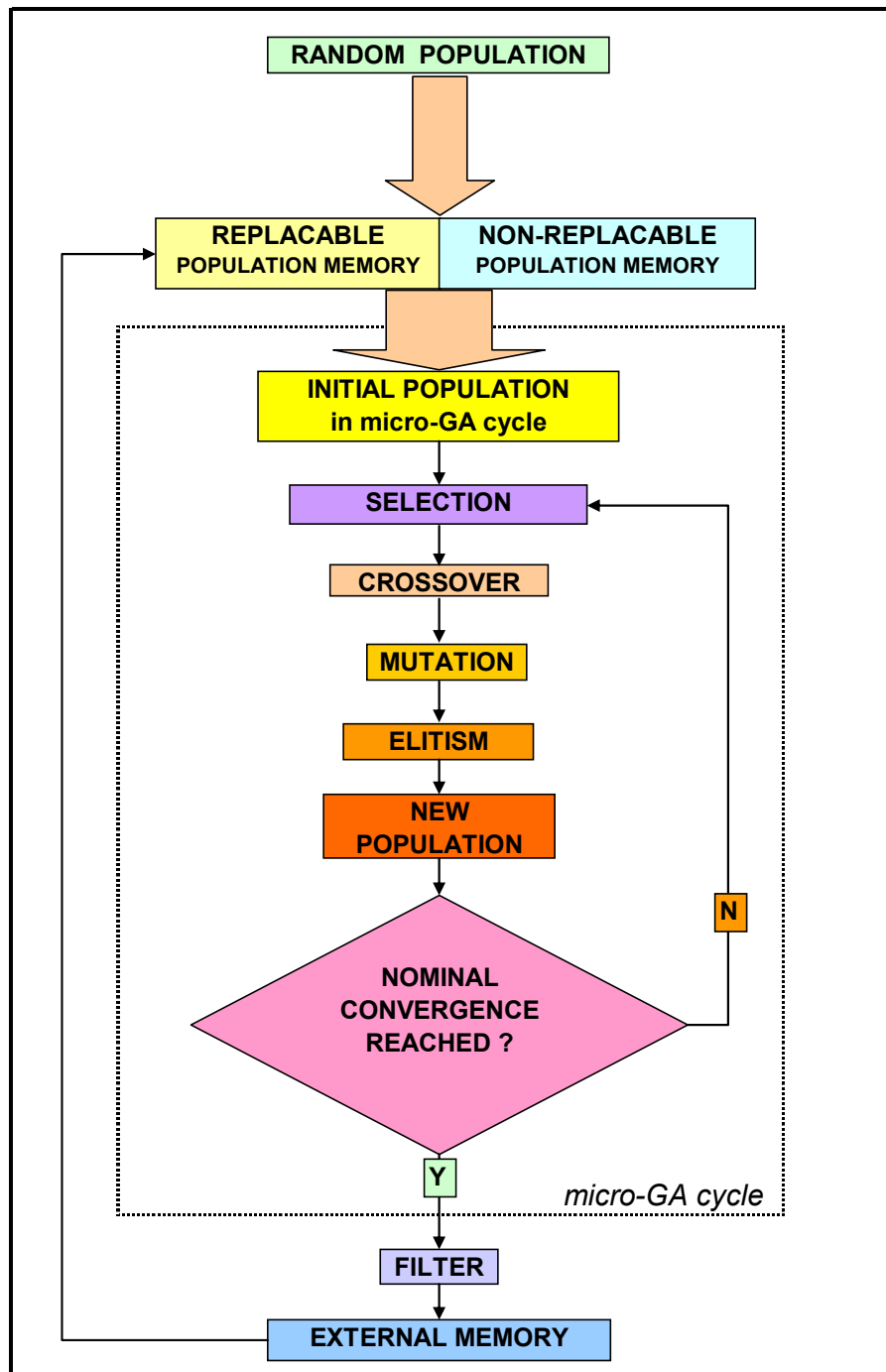


Figure 5 : Diagram of micro-GA



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3.3. Illustrative example

The 4-bar plane truss shown in below (Figure 6) is used to illustrate this approach. Two objectives were considered in this case: minimize volume and minimize its joint displacement δ . Four decision variables are considered (for details of this problem, see [4]).

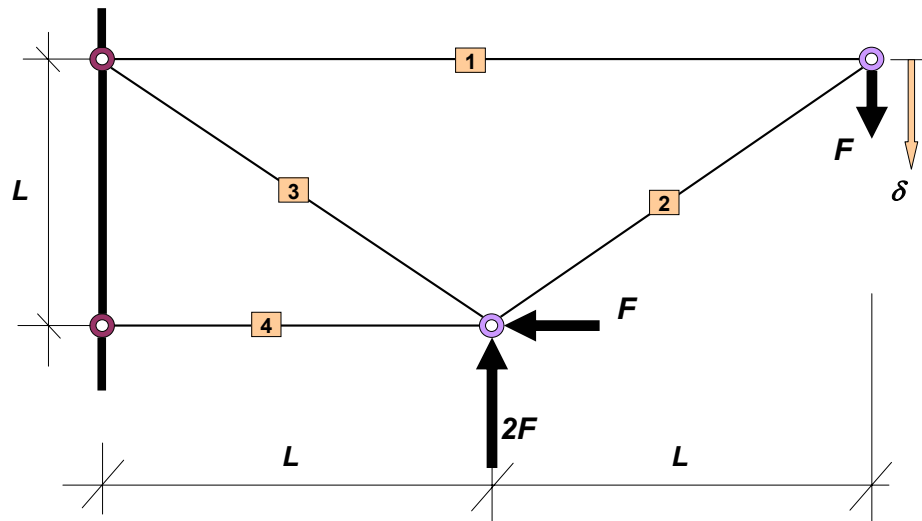


Figure 6 : Four-bar plane truss with one loading case

The Pareto front produced by micro-GA mentioned above, and its comparison against the global Pareto front (produced using an enumerative approach) is shown in the next figure (Figure 7).

3. CONCLUSIONS

The task of structural optimization is to support the constructor in searching for the best possible design alternatives of specific structures. The “best possible” or in the other words “optimal” structure means that structure which mostly corresponds to the designer’s objectives meeting of operational, manufacturing and application demands simultaneously.

Compared with the “trial and error”- method mostly used in engineering practice (and based on an individual, intuitive, empirical approach) the seeking of optimal solutions by applying MOP (mathematical optimization procedures) is much more efficient and reliable. Nowadays in the time of market economy also research has



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to be “market one”. In my opinion “to be market” is now the greatest challenge for applied optimization in Poland and everywhere [6].

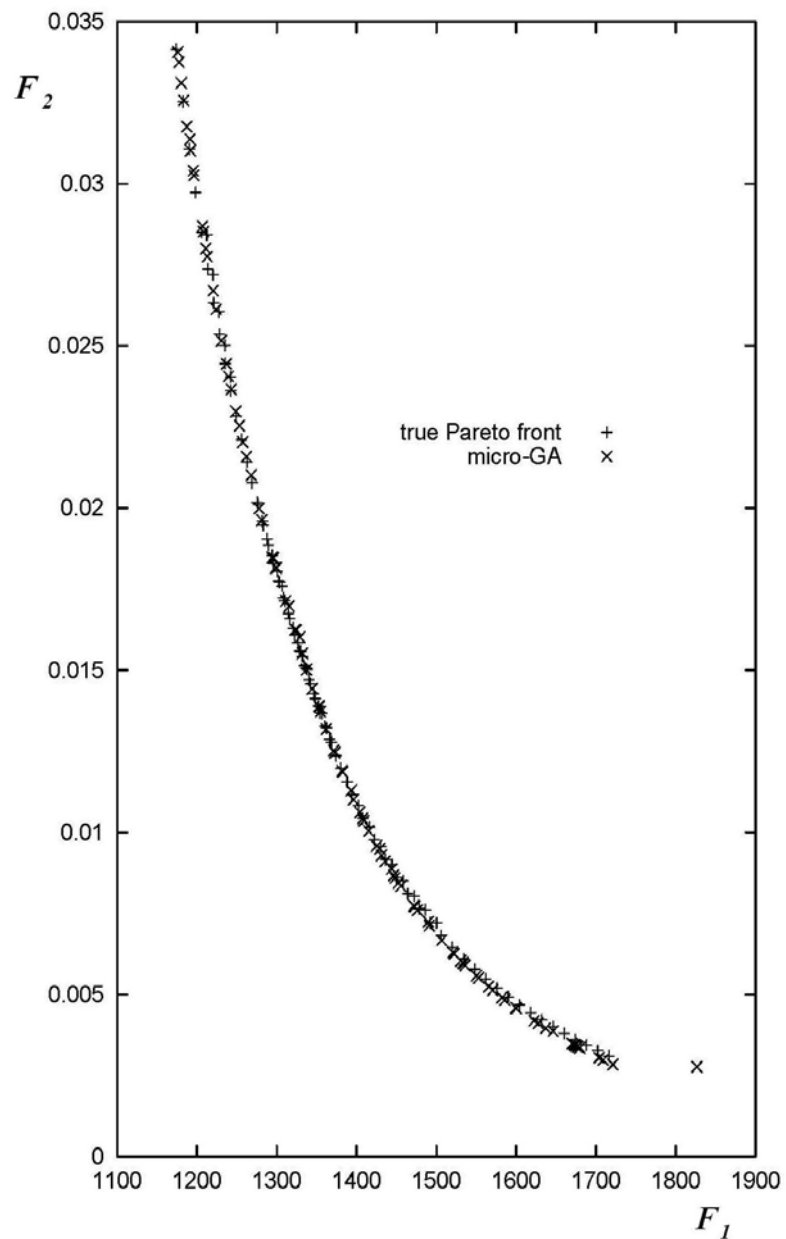


Figure 7. True Pareto front vs. front obtained by micro-GA



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Some examples of structural optimization problems modeling

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Summary

Some examples of different optimization problems models that were under interest of the author during some past years are presented in a paper. These problems concern: discrete optimization of steel frames with accounting for the second order effects in structural analysis, shape optimization of section under torsion by using BEM, and evolutionary structural optimization method for shape and topology strength optimization.

KEYWORDS: discrete optimization, plane frames, linear analysis, P-delta method, shape optimization, torsion problem, BEM, evolutionary structural optimization

1. DISCRETE SYNTHESIS OF STEEL FRAMES ACCOUNTING FOR P-DELTA EFFECTS

1.1. Mathematical formulation of the optimization problem

In the analysis and design of multi-storey steel frames it is necessary to consider the influence of nonlinear geometrical effects which are caused by vertical loads acting on horizontal displacements of the structure and on deflections of its columns. These additional effects generally occur in the overturning and torsional moments, and are known as P-delta effects [1]. As these effects are represented by changes in the internal forces distribution over the structure, and by changes in their values, they also have an influence on the results of the minimum-weight design of high-rise frames.

The design problem is formulated as follows [2]:

Obtain the minimum-material volume design of the structure taking into consideration the influence the second-order P-delta effects.

In the proposed optimization model values representing the cross-sectional member dimensions are assumed as a design variable vector \mathbf{X} and material properties as parameter vector \mathbf{P} . For the I-welded sections four simple design variables are taken: web plate and flange plate width and thickness. Set of constraints is defined



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by design constraints fixed on design variables and by behaviour constraints: displacements, stresses, and local stability in form of the mathematical formulas:

Geometrical constraints: $Q_g(\mathbf{X}, \mathbf{P})$

$$x_{imin} = x_{i1} < x_{in} < x_{imax}, x_{in} - x_{in-1} = \Delta x, x_{imax} - x_{imin} = 2^q \Delta x_i, \quad (1)$$

$$n = 1 (\text{min}), \dots, k (\text{max}), i = \bar{i}, \dots, \bar{j}, q = 1, 2, \dots, I.$$

Stress constraints: $Q_s(\mathbf{X}, \mathbf{P})$

$$\sigma_{k1}(\mathbf{X}, \mathbf{P})/R_{k1} - 1 \leq 0, k = 1, \dots, d. \quad (2)$$

Local stability geometrical and stress constraints: $Q_{ls}(\mathbf{X}, \mathbf{P})$

$$\sigma_{l2}(\mathbf{X}, \mathbf{P})/R_{l2} - 1 \leq 0, f_t(\mathbf{X}, \mathbf{P}) \leq a_t, l = 1, \dots, d, t = 1, \dots, f. \quad (3)$$

Displacement constraints: $Q_d(\mathbf{X}, \mathbf{P})$

$$v_{mh}(\mathbf{X}, \mathbf{P}) \leq v_{mhp}, v_{nv}(\mathbf{X}, \mathbf{P}) \leq v_{nvp}, m = 1, \dots, g, t = n, \dots, h. \quad (4)$$

In the equations (1) to (4): x_{imin} , x_{imax} , Δx_i – minimum, maximum, and constant step between design variables; σ , R – current stresses and their permissible values, f , a – current geometrical constraint value and its permissible value, v , v_p – current displacements and their permissible values.

It is obvious that to obtain the required optimal solution – the optimal structure – the following relation must be fulfilled:

$$Q_g \wedge Q_s \wedge Q_{ls} \wedge Q_d \in \{0\}. \quad (5)$$

As an objective function volume of the material used for a structure is taken. Optimization problem is then formulates in the following term:

Find the design variable vector \mathbf{X}^* in the feasible set (5), with parameters \mathbf{P} , which minimize the value of the global objective function.

1.2. Mathematical programming technique

Well-known discrete programming method called “backtrack” [3] was used to find the optimal vector \mathbf{X}^* . This combinatorial method that can solve nonlinear constrained function minimization problem by systematical search procedure was very useful in the presented optimization problem.

1.3. Examples of synthesis process

The process of discrete synthesis of steel frames was implemented in a computer program for second-order analysis. Optimization process was conducted until the difference of values of global objective function for two successive steps was less



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than the specified tolerance (for example 0,1%). As an example the results of optimization of two-bay three-storey frame are presented in Figures 1 and 2.

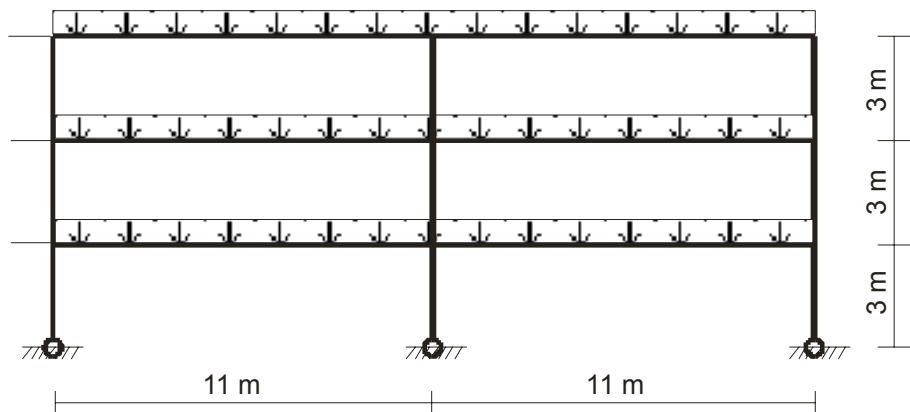


Figure 1. Optimization problem

The results of discrete synthesis of steel frames with accounting for P-delta effects in structural analysis shown that for a considered class of structures these effects led to obtain slightly “heavier” structures and, what was more important, led to material and forces redistribution in structural elements. These changes caused overstress in some elements that were optimized with linear analysis.

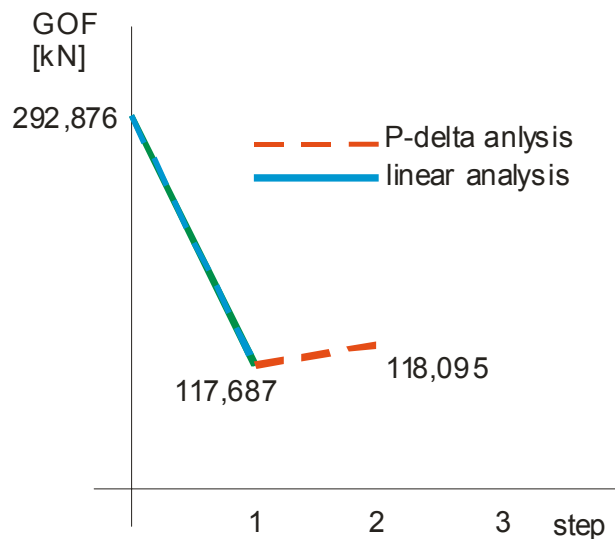


Figure 2. Synthesis results – iteration process



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2. SHAPE OPTIMIZATION OF SECTIONS UNDER SAINT - VENANT TORSION

2.1. Formulation of torsion problem for isotropic solids with BEM

The solution of the Saint-Venant torsion problem was based on that formulated by Gracia [4] and Gracia nad Doblaré in [5, 6], where a general problem of shape optimization of 2D elastic bodies based on the boundary element method was presented.

To formulate the Saint-Venant torsion problem for isotropic and homogenous solids and for multiply-connected domains \mathbf{W}_i the so-called Prandtl function was used leading to a Poisson equation:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = -2 \quad \text{in } \Omega, \phi_i = k_i \text{ in } \Gamma_i, i = 0, 1, \dots, N, \quad (6)$$

where N is the number of boundaries \mathbf{G}_i ,

$$\int_{\Gamma_i} \frac{\partial \phi}{\partial \mathbf{n}} d\Gamma_i = -2A_i, i = 1, \dots, N \quad (7)$$

where:

$$A_i = \frac{1}{2} \sum_{j=1}^{N_i} (\mathbf{r}_j \cdot \mathbf{n}_j) L_j \quad (8)$$

is the area enclosed by each internal boundary, N_i is the number of elements of each internal boundary, L_j is the length of the element 'j', \mathbf{n}_j is the normal to it, and \mathbf{r}_j is the radius-vector between the element and the origin of coordinates.

The introduction of the second Green's identity between the Prandtl function (6) and the fundamental solution of the Laplace equation led to an alternative formulation of Equation (6) in terms of boundary integrals [10,11,12]:

$$c(Q)\phi(Q) + \int_{\Gamma} \phi \frac{\partial}{\partial \mathbf{n}} \left(\ln \frac{1}{r} \right) d\Gamma = \int_{\Gamma} \frac{\partial \phi}{\partial \mathbf{n}} \ln \frac{1}{r} - \int_{\Omega} \nabla^2 \phi \ln \frac{1}{r} d\Omega \quad (9)$$

where the constant c takes values depending on situation of point Q on the boundary and r is the radius-vector joining boundary points and the coordinate system origin.

Using (6) Equation (9) was transformed to the BEM basis:



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$$c(Q)\phi(Q) - \int_{\Gamma} \phi \frac{\mathbf{r} \cdot \mathbf{n}}{r^2} d\Gamma = - \int_{\Gamma} \frac{\partial \phi}{\partial \mathbf{n}} \ln r + \int_{\Gamma} \left(\frac{1}{2} - \ln r \right) (\mathbf{r} \cdot \mathbf{n}) d\Gamma \quad (10)$$

and, by the similar way, a formulation for torsional stiffness was obtained:

$$D = -G \left[\frac{1}{4} \int_{\Gamma} r^2 (\mathbf{r} \cdot \mathbf{n}) d\Gamma + \frac{1}{2} \int_{\Gamma} \frac{\partial \phi}{\partial \mathbf{n}} r^2 d\Gamma \right]. \quad (11)$$

The approximation functions ϕ were formulated by using the simplest constant and linear approximations that led to the torsional stiffness expressed by:

$$D = -G \left[\frac{1}{4} \sum_{i=0}^N \sum_{j=1}^{N^{ei}} \int_{\Gamma_j} r^2 (\mathbf{r} \cdot \mathbf{n}) d\Gamma_j + \frac{1}{2} \sum_{i=0}^N \sum_{j=1}^{N^{ei}} \sum_{k=1}^2 q_j^k \int_{\Gamma_j} \phi_k r^2 d\Gamma_j \right]. \quad (12)$$

2.2. Formulation of optimization problem

The problem of shape optimization of sections under the Saint-Venant torsion can be stated in the considered case as it was formulated with details in [4, 5, 6, 7]:

Obtain the shape of the section with minimum area having a given torsional stiffness, and that fulfils some constraints related to the section geometry. It should be mentioned that as its dual problem the problem of finding the section with a given area and maximum torsional stiffness could be considered.

These additional constraints are as follows:

- some coordinates of the nodes can be bound;
- some boundary nodes can be fixed;
- the boundaries can not intersect;
- symmetry conditions have to be fulfilled.

The objective function $f(\mathbf{x})$ is defined as:

$$f(\mathbf{x}) = \frac{1}{2} \sum_{i=0}^N \sum_{j=1}^{N^{ei}} (\mathbf{r}_j \cdot \mathbf{n}_j) L_j, \quad (13)$$

and the restriction corresponding to the torsional stiffness:

$$h(\mathbf{x}) = D_0 + G \left[\frac{1}{4} \sum_{i=0}^N \sum_{j=1}^{N^{ei}} (I_j^1)_2 + \frac{1}{2} \sum_{i=0}^N \sum_{j=1}^{N^{ei}} q_j (I_j^1)_1 \right], \quad (14)$$

where D is defined by Equation (12), and as design variables \mathbf{x} , in this first approximation to the optimization problem, the non-restricted boundary node coordinates were taken.



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The method used in the optimization problem was based on the feasible direction method and the gradient projection one as it was described in [4, 5]. The restriction imposed on the torsional stiffness was transformed into "the constraint strip" by using the error bound ε_r and though the restriction is satisfied when:

$$(1 - \varepsilon_r) \leq \frac{D}{D_0} \leq (1 + \varepsilon_r), \quad (15)$$

where D is the torsional stiffness of the current design and D_0 is the constraint stiffness. The method of the automatic constraint strip adjusting at each iteration step was applied.

2.3. Interactive graphical program for shape optimization

The overworked interactive program [8] for definition, visualization and modification of the shape optimization problem's data and results consists of two fundamental parts: graphical unit used to define and redefine the problem geometry (graphical pre- and postprocessors) and optimization unit that performs a design process, basing its analysis part on the BEM.

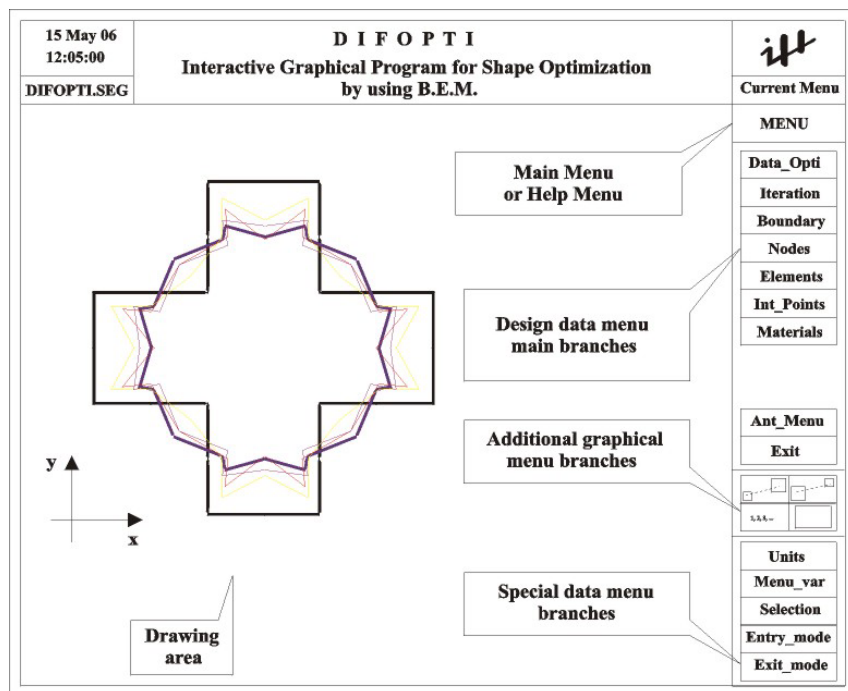


Figure 3. Optimization program graphical interface and section's shape evaluation during optimization process for the so-called "Greek cross"



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A general idea of an interactive work with the program consists in giving to a program user a possibility to define graphically the geometry of an initial shape to be optimized and then to have a chance to observe the optimization process at any iteration step.

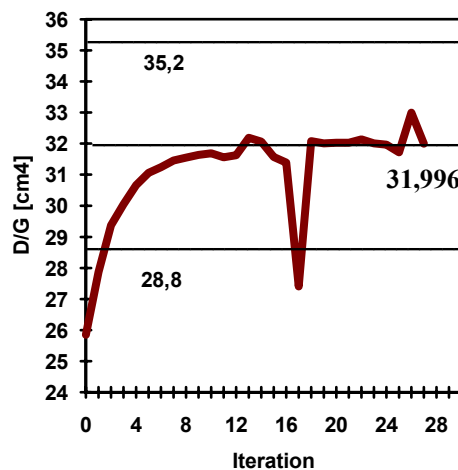
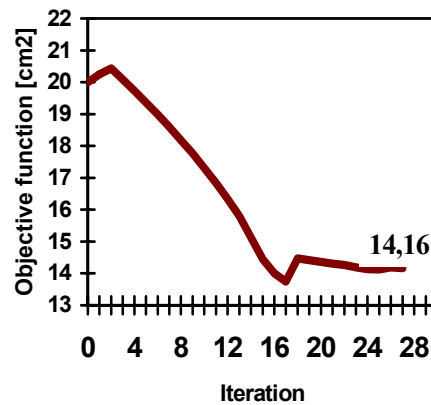


Figure 4. Evaluation of objective function and restriction during optimization

The graphical unit of the program contains a group of geometrical tools (calculating of boundary elements' length, areas closed by boundaries and angles between elements, detection of boundary intersections and mesh redefinition), completed by those of the visual presentation of optimized shapes (drawing of boundaries with different zoom levels, graphical presentation of the objective function and restriction evolution).



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An example of realized optimization process refers to the shape optimization of simply-connected domain that is so-called "Greek cross" with an obvious final result in the shape of a circle (Figure 3).

The optimization process converged rapidly with the constraint value D_0/G always inside the constraint strip (Figure 4). However, the boundary of final design is not very smooth because of lack of the mesh redefinition in case of appearance of geometrical mesh irregularities (Figure 3).

3. EVOLUTIONARY STRUCTURAL OPTIMIZATION

3.1. Formulation of the method

Evolutionary Optimization Method (ESO) is based on the simple concept that removing step-by-step inefficient material [9] leads to the optimal shape of the structure. This process is controlled by coefficients that define from which part of the structure, how many, and when the material is removed from the structure. In every case all the restrictions are fulfilled. This method is very useful in shape optimization problems.

This method was applied to solve strength shape optimization problems [9, 10]. Optimization with minimum material criteria leads to structures shaped in accordance with principal stresses trajectories in equivalent shield structure (structural domain) with identical geometrical and boundary conditions as searched structure. ESO process for that kind of problems can be described as follows:

1. Creation of finite element mesh in a initial domain.
2. Structural analysis with FEM to find stress distribution. In cases presented below [8, 9] for plane stress model Huber- von Mises stress was defined:

$$\sigma^{hvm} = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \cdot \sigma_y + 3 \cdot \tau_{xy}^2} \quad (16)$$

3. Calculation of stresses in every finite element σ_e^{hvm} and definition of material rejection criteria:

$$\sigma_e^{hvm} / \sigma_{\max}^{hvm} < RR_i, \quad (17)$$

where σ_{\max}^{hvm} is domain maximum stress value and RR_i is a current rejection ratio (for example 1%).

4. Removal of finite elements with stress that satisfy equation (17) until the process is steady (by assigning zero stiffness value to the element). It means that with the same values of R_i there are no elements that can be deleted.



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5. Introducing the so-called “evolutionary rate” ER ($RR_{i+1} = RR_i + ER$) and repetition of FEM and stress analysis, and element removal until new steady stage is reached (for example $E = 1\%$).
6. Optimization process is conducted until, for example, when there are no elements in the domain with stresses $\sigma_e^{hvm} < 25\% \sigma_{max}^{hvm}$.

3.2. ESO optimization examples

First example is a well-known structural optimization problem of the two-bar frame subjected to a single load placed in the middle of the long domain side. The optimal ratio of H/L can be obtained analytically and is $H/L = 2$ and a structure is a pin-jointed frame.

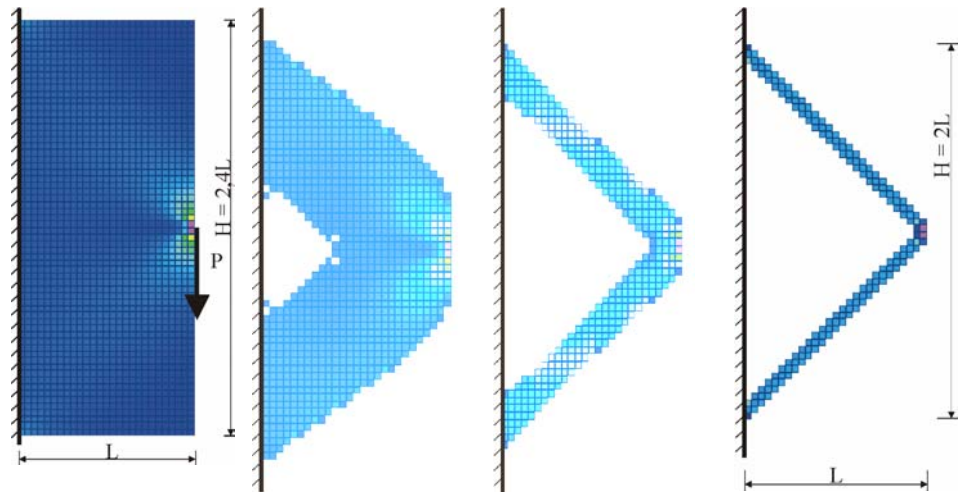


Figure 5. Design domain. ESO solution for $R = 5\%$, 12.5% , & optimal solution for $R = 30\%$

Second example is ESO solution for a Michell type structure with two fixed ends.

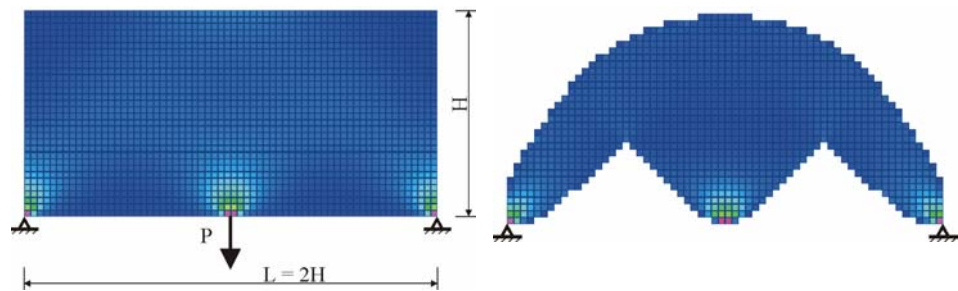


Figure 6. Design domain and ESO solution for $R = 5\%$.



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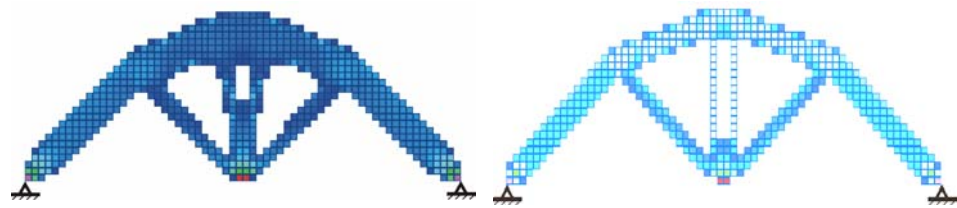


Figure 7. ESO solution for $R = 10\%$ and optimal solution for 15% .

4. FINAL REMARKS

A summary of author's interest in different problems of optimization that evaluated from simple discrete frame structural optimization with second order analysis, by shape optimization with BEM to the shape optimization with evolutionary method was presented above.

All the described optimization problems were also introduced into didactic programs in course of structural optimization and computer aided engineering allowing students to know a different way of structural designing.

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On the dynamic behavior of some suspended roofs

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Summary

This article presents a simple suspended cable system, in which there are presented calculating formulas for tensions, mainstay loading, the arrow and its increase after taking into consideration the cable's load. In the second part, a roof system with a double curvature is presented, its behavior and how it deals with the resonance phenomenon. The possible elimination of the flutter effect that might appear at this type of roof is being observed, as well.

KEY – WORDS: cable, tensed membranes, arrow, double layer cable, suspended simple roof, vibration.

1. INTRODUCTION

Cabled roofs allow big openings, which are indicated when it comes to public constructions: show rooms, stadiums, trading centers, gymnastics halls, etc. The suspended roofs structures present a relatively high sensibility concerning the winds dynamic action. Being light, the roofs encourage the flutter effect under the action of the lifting aerodynamic forces. This effect is also facilitated by high amplitudes and by cables reduced damping.

The winds action manifests itself through fluctuations, which are nearly periodic and can lead to resonance phenomena.

Both the flutter, which is generated by vertical oscillations, coupled with waving oscillations, and the resonance phenomenon can appear especially on certain structural systems, but on principle, both of them must be analyzed, since they can lead to the structure's collapse.

Cabled suspended roofs are economical especially in case of big openings. Delicate problems also appear at the cables anchorage or their pretension, if pretension exists.

The conception of the cable system and of the covering elements is responsible for the structure's flexibility, which is not significant. And so, the roof surface can be well defined in space and its deforming to different actions, especially the static



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ones must not lead to important modifications of the deformed system, compared to the initial uncharged one.

Cabled roofs are made just as the thin curved board roofs are made, with certain specific differences. There can be roofs that have the form of simple curved roofs, for example cylinder surfaces or double curved ones – hyperbolic, parable shaped, etc. A type of roof which is frequently used is the bicycle wheel type, where the cables are anchored in an exterior ring and in a central device.

Suspended systems can be made with a simple surface, with a simple net of cables or with a double surface made of two layer cable, distanced on struts. Thus, a cabled roof has the shape of an imaginary membrane extended above cables put on a surface or on two surfaces which are connected to each other and complementary.

A significant size of these roofs is the ratio between the maximum arrow and the span which is normally between 1:15 – 1:20.

Suspended roofs significant loads are: its own weight and the weight of the covering membrane, the snow action, the wind action, a possible seismic activity. The winds' action always has a static component and a dynamic component which generate vibration phenomena, accompanied by the already mentioned effects: flutter and resonance.

Irrespective of the analysis made for the suspended system projection, its behavior is of great interest both in terms of its individual suspended cable static action and its answer to the dynamic action.

2. SUSPENDED CABLE ACTIONED BY STATIC LOADS

Depending on the types of structure, the mainstays and the cables' anchoring points are situated on the same level, a case frequently seen, or on different levels. Generally speaking, on suspended roofs the cables are stretched and - as mentioned earlier – the arrow / span ratio is low.

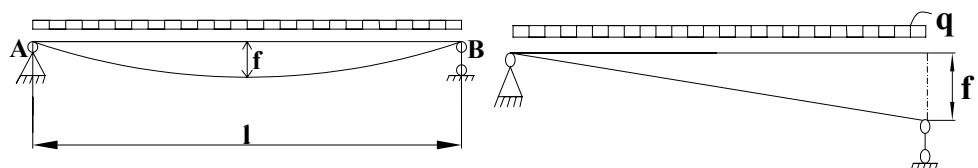


Figure 1. Uniform load of a cable



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With stretched cables, the deformed shape is parable-shaped, if the charging is uniform between the mainstay points, Figure 1. It can have as source the cable's weight, the covering elements, and the snow.

Due to its flexibility in a transversal manner, the cable is subjected to tension only. The tension in cable T in the section area has the direction line tangent to the line that represents the deformed configuration of the cable.

When it comes to vertically distributed loads, the horizontal component of the T effort in the cable is constant.

From the immediate relation:

$$H = T \cos \alpha \quad (1)$$

α being the angle made by the horizontal and the tangent to the deformed curve.

The conclusion:

$$T = H / \cos \alpha \quad (2)$$

And obviously the minimum T_0 value is for $\alpha = 0$, that is, when the arrow is maximal f , so T increases towards the mainstays, having maximum values in the suspension points.

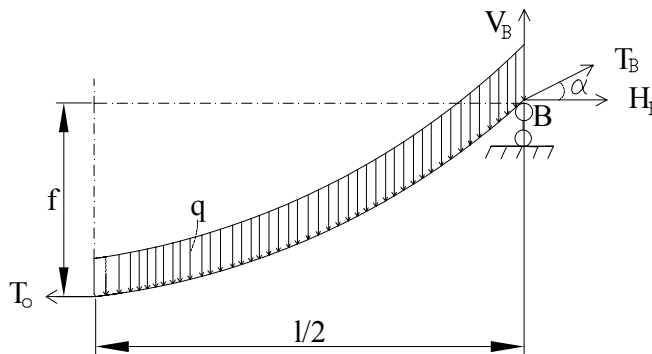


Figure 2. Equilibrium projection

From the equilibrium conditions, projection on the vertical and moment in relation to point B, represented in Figure 2, are obtained:

$$V_B - q \frac{l}{2} = 0 \quad (3.a)$$

$$T_0 f - q \frac{l^2}{8} = 0 \quad (3.b)$$



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It yields:

$$V_B = q \frac{l}{2} \quad (4)$$

$$T_o = \frac{ql^2}{8f} = H = H_B \quad (5)$$

The T_{max} value is T_B and it immediately leads to:

$$T_{max} = \sqrt{V_B^2 + H_B^2} = \sqrt{\left(\frac{q^2}{2}\right)^2 + \left(\frac{ql^2}{8f}\right)^2} = \frac{ql^2}{8f} \sqrt{1 + 16\left(\frac{f}{l}\right)^2} \quad (6)$$

The α angle made by the cable tangent in the support point results from:

$$\operatorname{tg} \alpha = \frac{V_B}{H_B} = 4 \frac{f}{l} \quad (7)$$

The initial length of the cable before it acts with q intensity is given by the relation:

$$L = l \left(1 + \frac{8}{3} \left(\frac{f}{l} \right)^2 \right) \quad (8)$$

As a consequence of the force of intensity q , the cable length modifies becoming $L' = L + \Delta L$ and:

$$\Delta L = \frac{TL}{EA} = \frac{T}{EA} \left(1 + \frac{8}{3} \left(\frac{f}{l} \right)^2 \right) \quad (9)$$

Where:

E = the cable equivalent elasticity module

A = cable section equivalent area

The increase of the arrow Δf due to the stretch of the cable Δl is expressed by the relation:

$$\Delta f = \frac{\Delta L}{\frac{16f}{15l} \left(5 - 24 \frac{f^2}{l^2} \right)} \quad (10)$$

Through the increase of the arrow, the values of the efforts $T_o = H$ and T_{max} are diminished according to the relations (5) and (6).



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3. CABLE VIBRATIONS FREQUENCIES

A tensed cable which vibrates is assimilated to a string and the frequencies of the free vibrations are calculated accordingly.

$$\omega_n = \frac{n\pi}{l}c \quad n=1,2,\dots \quad (11)$$

Where c , meaning speed, has the expression:

$$c = \sqrt{\frac{H}{\rho A}} \quad (12)$$

H = the effort in the string, and $\rho A = m$ is the mass of unit length.

The n modules own vibrating frequency of a stretched cable, with no amortization, with an l length, perfectly flexible and, with a uniformly distributed load on a horizontal projection of m , intensity is given by the expression:

$$\varpi_n = \frac{n\pi}{l} \sqrt{\frac{T}{m}} \quad n=1,2,\dots \quad (13)$$

Obviously, in the case of the uniform q loading $m=q/g$, where g = the gravitational acceleration.

Because T is proportional to load q , the natural frequency does not depend on the value of the load, with the mentioning of the fact that in calculating T the initial arrow f is used, and in the cable there are no other tensions.

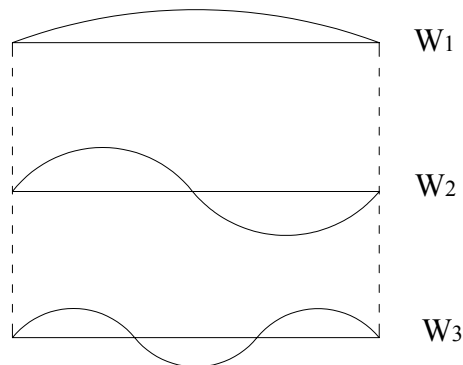


Figure 3. The first 3 vibration modes

In the stretched cable, for $T = \frac{ql^2}{8f} = H$, the ϖ_n frequencies become:



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$$\omega_n = n\pi \sqrt{\frac{g}{8f}} \quad (14)$$

The frequencies increase with n , the smallest corresponding to $n = 1$, n is the number of the vibration waves. In Figure 3, the first 3 vibration modes are presented.

Frequencies are independent in relation to m if there are no other actions and / or tensions in the cable, which modify the answer in frequencies. In this way, the ballast of the cables can interfere, by increasing the f arrow, as well as the efforts in the cable. Efforts due to the cables pre-tensioning can also occur.

An exterior disturbing force with pulsating character, for example one that is given by the wind action, whose frequency is ω_e , modifies the answer in the cable frequencies, i.e. the cable vibration amplitudes are determined by the ratio ω_e / ω_n .

Near the values $\omega_e / \omega_n = 1, 2, 3, \dots$ may be seen an excessive increase of the amplitudes and there occurs the resonance phenomenon.

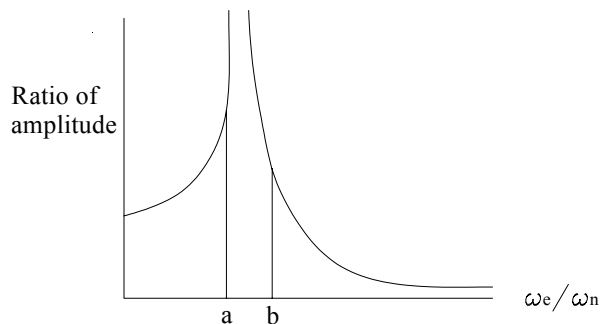


Figure 4. Interval of amplification

Therefore, in a graphic representation (Figure 4) in which appear the ratios ω_e / ω_n on the abscissa, there may be seen the intervals of amplification and resonance for which the suspended roofs have a destructive character.

The increase of the effort in the cable by the roof ballast or through the cable's pre-tensioning increases the roof rigidity and also, introduces amortizations through constructive means.

Erecting roofs with the shape of two shells by introducing double cables distanced through vertical stanchions leads to interconnected surfaces with a high rigidity which diminishes and even removes both the flutter and the resonance phenomenon.



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4. EXAMPLE OF CALCULATION FOR A ROOF MADE OF ONE CABLE LAYER

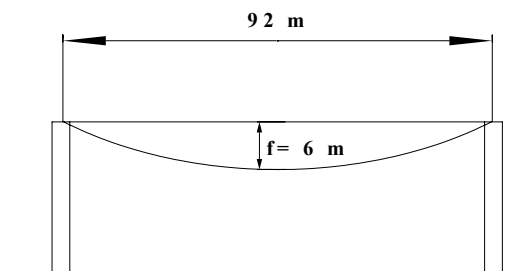


Figure 5. Section

The span is 92 m, the maximum arrow 6 m.

The dead load, the cable weight, included is:

$$q_p = 1000 \text{ N} / \text{m}^2 .$$

The live load on the cables is:

$$q_a = 1556 \text{ N} / \text{m}^2$$

The distance between 2 successive cables is:

$$l_c = 1.2 \text{ m}$$

Assuming

$$f = \frac{1}{15} l$$

It yields $f = 6.13 \text{ m}$.

The total load is:

$$q_t = (q_p + q_a) l_c = (1000 + 1556) 1.2 = 2130 \text{ N} / \text{m} = 2130 \text{ kN} / \text{m}$$

The cable tension is present in relation (6):

$$T = q \frac{l^2}{8f} \sqrt{1 + 16 \left(\frac{f}{l} \right)^2} = \frac{2.13092^2}{8 \cdot 6.13} \sqrt{1 + 16 \left(\frac{6.13}{26} \right)^2} = 378 \text{ kN}$$

There can be used a galvanized steel cable with $\varnothing = 5 \text{ cm}$ and the section area $A = 15.6 \text{ cm}^2$, which has an elasticity module $E = 165.4742 \text{ KN} / \text{mm}^2$.



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In order to determine the cable elongation, expression (9) is used in which for T , has been assumed the value from the extremities and for L the value determined in relation (8), so:

$$L = l \left(1 + \frac{8}{3} \left(\frac{f}{q} \right)^2 \right) = 92 \left(1 + \frac{8}{3} \left(\frac{6.13}{92} \right)^2 \right) = 93.089 m$$

$$\Delta L = \frac{TL}{EA} = \frac{378 \times 93,089}{165.47 \times 42.1560} = 0.136 m$$

It is possible to calculate the growth of the arrow Δf due to the elongations ΔL :

$$\Delta f = \frac{\Delta L}{\frac{16}{15} \frac{f}{q} \left(5 - 24 \left(\frac{f}{l} \right)^2 \right)} = \frac{0.136}{\frac{16}{15} \frac{6.13}{92} \left(5 - 24 \left(\frac{6.13}{92} \right)^2 \right)} = 0.391 m$$

Next the cable's own frequencies may be calculated by means of relation (13):

$$\omega_n = n \frac{\pi}{l} \sqrt{\frac{T}{q/g}} = n \frac{\pi}{92} \sqrt{\frac{378}{2.13/981}} = n \cdot 1.425 Hz$$

For $n = 1, 2, 3, \dots$ the first pulsations are obtained, the fundamental one being 1.425 Hz.

The values T and ω_n being corrected, $f + \Delta f$ will be used for the arrow and $L' = L + \Delta L$ for the cable length.

$$T' = q \frac{l^2}{8(f + \Delta f)} \sqrt{1 + 16 \left(\frac{f + \Delta f}{l} \right)^2} = \frac{2.130 \cdot 92^2}{8 \cdot (6.521)} = \sqrt{1 + 16 \left(\frac{6,521}{92} \right)^2} = 359.2 kN$$

$$\omega'_n = n \frac{\pi}{L + \Delta L} \sqrt{\frac{t}{q/g}} = n \frac{\pi}{93.225} \sqrt{\frac{378}{2.15/9.51}} = n \cdot 1.406 Hz$$

It can be concluded that the pulsation determined with the initial values of the cable length is not essentially different from the pulsation determined using the real length $L + \Delta L$.

Concerning the cable tension, a diminishing can be noticed once the arrow increases through the cable deformation (its elongations), a favorable effect for the suspended roofs, since through the cables' deformation there is produced a tendency of structural stabilization.



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5. ROOFS WITH DOUBLE SURFACES

The cables which form double surfaces coupled through vertical rigid bars suggest a behavior to applied loads which is similar to a beam, the inferior cables being further stretched while the tensions in the superior cables are diminished. Mention should be made that regularly the ratios $f : L$ remain, more or less, the same for the convex and concave cables and the charging capacity of the roof surface depends on the way in which the curve of the roof surface is made and less on the individual cables' curves.

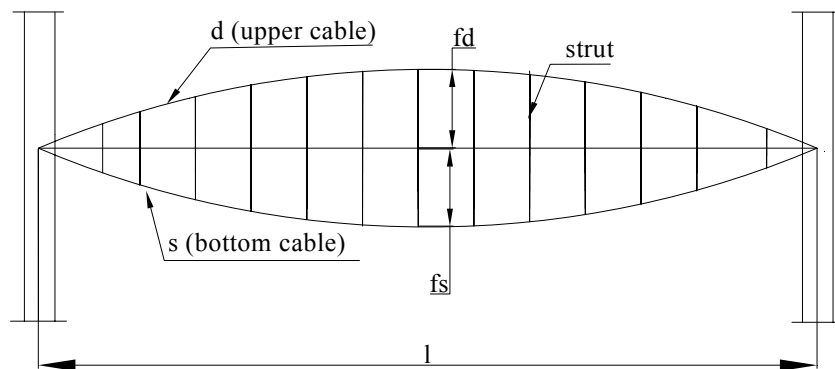


Figure 6. Double layer cable

The cables d and s are used with initial tensions Td and Ts and the value of these tensions is function of the number and the positions of the vertical struts, to their dimensions and weights of the two types of cable, Figure 6. When used, the charging is given by its own weight and by the weight of the struts.

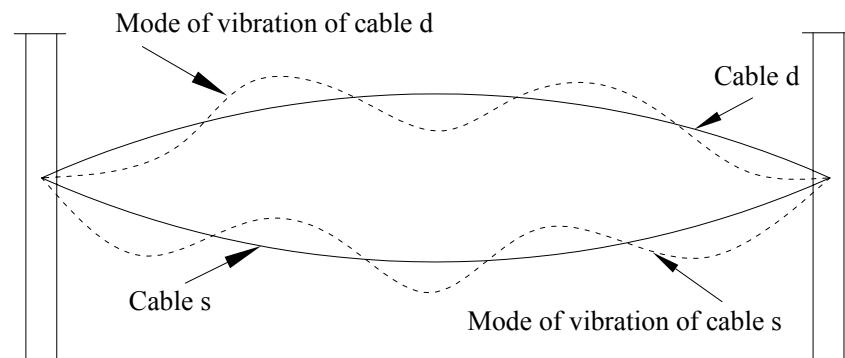


Figure 7. The geometry of vibration



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Through dead and live loading, the tension in the inferior cable increases with delta T_d and the one in the superior cable diminishes with delta T_s . But these variations must be controlled; the excessive diminishing of the tension in the superior cable is especially dangerous. But these observations show that the pulsations of the two surfaces where the inferior one is generated by the cables underneath and the superior one by the upper cables will be different, Figure 7.

An energetic flux will be created between the two surfaces resulting in the amortization of the vibrations.

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Harmonization of the Romanian structural design codes with the European codes (Euronorms). National applicable documents

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Summary

Harmonization of the technical base for the Civil Engineering Design and the classes, quality and performances of the materials, equipments and broadly the construction products must assure the growing mobility of the designers and/or the project work, the elimination of the technical and commercial barriers inside the construction industry, the equal treatment inside all the European countries of various structural systems, materials and products.

The translation of the structural Eurocodes and the draw up of the CR (Romanian Codes) are very important operations and must be duplicated by a large action of dissemination and explanation to make them known to the engineers that should apply or work collateral with these norms. The DANs (National Appliance Documents) must be send to investigation for a 2 years period and enter in application with a temporary code status, simultaneously with the existent national Codes.

KEYWORDS: Euronorm, Eurocode, Romanian Design Code

1. THE EUROPEAN ECONOMIC INTEGRATION

The idea of co-operation and afterwards the European economic integration was born in 1923, at the same moment with the Pan-European Movement; it has quickly evolved after the IInd World War by the foundation of a large number of organizations having various missions (co-operation, defense, economic assistance etc) as follows: the European Federal Union (1946), the European League for Economic Co-operation (1947), the Marshall Plan (1948-1952), the Organization for European Economic Collaboration (1948), the Occidental European Union (1948), NATO (1949), the European Council (1949, center at Strasbourg), the European Community for Coal and Steel (1951), the European Community for Atomic Energy (EURATOM, 1957).



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The conditions for the set up and the governing principles of the **European Economic Community, EEC**, or the **European Union, EU** – briefly, have been stated by the Rome Treaty in 1957. The agreement became operative in 1959. The activities of this organization are coordinated by 3 units: the European Parliament, the European Union Council and the European Commission. The **European Union Council** is the main institution for the decision inside the European Union; it groups following a certain schedule the representatives of the membership countries at the ministry level (Foreign Affairs, Finances, Education – Teaching, Research, Public Works and Communications etc.).

The main **objectives** of UE are: the promotion of a continuously, harmonious and equilibrated development of the membership countries, **the set up of a Common Market, the harmonization of the laws** in order to make possible the function of the Common Market, the elimination of the charges, the free motion of goods, persons, services and capital, the creation of new working places etc.

Starting with 1985, the EU Foundation Treaty has been modified and completed by series of decisions of the EU Council in the view of the creation of the **European Single Market or Common Market**. Under these circumstances it was set up the basis for the initiation of the harmonization process of the rules and technical codes at the Community level, as a premise for the elimination of the existent restrictions concerning the free circulation of goods and services.

The Common Market inside EU rose officially in 1993, occasion in which the harmonization of the prescriptions that existed in the membership countries and their transformation upon new bases in Euronorms became urgent and necessary. The construction market as part of the internal market of EU – in which the competition must act freely – has an important percentage, therefore it results the importance granted to the system of construction prescriptions, for the purpose of good proceedings that belong to this branch.

2. THE HARMONIZATION OF THE TECHNICAL PRESCRIPTIONS REGARDING THE CONSTRUCTION DOMAIN

The harmonization of the technical base for i) Civil Engineering Design and ii) the classes, quality and performances of the materials, equipments and broadly the construction products must assure the growing mobility of the designers and/or the project work, the elimination of the technical and commercial barriers inside the construction industry, the equal treatment inside all the European countries of various structural systems, materials and products.



Harmonization of the Romanian codes with the European codes. National applicable documents

A state member of EU or associated to, must be able not just to create opportunities for the production of goods in accordance to the community provisions but also to guarantee that all the products introduced into this market – projects, materials, equipments etc. – meet the codes. This means that all the membership states have to embrace the suitable legislation and the set up of all the necessary structures, techniques etc. to apply effectively the new legislation. These structures – testing laboratories, research institutes, metrology institutes etc. – must gain the trust of the entire EU.

From the law perspective, the harmonization measures of the system and the technical provisions were stated mainly by the means of two documents of the EU Council, as follows:

- The Public Works Directive, 89/440/EEC;
- The Directive for the Construction Products (DPC), 89/106/EEC.

These Directives are establishing both the functioning principles of the construction market and the criteria for the set up of the technical codes, also the way to correlate/ interpret the laws, codes and the decrees from the membership countries. The importance of the Directives consists of the fact that are based on a **new, modern approach** of the entire activity inside the constructions domain, that marks the provision for a product or a class of products (materials, elements, structures, buildings as a whole, installations and equipments, projects etc.) of a number of **main demands** or **essential requirements**. These six **essential requirements** are:

- a) Strength and stability,
- b) Exploitation safety;
- c) Fire safety,
- d) Hygiene, human health, the environment restoration and protection;
- e) Thermal and hydro- insulation, energy reduction and heat retention;
- f) Protection against the noise.

In relationship with the six requirements, six documents were elaborated, named **(Interpretative Documents, ID)** named ID1...ID6 that stand out the design, products and construction demands. According to these documents, the legislation must refer mainly to the:

- A) Technical provisions for the construction design;
- B) Codes pertinent to the material and product quality used for the constructions;
- C) Provisions that state the way of elaboration of the technical agreements for new products, equipments, materials, procedures etc.

The EU Council ordered that the elaboration of the European system of provisions and technical codes should be the task of 3 of the most important standardization institutions: (Comité Européen de Normalisation, CEN), the European Committee



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of Electro technique Standardization (CENELEC) and the European Institute for Standardization in Telecommunications (ETSI). Now there are more than 5000 European Codes. It is expected that in the end the Euronorms will replace completely the National Codes. The publishing of the stated provisions is mainly performed by the Secretariat of the Free Exchange European Association (AELE). This paperwork will present only the works regarding the harmonization of the technical design provisions (as stated previously at pct. A), namely Structural Euronorms.

3. HARMONIZED DESIGN CODES STRUCTURAL EURONORMS

On the basis of the Interpretative Document no. 1 (ID1) with respect to the exigency „a) strength and stability” it was stated that the commitment of the harmonized design provisions or the structural Euronorms (so-called EUROCODES) must be the task of a **Technical Committee, (TC)** from the frame of the European Standardization Committee (CEN) named **CEN/TC 250 Structural Eurocodes**, by having the following mission: „The standardization of the rules and structural design methods for buildings and engineering structures by taking into account the connections between the computational provisions, the material behavior, the building technology and control”.

The initiative of elaboration of international codes for the design of the strength structures appeared in 1974 and it is set on the co-operation between technical-scientific and professional organizations, having a well-known activity in the European and international field. These organizations are:

- IABSE International Association for Bridge and Structural Engineering
- CIB Conseil International du Bâtiment
- RILEM International Association of the Testing and Research Laboratories for Materials and Constructions
- CEB Euro – International Committee for Concrete
- FIP International Federation for Pre stressed Concrete
- ECCS European Convention for Constructional Steelworks
- JCSS Joint Committee on Structural Safety
- ISSMFE International Society for Soil Mechanics and Foundation Engineering

The basic rules for the structural calculus have been elaborated in the frame of JCSS. On the basis of the hazard concept safety conditions and serviceability conditions were developed, by considering the structural reliability criteria.



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These conditions assured the common base of the computational codes, thus creating the opportunities for the elaboration of the Structural Eurocodes. A harmonization process was followed for the frame principles of the codes inside the membership countries regarding the building materials, the building methods, building classes and engineering constructions.

In the Interpretative Document no. 1 (ID1) there are also mentioned:

- Eurocodes will serve as reference codes, being submitted to the authority appointment of the membership countries and have the goals:
 - The supply of essential checking criteria for the structures of the buildings and engineering works;
 - The insurance of the technical-juridical basis of the specific building contracts and building works and engineering services;
 - The insurance of the frame necessary to set up the technical provisions for the materials and products used in the construction industry.
- Under the prerogatives of a coherent system and comprehensive in laws the EUROCODE Program supplies various design methods and several other specific design elements important in practice, covering all the building types and engineering works made of various building materials.
- Eurocodes should also rely on the reference codes of International Organization for Standardization, ISO.

The EUROCODE Program (EC) endows nine Codes as follows:

EUROCODE 1 (EC1)	Basis of design and actions on structures.
EUROCODE 2 (EC2)	Design of concrete structures (general rules, RC structures, pre cast concrete, pre stressed concrete, plain concrete).
EUROCODE 3 (EC3)	Design of steel structures
EUROCODE 4 (EC4)	Design of composite steel and concrete structures
EUROCODE 5 (EC5)	Design of timber structures
EUROCODE 6 (EC6)	Design of masonry structures
EUROCODE 7 (EC7)	Geotechnical design
EUROCODE 8 (EC8)	Design provisions for earthquake resistance of structures
EUROCODE 9 (EC9)	Design of aluminum structures

The provisions of a Eurocode are structured on items/paragraphs that might be: **basic principles** and **application rules**.

The **basic principles** are consisting of:

- General definitions and declarations that do not allow alternative;
- Demands, models and analytical methods that do not permit alternatives besides those especially stipulated.



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The **application rules** are unanimously acknowledged rules that follow the principles and fulfill their requests.

The principles are recognized after P letter denomination that follows the item number or the paragraph. Other paragraphs or items (without P) represent application rules.

An Eurocode appliance always needs the obey of the principles while the application rules having a recommendation status can be replaced, in several special cases, by rules or equivalent methods if one demonstrates that these are paying respect to the basic principles; in this way it is provided a larger flexibility for application, allowing alternatives inside the code.

In the first step of the code elaboration these were partially or entirely published as **Temporary European Norms** or **Pre-norms**, abbreviated as **ENV**. Thus, the Ist part (that usually refers to fundamental/general requirements) of EC2, EC3, EC4 and EC5 was published in ENV format in 1992. The Ist part of EC1 and EC8 was published in ENV version in 1994.

Each Eurocode or part of a Eurocode published in an ENV format must be accompanied by a National Appliance Document (DAN) in order to be able to perform the responsibility upon the basic requirements by the country membership authorities. These documents will be written and used together with the ENV experimental version of the specific European Norm ENV, having the following functions:

- the harmonization of Norms by setting several connections between the ENV and national provisions;
- the correlation of the ENV with the structural safety protection level existing in that country;
- the setting of the numerical values of the ENV for several amounts (safety coefficients, action intensity – wind, snow, earthquake, temperature variations etc.). In ENV these amounts are emphasized by including inside square brackets []. The authorities of each country will replace the square braced values with effectively applicable values at the national level;
- the set up of alternatives to the application rules paying respect to the basic principles.

The second and the last step refers to the conversion of the Pre- norms into European Codes, EN, that will become valid/compulsory in the EU membership countries, the corresponding National Codes and standards following to be retired from use after a stated period. The appliance of the EN provisions will open the market of the design companies that will be able to co- operate with the foreign building companies for projects all over Europe. It is expected the EUROCODE Norm package to become operational in 2005-2007, having the status of unique European Codes after a decision at the political level of EU.



Harmonization of the Romanian codes with the European codes. National applicable documents

4. CONSIDERATIONS CONCERNING THE HARMONIZATION OF THE ROMANIAN CODES TO THE STRUCTURAL EUROCODES. NATIONAL APPLICABLE DOCUMENTS

The responsibility for the correctly structural behavior is mainly in the charge of state. For this reason, the state responsible authorities must be involved in the conception/design activities, the building process and construction service. The state implication is materialized in the fact that the above mentioned activities must roll on according to the juridical technical prescriptions that must be obeyed unconditionally.

In our country the technical provisions regarding the constructions have been treated uniformly in the frame of the Ministry of Public Works and Land Planning, MLPAT, now Ministry of Public Works, Transportation and Dwelling, MLPTL, under the co-ordination of the General Technical Direction. There were set up the Technical Committees for the specializations that on the basis of the 10/1995 law regarding the construction quality and the corresponding Application Rules; these Committees have the following duties in order to accomplish the six requirements:

- The draw up of the technical prescriptions for the components of the quality system in constructions;
- Technical agreement for products, procedures and equipments;
- The compliance authentication for the quality of the products used in constructions.

In November 1997 MLPAT launched the Romanian Codes Program – CR for the „Draw up of the technical prescriptions concerning the civil and industrial engineering structures during the 1997-2000 and the harmonization with the technical prescriptions from EU”.

The CR Program elements were defined in principle according to the 1-9 Eurocodes, the basic idea is those to become National Applicable Documents (DAN) i.e. the content of an ENV Eurocode to be transferred entirely to the Romanian version with the adjustments corresponding to our country. Thus EC1 in the ENV format would become CR1 – Basic elements for the design (the fundamentals of design) and actions on constructions, EC2 – ENV to CR2 – Design of concrete structures and so on.

Some drawbacks in the co-ordination of the CR Program and the limit of the founding made it run discontinuously and it was not possible to follow the European evolution of the EUROCODE. From this reason instead of appearing the Romanian translation of the Eurocodes together with the corresponding DANs there were draw up several Codes that have neither the name of the chapter from the original Eurocode nor of the CR Program. For instance, in the Construction Bulletin vol. No. 19-20 from 2001 it was published the Code NP 042-2000 titled



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„The computational checking of the steel members and joints” which is in fact the ENV 1993-1-1 of EC3:”The calculus and design of steel structures”, part 1.1. „General rules and provisions for buildings”. The same ambiguous situation it is present at the computation of the structures subjected to seismic actions (EC8 and P100), the computation of the timber structures (EC5 and the NP 005-1996 and NP 019-1997 Codes) etc.

The translation of the structural Eurocodes and the draw up of the CR Codes are very important operations and must be duplicated by a large action of dissemination and explanation to make them known to the engineers that should apply or work collateral with these norms. The DANs must be send to investigation for a 2 years period and enter in application with a temporary code status, simultaneously with the existent national Codes.

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Retrofitting of unburned clay blocks (wattle) buildings damaged by earthquakes

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Summary

The paper deals with the experimental investigations on shaking table of a full-scale model of an adobe-wall building.

After tests performed on the model built according to traditional practice, the severely damaged building has been strengthened by tie-roads and reinforced jacketing and subject to strong shaking again.

The results are presented and the efficiency of strengthening is discussed [1].

KEYWORDS: retrofitting, seismic range, maximum basic acceleration, unburned clay blocks, earthquakes

1. INTRODUCTION

The main objective of the study was checking the efficiency of a retrofitting method of the structures made of unburned clay blocks masonry, with clay mortar, strongly damaged by the surface earthquakes.

This research was imposed by the necessity of fast bringing into operation of the locative fond seriously damaged by the seismic events that took place in 1991 in Romania.

2. PROPOSED SOLUTION

In order to accomplish this objective, an experimental structure at natural scale was designed, representing a building with two rooms: one having 3.90x4.80 m and another 2.70x4.80 m. The structure was conceived and tested in seismic range up to the precollapse stage, on the 140tf oscillating platform belonging to the National Institute for Building Research - Branch Iasi (photo 1).



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Photo 1. Damaged structure

For retrofitting the damaged structure a simple and efficient solution was designed, consisting in the introduction at the upper side of the masonry wall, on both directions (transversal and longitudinal), of horizontal metallic backstays, less prestressed having $\Phi 12\div 14$ mm diameter, fixed at the ends with double bolt and nut. At the same time, on both faces of the damaged diaphragms a plastering was applied, composed by mortar M 25, with 3-4 cm thickness, on a metallic mesh having the square eyes with the side 5 cm, mesh conceived by wires with $\Phi 2,5\div 3$ mm thickness.

The metallic meshes were fixed at 1 cm from the face of the walls by means of metallic anchors that traverse the diaphragms, 50 cm distance some from the others having a quasi - uniform distribution on the surface of the wall. At the intersection of the diaphragms, the welded or linked metallic meshes were joined by their overlap on 3-4 eyes distance (15-20 cm), and at the level of the foundation the meshes were connected on the same distance (3-4 eyes) in the concrete basement.



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Photo 2. Retrofitted structure

3. RESULTS OF THE EXPERIMENT

Testing the structure, in the initial stage (initial) and retrofitted, in seismic range took place using the seisms with progressive intensities and variable frequencies specifically for the surface and real earthquakes from the Banat zone – Timisoara – Romania.

The maximum parameters for the **initial structure** are:

- The maximum basic acceleration: $a_0 = 2,95 \text{ m/s}^2$.
- The amplitude of the horizontal displacement, at the level (+ 2,60 m) : $D_{\max} = + 55 \text{ mm}$
- Dynamic amplification coefficient decreases from $\beta = 1.65$ to $\beta = 0,8$, for the increases of the acceleration from the $a_0 = 0,59 \text{ m/s}^2$ to $a_0 = 2,95 \text{ m/s}^2$.
- The oscillation fundamental period in the transversal direction (the direction of the seismic action) was increased from $T_i = 0,250 \text{ s}$ in initial phase to the $T_f = 0,285 \text{ s}$ in the yield phase.
- Viscous damping %, ν was increased from the (3÷3.55)% to (5÷5.91)%.

The maximum parameters for the **consolidated structure** are:



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- The maximum basic acceleration: $a_0 = 5,7 \text{ m/s}^2$.
- The amplitude of the horizontal displacement, at the level + 2,60 m; $D_{\max} = +5,5 \text{ mm}$
- Dynamic amplification coefficient decreases from $\beta = 1.42 (a_0 = 0,78 \text{ m/s}^2)$ to $\beta = 0,795 (a_0 = 5,74 \text{ m/s}^2)$.
- The oscillation fundamental period in the transversal direction (the direction of the seismic action) was increased from $T_i = 0,133 \text{ s}$ in initial phase to the $T_f = 0,192 \text{ s}$ in the yield phase.
- Viscous damping %, ν was increased from the $(5 \div 5,80)\%$ to $(8 \div 8,44)\%$.

3. CONCLUSIONS

The wattle masonry's compound with the retrofitting plastering was good up to the limitation of the re-collapsed structure.

On the experimental data basis corroborated with theoretical investigations it can be affirmed that the retrofitting solution proposed presents a practical guide used at individual household level.

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The new approach of building analysis according to Fire Safety Engineering in Romania

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Summary

The new European approach concerning the constructions fire safety will determine major changes in Romania referring to: construction products fabrication, commercialization and utilization, construction products classification from the viewpoint of their behavior to fire action and also in the elaboration of the specific technical regulations.

As concerns the products that assure the construction safety to fire action, there are three important directions which must be followed in order to satisfy the European Union recommendations:

- *application of CE mark on the products;*
- *elaboration of Eurocodes;*
- *elaboration of a new conception in approaching the construction fire safety, integrated in the global structural analysis.*

The present paper consists in a revue of all actual and long-term measures applied in Romania, referring to the analysis of buildings to fire action, starting from the European documents, which must be respected when our country will be a part of European Union.

KEY WORDS: fire safety, fire security.

1. INTRODUCTION

There are three important directions which must be followed in order to satisfy the European Union recommendations as concerns the construction products which assure the building fire safety:

- application of CE mark on the products;
- elaboration of Eurocodes;
- elaboration of a new conception in approaching the construction fire safety, integrated in the global structural analysis.



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The first direction of action has several stages that lead to a generally agreed European conception, based on the necessary documents, elaborated some years ago, such as:

- *Directive 89/106/CEE*, that defines the general frame which harmonizes the rules referring to the free circulation of construction products in the European Union, as well as the essential construction requirements (including the essential requirement no.2, referring to their fire security). This requirement is reflected in the Romanian code *HG622/2004 (About the Conditions of Introducing on the Market the Construction Products)*;
- *Interpretative Document no.2*, which founds the European conception as concerns the construction fire security (explaining in specific terms *Directive 89*). It is transposed in the Romanian legislation through the code *Guide for the Interpretation of Construction Essential Requirements in Order to Assess the Adequate Use of Construction Products – Indicative GT 051-02 (chapter 2 – Interpretation of the Essential Requirement Fire Safety)*;
- *Guide G*, that contains proposed procedures for the optimization of the evaluation and classification systems used for the fire reaction performance of construction products (without a corresponding code in Romania).

The last two directions are referring to the design activity and their purpose is the achievement of a unitary, harmonized technical background which has to facilitate the circulation of construction design services.

2. OBJECTIVES AND STRATEGY OF THE ESSENTIAL REQUIREMENT. FIRE SECURITY

According to code [2], the constructions should be designed and carried out so that, in case of a fire, the following objectives to achieved:

- the stability of construction subjected to fire action for a certain time;
- the users protection and evacuation, taking into account the construction destination;
- limitation of material goods loss;
- limitation of fire propagation inside the building and in the neighborhood;
- protection of fire men and of other forces that contribute to people evacuation and salvation, protection of material goods, limitation and extinction of fire, fire effects removal.

The achievement of all these objectives is performed by using a *fire security strategy*, which consists mainly in fire prevention and in the limitation of its effects, as much as possible. From this perspective, the following strategic directions could be identified:

- minimization of the factors that influence the fire breaking out;



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- limitation of fire propagation from the fire room toward the surrounding spaces;
- correlation of intervention operations with the other protection measures and means.

The required level of fire security is ensured by applying different procedures and methods which could be integrated in the newer concept of *fire security engineering*.

3. FIRE ACTION

3.1 Generalities

The essential requirement *Fire Security* should be satisfied with an acceptable probability during the construction life time, reasonable from economic point of view. The fulfillment of the essential requirement is ensured by interdependent measures which are referring especially to:

- *building*: location, design, construction, service;
- *construction products*: properties, performances and utilization in the building.

The fire performances of construction and/or of construction products are considered with respect to the typical specified action.

The code *SR EN ISO 13943 – Fire Safety. Vocabulary* defines the following basic concepts of fire security:

- fire, as combustion deliberately organized in order to produce useful effects, its propagation in space and time being closely controlled;
- fire, as combustion that develops in space and time without any control and implies material damages. The interruption of its propagation presumes the intervention of specialized forces and equipments.
- fire situation, stage in the fire development characterized by the nature, severity and magnitude of thermal action upon the construction products.

As a consequence of the previous definitions it is correct to say *construction fire safety* and *occupants fire security*, or, more concise, *fire safety and security in constructions* [5].

Starting from these definitions it is correct to say *fire scenario* for the qualitative estimation of the phenomenon involved by the fire breaking out and *design fire* for the quantitative estimation of the phenomenon.

The fire action is considered in the study of construction behavior as a mechanical one, a thermal one or combination of them.



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The mechanical action may be generated by loads, imposed deformations, temperature changes.

The thermal action may be the consequence of radiation, convection and conductivity phenomena.

The level of thermal action with respect to time defines the stage of fire development and could be simulated in order to assess the performance of a product in the circumstances of its final utilization.

Unlike the old semi-empirical testing methods, the new methods are founded on *scenario* that recreate the circumstances of real fires, and are based on fundamental research in the field of fire thermodynamics. So, we can identify the *reference scenario* and the *fire scenario*.

3.2 Reference scenario and fire scenario

The **reference scenario** defines the thermodynamic state corresponding to the development of a fire in a compartment and is used to establish the representative and reproducible testing methods.

Several types of reference scenarios can be identified, depending on the purpose of the fire performance assessment.

Assessment of the performance – reaction to fire action

In this case, the reference scenario presumes the initiation of a fire in a compartment, which may develop and attain eventually the flashover point. It includes three circumstances that correspond to three distinct stages in the development of a fire:

- initiation (simulated by lighting a small fire on a limited surface of a product);
- development (simulated by a product burning in a corner of the room, generating a heat flux on the surrounding surfaces – method SBI);
- post-flashover (simulated by all combustible products burning – generalized burning).

Assessment of the performance – fire resistance

In this circumstance, the reference scenarios are based on a temperature – time curve, which defines the evolution of gas temperature near the surfaces of a structural element. They may be natural or conventional scenarios.

Reference scenarios for natural fires

The evaluation of the thermal action corresponding to a fire which develops in a closed space of a building (room, group of rooms, portion of a building), must consider:



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- the thermal load (type, quantity and burning velocity);
- the fire air supply;
- the shape and dimensions of the structural elements that form the fire compartment;
- the thermal and mechanical properties of the non-structural elements that separate the fire compartment;
- the influence of the equipments for fire extinction (as an example, the effects of the equipments with sprinklers);
- the action of fire men team (which could be facilitated by a system of fire detection).

The requirement *Fire Security* imposes the limitation of fire propagation and the assurance of construction stability for a specified period of time. This aspect can be expressed by the fire resistance of all types of building elements.

The adopted (conventional) model for the thermal action, corresponding to a generalized fire, is that one given by the *standard temperature – time curve* (taken from ISO 834):

$$T = 345 \cdot \log_{10}(8 \cdot t + 1) + 20 \quad (1)$$

where T is the gas temperature in the furnace, expressed in °C, and t the thermal exposure time, during the fire test, expressed in minutes.

This model of thermal action is used in the assessment of the performances of products exposed to a fire which is full evolution.

The adopted (conventional) model for the thermal action in case of a more severe fire (with a greater rate of temperature increasing than that given by the standard curve) is that one given by the *harmonized hydrocarbon curve*:

$$T = 1080 \cdot (1 - 0.325^{-0.167t} - 0.675^{-2.5t} + 20) \quad (2)$$

where the time, t , is expressed in minutes.

The adopted (conventional) model for the thermal action in case of a smoldering fire (with a lower rate of temperature increasing than that given by the standard curve) is that one given by the *smoldering combustion curve*:

$$T = 154 \cdot t^{0.25} + 20 \quad (3)$$

where the time, t , is expressed in minutes.



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Reference scenarios for conventional fires

In case of special constructions (road tunnels, nuclear power plants) the technical prescriptions may impose extreme fire scenarios, for which the thermal action model is given by characteristic conventional curves for such severe circumstance.

The **fire scenario** defines from quality point of view the fire evolution in time, identifying the characteristic features, which make the difference between several possible fires. This concept is connected to other concepts:

- *design fire scenario* (the fire scenario whose selection is based on the risk);
- *design fire* (the quantitative description of the features which are established for the design fire scenario).

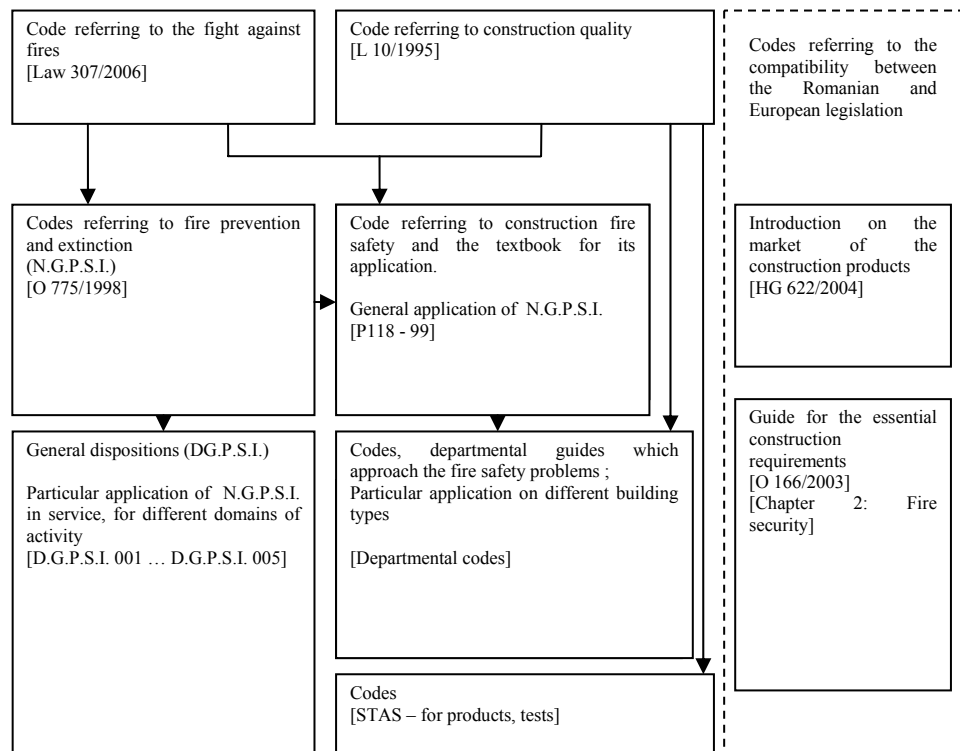


Figure 1. Romanian legislation referring to construction fire safety and security (2006)

The approach of the previously mentioned concepts is performed in a well established succession:

- identification of fire scenarios;
- selection of design fire scenarios;
- elaboration of design fires.



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The fire scenario describes the process of fire initiation, development, generalization and regression, the characteristics of the building and of the systems which influence the fire evolution.

The code *SR EN ISO 13943* defines the fire scenario as a detailed description of the conditions, including the environment, in which one or several phases of an actual fire develop.

The interaction fire – building – occupants could generate a great number of possible fires and the practical impossibility of analyzing all scenarios. Therefore, a reasonable number of representative scenarios must be selected.

At limit, a single fire scenario could be considered, if it offers enough information for the correct evaluation of the security levels needed in the design activity.

In the fire scenarios selection, the process of *fire risk* assessment and hierarchization has an important role. For each project, a number of scenarios of fire with high risk are selected which become *design fire scenarios*.

For each fire scenario, a *design fire* is established (or more, in case of fire propagation in the surrounding compartments), activity that consists in a process of fire quantification, as a result of studying its characteristics.

The model selection and use imply especially the analyst experience.

4. THE ESSENTIAL REQUIREMENT *FIRE SECURITY* CHECKING

4.1. Generalities

The requirement fire safety / fire security is expressed in the national prescriptions in three ways or combinations [2], starting from the enunciation:

- of a minimum performance requirement for the construction;
- of a minimum fire performance of the products;
- of the critical levels of toxicity that the people could be exposed to.

4.2. Eurocodes

These European codes are recognized as reference documents to prove the conformity of constructions with the *Essential Requirement 1 – Stability and Mechanical Resistance* and the *Essential Requirement 2 – Fire Security*.

Romania has adopted as codes the following European documents:

- *SR EN 1990/2004 Eurocode 1. The bases of structures design;*



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- SR ENV 1992-1-2/2004 Eurocode 2: Design of concrete structures. Part 1-2: General rules: design of structures to fire action;
- SR ENV 1993-1-2/2004 Eurocode 3: Design of steel structures. Part 1-2: General rules: design of structures to fire actions;
- SR ENV 1994-1-2/2004 Eurocode 4: Design of composite structures made of steel and concrete. Part 1-2: General rules: design of structures to fire action.

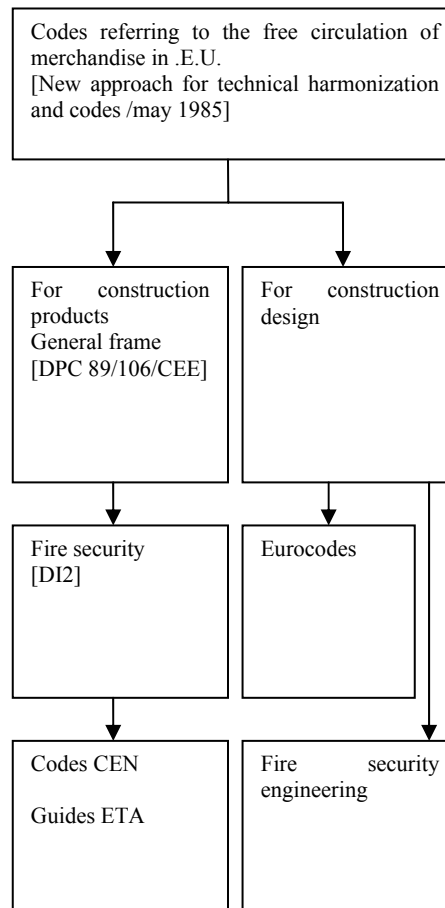


Figure 2. European legislation referring to construction fire safety and security (2006)

The procedures, methods and relations recommended by the Eurocodes can be used in the design of constructions to fire action because their results are recognized on the European market.



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5. FIRE SECURITY ENGINEERING

One of the most recent concepts that must assure the fulfillment of the essential requirement fire security of constructions is the *fire security engineering*. It consists in applying some technical, engineering principles in the assessment of the safety level and in the design of the needed security measures.

The fire security engineering refers to the *design of fire security* and the *management of fire security* its purpose being a set of adequate procedures and methods, scientifically founded. Its application must involve efficient costs for construction design and management, it must be recognized all over the world, it must assure the users security, it must reduce the damages produced by the fire to the construction and to the environment and finally, it must protect the patrimony.

The new concept is used for:

- obtaining information about the development and propagation of fires throughout the construction and the fire influence;
- the evaluation of actions (thermal, mechanical, and so on) upon the compartment and construction;
- the evaluation of construction products performances;
- the evaluation of performances for detection and extinction equipments;
- the evaluation and design of the evacuation and salvation procedures.

The fire security engineering is a concept that develops in our days and the European documents which define it are codes from *ISO 13387*.

According to the new concept, the design is based *on performance, not on prescriptions*.

6. CONCLUSIONS

At the moment, in Romania, the legislation referring to fire safety and security in constructions approaches the problems in the traditional manner (according to Law 307/2006 – *Construction Fire Safety*) and in the European manner, that is known by few Romanian designers. The Romanian legislation is shown in Figure 1 and the European legislation, in Figure 2.

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