UNIVERSITATEA TEHNICĂ CLUJ-NAPOCA

**Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)**
**Journal homepage: http://constructii.utcluj.ro/ActaCivilEng**

**ISSN 1221-5848**

ACTA TECHNICA NAPOCENSIS

**ISSN: 1221-5848**
**WWW**: http://constructii.utcluj.ro/ActaCivilEng

**Editor-In-Chief: Prof. Cosmin G. Chiorean**
*(Tech. Univ. of Cluj-Napoca, Romania)*
Phone/Fax: 40-264-594967
E-mail: cosmin.chiorean@mecon.utcluj.ro

**Affiliation to Organization:**
Technical University of Cluj-Napoca, Faculty of Civil Engineering, Romania

**Editorial Office Address:**
Technical University of Cluj-Napoca
15 C Daicoviciu Str., 400020 Cluj-Napoca, Romania

**Published by:**
UTPRESS
34 Observatorului Str., 400775 Cluj-Napoca, Romania
Phone: 40-264-401999
E-mail: utpress@biblio.utcluj.ro
Fax: 40-264-430408

**Abstracting and Indexing**
INDEX COPERNICUS JOURNALS MASTER LIST

**Aims and Scope**: *Acta Technica Napocensis:Civil Engineering & Architecture* provides a forum for scientific and technical papers to reflect the evolving needs of the civil and structural engineering communities. The scope of *Acta Technica Napocensis: Civil Engineering & Architecture* encompasses, but is not restricted to, the following areas: infrastructure engineering; earthquake engineering; structure-fluid-soil interaction; wind engineering; fire engineering; blast engineering; construction materials; structural mechanics; water resources; hydraulics and coastal engineering; structural reliability/stability; life assessment/integrity; structural health monitoring; multi-hazard engineering; structural dynamics; optimization; expert systems and neural networks; experimental modeling; performance-based design; engineering economics, constructional management; architecture; planning and built environment studies. *Acta Technica Napocensis: Civil Engineering & Architecture* also publishes review articles, short communications and discussions, book reviews, and a diary on national and international events related to any aspect of civil engineering and architecture. All articles will be indexed by the major indexing media, therefore providing maximum exposure to the published articles.

# Editorial Board

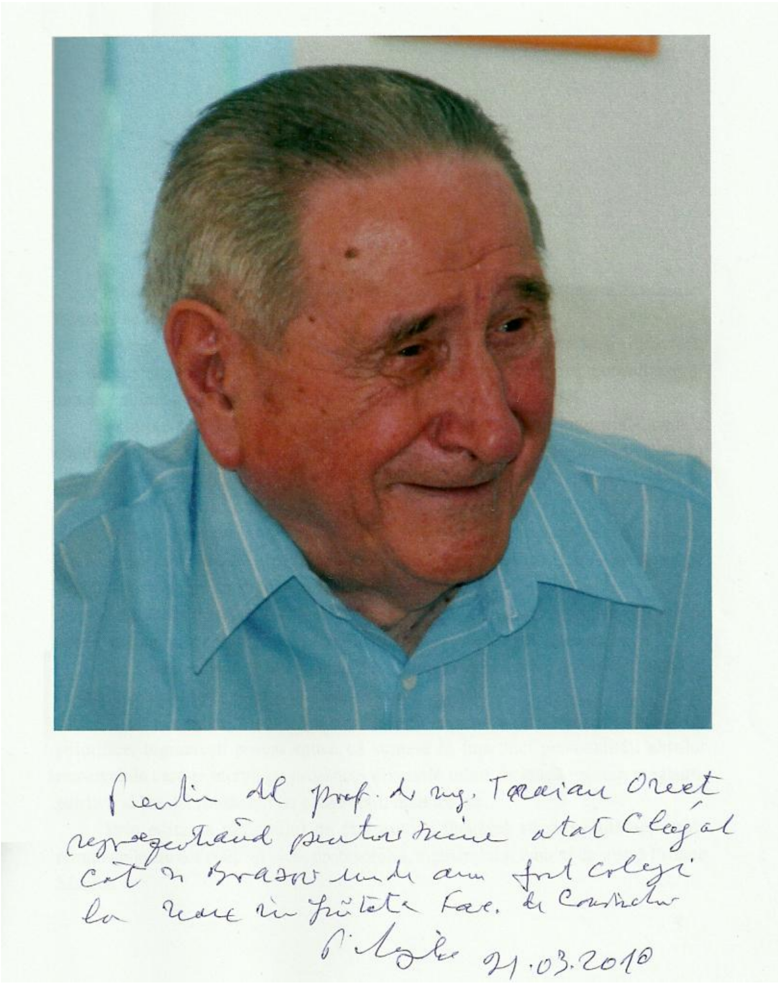
# *Contents of Acta Technica Napocensis: Civil Engineering & Architecture, Vol. 53, 2010*

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: [http://constructii.utcluj.ro/ActaCivilEng](http://constructii.utcluj.ro/ActaCivilEng)*

# OMAGIU
# Profesorului Panaite MAZILU

Prof.Dr.Ing. Traian ONEȚ*[1]
Membru Titular al Academiei de Ştiințe Tehnice din România
Doctor Honoris Causa

[1]*Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*




Am avut privilegiul să îl cunosc, în mod indirect, pe Profesorul Panaite Mazilu în anii mei de studenție la Cluj, unde am învățat disciplina „Statica Construcțiilor” după tratatul domniei sale, care era cunoscut şi utilizat în toate facultățile din țară.

Mai apoi, după ce am optat pentru cariera universitară la Institutul Polithnic din Cluj, am avut bucuria să-i cunosc toate valențele personalității sale:

- cadru didactic ilustru
- cercetător neostenit
- proiectant desăvârşit

* Corresponding author: Tel./ Fax.: 0745314242
E-mail address: traianonet@gmail.com

- constructor de prestigiu
- expert tehnic de renume
- distins orator

Îndeaproape l-am cunoscut însă pe Omul Panaite Mazilu în intervalul de timp 1993-2004 în care ne-am adus, împreună cu alte cadre didactice de prestigiu, aportul la fondarea Facultăţii de Construcţii din Braşov. Activitatea de profesori invitaţi, pe care am susţinut-o, numeroasele discuţii şi schimburi de păreri, festivităţile de absolvire, manifestările ştiinţifice organizate în Universitatea „Transilvania" din Braşov, ne-au îmbogăţit orizonturile şi ne-au apropiat, formând împreună cu ceilalţi colegi localnici un corp didactic distins şi apreciat în prezent.

Este de netăgăduit faptul că Magistrul nu este numai un întemeiator de şcoală inginerească de construcţii şi de şcoală de cercetare ci, domnia sa, este o ilustră personalitate, o adevărată instituţie.

În jurul său au răsărit, în decursul timpului, vlăstare din care s-au dezvoltat apoi iluştrii profesori, cercetători şi ingineri constructori.

La vârsta aniversară de 95 de ani, cum am putea să-l omagiem mai bine pe părintele constructorilor din România, Profesorul Panaite Mazilu, Membru de onoare al Academiei Române, decât urmărindu-i învăţăturile şi exemplul de dăruire pentru profesie.

Să fim mândri că îi suntem contemporani, să-i omagiem opera şi să-i urăm mulţi ani în sănătate şi noi impliniri de prestigiu.

Prof.dr.ing. Traian ONEŢ
M.c.Acad.STR
Doctor Honoris Causa

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# LAUDATIO
# Profesorului consultant dr.ing. Victor Gioncu
# Facultatea de Arhitectură, Universitatea ”Politehnica” Timişoara

Prof.Dr.Ing. Traian ONEŢ*[1]
Membru Titular al Academiei de Ştiinţe Tehnice din România
Doctor Honoris Causa

[1]*Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

Îmi revine onoranta şi plăcuta îndatorire să rostesc <<LAUDATIO>> profesorului Victor Gioncu, distinsă personalitate a comunităţii ştiinţifice româneşti şi internaţionale, deţinător al unei temeinice instrucţii, al unei vaste culturi, al unor înalte trăsături umane, al unei conduite academice remarcabile ş i a unei impresionante experienţe profesionale.

Este foarte greu, dacă nu imposibil, să rezumi activitatea de o viaţă printr-o astfel de temeritate. Îndrăznesc totuşi sa încerc, cunoscându-i îndeaproape trăsăturile personalităţii domniei sale şi contând pe prietenia statornicită între noi de la admiterea la doctorat sub conducerea ştiinţifică a distinsului Profesor Constantin Avram, membru corespondent al Academiei Române. Sper să mă judece cu îngăduinţă şi să îmi fie iertate eventualele scăpări sau stângăcii de exprimare.

Profesorul, inginerul constructor (executant, proiectant, cercetător) şi omul de ştiinţă Victor Gioncu, este cunoscut şi **recunoscut** în ţară prin trăsăturile sale definitorii de :

- remarcabil inginer de şantier;
- cadru didactic de prestigiu la Facultatea de Construcţii din Timişoara secţiile CCIA şi Arhitectură;
- proiectant de avangardă;

* Corresponding author: Tel./ Fax.: 0745314242
E-mail address: traianonet@gmail.com

- cercetător ştiinţific celebru la INCERC Timişoara, asociind în mod exemplar activitatea de cercetare cu cea de proiectare şi învăţământ superior;
- memebru titular al Academiei de Ştiinţe Tehnice din România;

Pentru vasta şi excelenta activitate prestată a fost :
- distins cu numeroase premii naţionale (Academia Română 1978, Convenţia Europeană pentru Structuri Metalice 1997, AGIR 2006, AICPS 2008);
- a fost primit membru în asociaţii profesionale (AICPS- membru fondator şi membru al Consiliului Naţional AGIR, SBIR- Societatea Bănăteană de Inginerie Seismică- preşedinte);
- a fost atestat pentru calitatea de verificator de proiecte şi expert MLPAT;
- a fost desemnat specialist al Ministerului Culturii Cultelor şi Patrimoniulu Naţional

Ca specialist de renume **remarcat pe plan internaţional**, profesorului Victor Gioncu i-a revenit misiunea şi onoarea să fie desemnat :
- membru al comitetelor de organizare ale conferinţelor internationale (Timisoara, Guilford, Budapesta, Copenhaga, Beijing, Venetia, Liege, Lisabona, Roma, Sidney, Kyoto, Montreal, Napoli, Yokohama, Philadelphia, Krakow, Bratislava);
- raportor general la conferinţe internaţionale (20 conferinte 1982-2010),
- profesor invitat la universităţi din străinătate (Guilford University,Rio de Janeiro University, Internaţional Advances School Budapesta, CISM Udine) pentru cursuri si conferinţe (23conferinte 1985-2007)
- organizator al unor cicluri de conferinte internaţionale:
  -Coupled Instabilities in Metal Structures
  -Behavior of Steel Structures in Seismic Areas
- organizator de cursuri speciale internationale în Italia privind:
  -SEISMIC Resistent Steel Structures,
  -Phenomenological and Mathematical Modelling in Structural Instability,

Activitatea domniei sale a fost rasplătită cu:
- premii naţionale şi internaţionale;
- acceptarea ca membru în comisii de editare a unor reviste de specialitate
- editor pentru numere speciale din reviste;

Profesorul Victor Gioncu se remarcă deasemenea printr-o vastă şi valoroasă activitate publicistică cuprinzând:
- 9 cărti;
- capitole speciale in 8 cărţi publicate;
- editor a 5 cărţi în colaborare cu personalitati din străinătate;
- autor a 5 cursuri universitare pentru studentii arhitecţi;
- publicarea a 238 articole in intervalul 1964-2010,grupate pe următoarele domenii:
  -structuri spaţiale;
  -bare cu pereti subţiri formate la rece;
  -stabilitatea structurilor;
- calculul seismic al structurilor;
- analiza,consolidarea si reconversia clădirilor şi monumentelor istorice;
- arhitectură şi structuri;

Activitatea bogată a profesorului Victor Gioncu este parţial reflectată prin 8 menţiuni in volume de personalităţi din România şi din SUA.

Exponent ilustru al activitătii de construcţii din România, magistrul numeroaselor promoţii de absolventi ingineri si arhitecți şi participant activ la viața ştiințifică internațională, profesorul Victor Gioncu merită stima si considerația noastră.

Suntem mândri pentru că îi suntem colegi şi colaboratori în profesia de constructor, si personal pentru activitatea din cadrul Academiei de Ştiințe Tehnice din România, Sectia Constructii si Urbanism.

Toate considerațiile de mai sus si multe altele ne indreptatesc asadar sa-l rugam pe Victor Gioncu sa accepte prinosul nostru de recunostinta, al colegilor clujeni, prin gestul de onoare pe care Universitatea Tehnica din Cluj-Napoca il face conferindu-i inaltul titlu stiintific de Doctor Honorius Causa.

Îi dorim viață lungă, în pace şi sănătate, noi împliniri şi satisfacții profesionale, bucurii familiale si sociale, spre lauda nobilei profesii de constructor pe care profesorul Victor Gioncu o onorează prin faptele sale, prin ținuta academică pe care o etalează si prin modelul de munca si viata pe care il ofera tinerei generatii.

Cluj-Napoca, la 4 octombrie 2010

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Steel Box Members Subjected to Compression

Petru Moga[*1], Ştefan I. Guțiu[*2], Octavian Ilea[3]

[1,2,3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*In this paper are presented two numerical examples to evaluate the compression resistance of box section members made-up by stiffened plates and a comparative analysis are performed taking into account the similar non-stiffened sections. The design procedure is performed in accordance with EN 1993-1-1. Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings and EN 1993-1-5. Eurocode 3: Design of steel structures. Part 1-5: Plated structural elements, respectively the Romanian equivalent standards.*

## Rezumat

*În lucrare se prezintă două exemple de evaluare a rezistenței la compresiune a unor elemente cu secțiune cheson, realizate din plăci plane rigidizate şi se face o analiză comparativă privind capacitatea portantă cu cea chesoanelor de aceeaşi arie a secțiunii, dar realizate din plăci nerigidizate. Calculul elementelor este efectuat în conformitate cu euronormele EN 1993-1-1. Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings şi EN 1993-1-5. Eurocode 3: Design of steel structures. Part 1-5: Plated structural elements, respectiv standardele române echivalente acestora.*

**Keywords:** Eurocode, compression resistance, stiffened box section, steel members

## 1. Basis

Compression members with a transversal box section have a large use for trusses, bridge arches and columns, due to their high flexural and torsional rigidity, respectively a high compression load carrying capacity.
In the case of the large box section walls and high plate slenderness ratio it is recommended to dispose longitudinal stiffeners and if necessary also transversal stiffeners with the scope to improve the mechanical behavior of the element, especially concern the local and overall stability and implicitly the increasing of the load carrying capacity.
The buckling design resistance is given by:

$$N_{b.Rd} = \chi \cdot A_{eff} \frac{f_y}{\gamma_{M1}} \tag{1}$$

For transversal member cross-section made-up by plates with or without longitudinal stiffeners, the effective area for the global buckling the effective areas of the walls should be accounted for.

$$A_{eff} = \sum A_{c.eff} \tag{2}$$

[*] Corresponding author: Tel./ Fax.: 40-264-401839/40-264-592072
E-mail address: petru.moga@cfdp.utcluj.ro, stefan.gutiu@cfdp.utcluj.ro

The effective area of the compression zone of the stiffened plate, Fig. 1, should be taken as ([1], [2] - § 4.5):

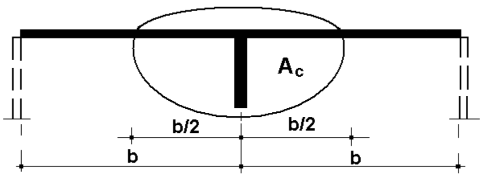
$$A_{c.eff} = \rho_c A_{c.eff.loc} + \sum b_{edge.eff} t \tag{3}$$

where $A_{c.eff.loc}$ is the effective section areas of all the stiffeners and subpanels:

$$A_{c.eff.loc} = A_{sl.eff} + \sum_c \rho_{loc} b_{c.loc} t \tag{4}$$

where: - $\sum_c$ applies to the part of the stiffened panel width;

- $A_{sl.eff}$ is the sum of the effective section of all longitudinal stiffeners;
- $b_{c.loc}$ is the width of the compressed part of each subpanel;
- $\rho_{loc}$ is the reduction factor for each subpanel.



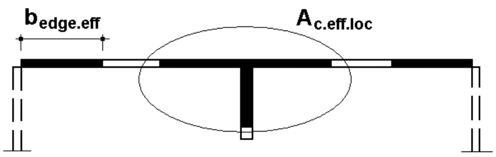




Figure 1. The effective area of the compression zone of the stiffened plate

In determining the reduction factor $\rho_c$ for overall buckling, the reduction factor for column-type buckling which is more severe than the reduction factor for plate buckling, should be considered. Interpolation should be carried out between the reduction factor $\rho$ for plate buckling and the reduction factor $\chi_c$ for column buckling to determine $\rho_c$.

$$\rho_c = (\rho - \chi_c)\xi(2-\xi) + \chi_c \tag{5}$$

where: $\xi = \dfrac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1; \quad 0 \le \xi \le 1$

In the case of a stiffened plate with one longitudinal stiffener , the elastic critical buckling stress can be calculated with ( [3],[4] – Annex A, § A.2.2):

$$\sigma_{cr.sl} = \begin{cases} \dfrac{1.05\ E}{A_{sl.1}} \dfrac{\sqrt{I_{sl.1}\ t^3\ b}}{b_1\ b_2} & -\text{if} : a \ge a_c \\ \dfrac{\pi^2 E\ I_{sl.1}}{A_{sl.1} a^2} + \dfrac{E\ t^3 b a^2}{4\pi^2 (1-\upsilon^2) A_{sl.1}\ b_1^2\ b_2^2} & -\text{if} : a < a_c \end{cases} \tag{6}$$

with: $a_c = 4.33 \sqrt[4]{\dfrac{I_{sl.1}\ b_1^2\ b_2^2}{t^3\ b}}$

$A_{sl.1}$; $I_{sl.1}$ – are the gross area and the second moment of area of the column.

For stiffened plates with at least three equally spaced longitudinal stiffeners the plate buckling coefficient may be approximated by ( [1],[2] – Annex A, § A.1):

$$k_{\sigma.p} = \begin{cases} \dfrac{2\left[(1+\alpha^2)^2 + \gamma - 1\right]}{\alpha^2(\psi+1)(1+\delta)} & \text{if} \quad \alpha \le \sqrt[4]{\gamma} \\ \dfrac{4\left(1+\sqrt{\gamma}\right)}{(\psi+1)(1+\delta)} & \text{if} \quad \alpha > \sqrt[4]{\gamma} \end{cases} \tag{7}$$

with: $\psi = \dfrac{\sigma_2}{\sigma_1} \ge 0.5$; $\gamma = \dfrac{I_{sl}}{I_p}$; $\delta = \dfrac{\sum A_{sl}}{A_p}$; $\alpha = \dfrac{a}{b} \ge 0.5$

where:

$I_{sl}$ is the sum of the second moment of area of the whole stiffened plate;

$I_p = \frac{bt^3}{12(1-\upsilon^2)} = \frac{bt^3}{10.92}$ is the second moment of area for bending of the plate;

$\sum A_{sl}$ is the sum of the gross area of the individual longitudinal stiffeners;

$A_p$=b t is the gross area of the plate;

$\sigma_1$ is the larger edge stress;

$\sigma_2$ is the smaller edge stress.

## 2. Working example 1

It is evaluated the design buckling resistance of a uniform compressed member, Fig. 2, taking into account the local buckling of the stiffened plates and the overall buckling.
Material and design characteristics of gross section:

Steel: S 355

$L = L_{cr.y} = L_{cr.z} = L_T = 15.0\ \text{m}$

$A = 1100\ \text{cm}^2;\quad I_y = I_z = 3.822 \cdot 10^6\ \text{cm}^4;$

$I_\omega = 1.817 \cdot 10^6\ \text{cm}^6;\quad i_y = i_z = 58.9\ \text{cm}.$

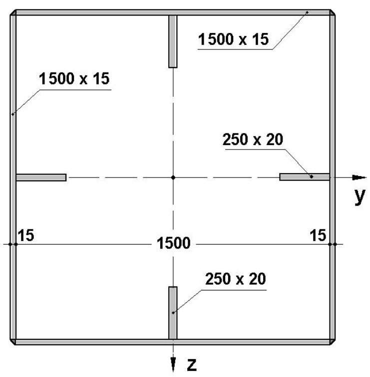



Figure 2

### *2.1 Effective area of a stiffened panel*

### *2.1.1 Adjacent plates of stiffener*

Each plate is an internal uniform compression element ($\psi = 1;\ k_\sigma = 4$).

The relative slenderness: $\bar{\lambda}_p = \frac{b_p / t}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = \frac{740/15}{28.4 \cdot 0.81\sqrt{4}} = 1.072 > 0.673$

The reduction factor: $\rho_{loc} = \frac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^2} = \frac{1.072 - 0.22}{1.072^2} = 0.74$

The effective width: $b_{eff} = \rho_{loc} \cdot b_p = 548\ \text{mm} \Rightarrow b_{edge.eff} = b_{eff}/2 = 274\ \text{mm}$

### *2.1.2. Stiffener*

The plate is an outstand compression element ($\psi = 1;\ k_\sigma = 0.43$).

The relative slenderness: $\bar{\lambda}_p = \dfrac{b_p / t}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = \dfrac{250/20}{28.4 \cdot 0.81\sqrt{0.43}} = 0.828 > 0.748$

The reduction factor: $\rho_{loc} = \dfrac{\bar{\lambda}_p - 0.188}{\bar{\lambda}_p^2} = \dfrac{0.828 - 0.188}{0.828^2} = 0.93$

The effective width: $b_{eff} = \rho_{loc} \cdot b_p = 0.93 \cdot 250 = 232$ mm

In Fig. 3 is presented the effective cross-section of the stiffened panel.

Effective area: $A_{c.eff.loc} = A_{sl.eff} + \sum_c \rho_{loc} b_{c.loc} t = 2 \cdot 23.2 + 1.5\,(2 + 2 \cdot 27.4) = 131.6\ \mathrm{cm}^2$

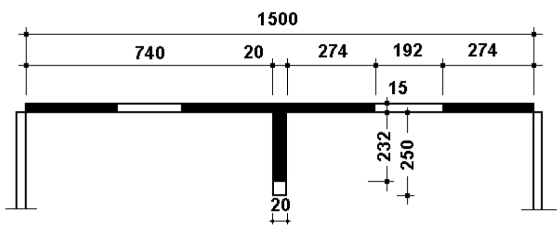



Figure 3. Effective cross-section of the stiffened panel

### *2.1.3.Reduction factor for overall buckling*

*a) Plate type behaviour ( [1],[2] - § 4.5.2)*

The relative plate slenderness $\bar{\lambda}_p$ of the equivalent plate is defined as:

$$\bar{\lambda}_p = \sqrt{\frac{\beta_{A.c} f_y}{\sigma_{cr.p}}} = \sqrt{\frac{0.80 \cdot 3550}{5024}} = 0.75 > 0.673 \Rightarrow \rho = \frac{0.75 - 0.22}{0.75^2} = 0.94$$

where: $-\beta_{A.c} = \dfrac{A_{c.eff.loc}}{A_c} = \dfrac{131.6}{164} = 0.80;\ A_c = 164\ \mathrm{cm}^2$

With the dimensions showed in Fig. 4 the following numerical results are obtained:

$b_1 = b_2 = 750$ mm, $A_{sl.1} = 164\ \mathrm{cm}^2$, $I_{sl.1} = 8727\ \mathrm{cm}^4$, i=7.3 cm,

$$a_c = 4.33 \sqrt[4]{\frac{8727 \cdot 75^2 \cdot 75^2}{1.5^3 \cdot 150}} = 662\ \mathrm{cm},\ a = 1500\ \mathrm{cm} > a_c = 662\ \mathrm{cm}.$$

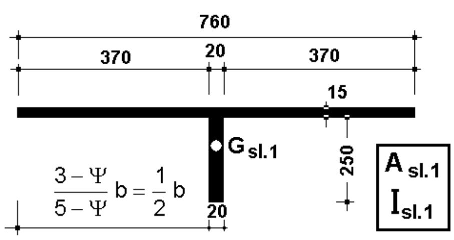



Figure 4

It results: $\sigma_{cr.p} = \sigma_{cr.sl} = \dfrac{1.05 \cdot 2.1 \cdot 10^6}{164} \dfrac{\sqrt{8727 \cdot 1.5^3 \cdot 150}}{75 \cdot 75} = 5024\ \mathrm{daN/cm}^2$

*b) Column type buckling behaviour ( [1],[2] - § 4.5.3)*

For a stiffened plate the elastic critical column buckling stress is:

$$\sigma_{cr.c} = \frac{\pi^2 \; E\, I_{sl.1}}{A_{sl.1} \cdot a^2} = \frac{\pi^2 2.1 \cdot 10^6 \cdot 8727}{164 \cdot 1500^2} = 490 \text{ daN/cm}^2$$

The relative column slenderness: $\bar{\lambda}_c = \sqrt{\frac{\beta_{A.c} f_y}{\sigma_{cr.c}}} = \sqrt{\frac{0.80 \cdot 3550}{490}} = 2.41$,

where: $\beta_{A.c} = \frac{A_{sl.1.eff}}{A_{sl.1}} = 0.80$ (Fig. 5); $A_{sl.1.eff} = 131.6 \text{ cm}^2 = A_{c.eff.loc}$

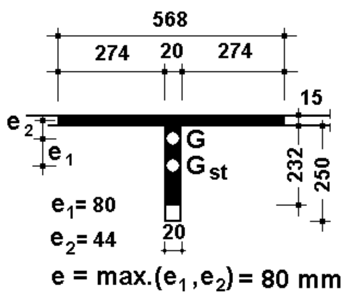



Figure 5

The reduction factor $\chi_c$ should be obtained in accordance with [3]. For stiffed plates the value of $\alpha$ should be increased to: $\alpha = \alpha' + \frac{0.09}{i/e} = 0.49 + \frac{0.09}{7.3/8} = 0.59$, where: $e = \max(e_1; e_2)$ and $\alpha' = 0.49$ for open section stiffener. It results: $\chi_c = 0.136$, with: $\phi = 4.06$.

### 2.1.4. Reduction factor $\rho_c$

The coefficient $\xi$ : $\xi = \frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1 = \frac{5024}{490} - 1 = 9.25 > 1 \quad \Rightarrow \xi = 1$

$$\rho_c = (\rho - \chi_c)\,\xi\,(2 - \xi) + \chi_c = (0.94 - 0.136) \cdot 1 \cdot (2-1) + 0.136 = 0.94 = \rho$$

The effective area of the compressed plate (wall) will be:

$$A_{c.eff} = \rho_c A_{c.eff.loc} + \sum b_{edge.eff} \cdot t = 0.94 \cdot 131.6 + 1.5 \cdot 2 \cdot 27.4 = 206 \text{ cm}^2$$

The effective area of the whole cross-section: $A_{eff} = \sum A_{c.eff} = 4 \cdot 206 = 824 \text{ cm}^2$

### 2.2. Buckling resistance of member ([3], [4] - § 6.3)

The non-dimensional slenderness: $\bar{\lambda} = \frac{\lambda}{\lambda_1}\sqrt{\beta_A} = \frac{25.47}{76.06}\sqrt{0.75} = 0.29$

where: $\lambda = \frac{L_{cr}}{i} = 25.47$; $\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9 \cdot \varepsilon = 76.06$; $\beta_A = \frac{A_{eff}}{A} = 0.75$

It is obtained: $\chi = 0.807$, with: $\phi = 0.653$

The buckling resistance of the compression member will be:

$$N_{b.Rd} = \chi \cdot A_{eff} \frac{f_y}{\gamma_{M1}} = 0.807 \cdot 824 \frac{3550}{1.1} 10^{-2} = 21\,460 \text{ kN}$$

### *2.3. Comparative analysis*

The resistance of a similar section but with unstiffened plate, Fig. 6, is comparatively analyzed. The gross areas of the two cross-sections of members are approximately equal.
$A = 1080 \text{ cm}^2$; $I_y = I_z = 4.123 \cdot 10^6 \text{cm}^4$; $i_y = i_z = 61.8 \text{ cm}$.

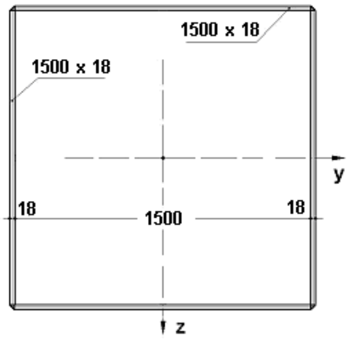



Figure 6

#### *2.3.1.Effective cross-section*

Each plate is an internal uniform compression element ($\psi = 1$; $k_\sigma = 4$).

$$\frac{c}{t} = \frac{b_p}{t} = \frac{1500}{18} = 83.3 > 42 \cdot \varepsilon = 34.02 \Rightarrow \text{Class 4}$$

The relative slenderness: $\bar{\lambda}_p = \dfrac{b_p / t}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = \dfrac{83.3}{28.4 \cdot 0.81\sqrt{4}} = 1.81 > 0.673$

The reduction factor: $\rho = \dfrac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^2} = \dfrac{1.81 - 0.22}{1.81^2} = 0.485$

The effective plate width: $b_{eff} = \rho_{loc} \cdot b_p = 728 \text{ mm} \Rightarrow b_{edge.eff} = b_{1.eff} = b_{eff} / 2 = 364 \text{ mm}$.

Figure 7 presents the effective cross-section. $A_{eff} = 524 \text{ cm}^2$; $\beta_A = 0.485$

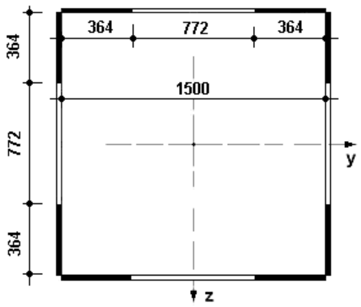



Figure 7

#### *2.3.2.Buckling resistance of unstiffened member*

The non-dimensional slenderness: $\bar{\lambda} = \frac{\lambda}{\lambda_1}\sqrt{\beta_A} = \frac{1500/61.8}{76.06}\sqrt{0.485} = 0.22$

It is obtained: $\chi = 0.99$, with: $\phi = 0.53$

The buckling resistance of the compression member will be:

$$N_{b.Rd} = \chi \cdot A_{eff}\frac{f_y}{\gamma_{M1}} = 0.99 \cdot 524\frac{3550}{1.1}10^{-2} = 16\,742\ \mathrm{kN}$$

The efficiency grade relative to buckling resistance of the stiffened box section comparative with the unstiffened section is given by the ratio:

$$E = \frac{N_{b.Rd}^{\text{cheson rig.}}}{N_{b.Rd}^{\text{cheson nerig.}}} = \frac{21\,460}{16\,742} = 1.28 = 28\ \%$$

## 3. Working example 2

The design buckling resistance of a uniform compressed member is evaluated, Fig. 8, taking into account the local buckling of the stiffened plates and the overall buckling.

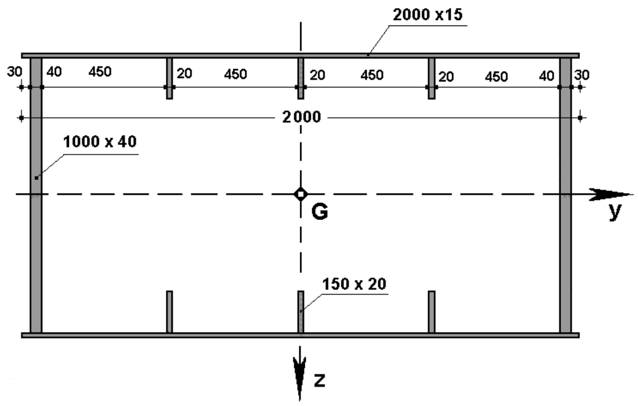



Figure 8

Steel: S 355; Critical lengths: $L_{cr.y} = 30\ \mathrm{m}$, $L_{cr.z} = 70\ \mathrm{m}$; Internal diaphragms spaced to 10 m.

Gross section characteristics: A = 1580 cm$^2$, $I_y = 2.541 \cdot 10^6\,\mathrm{cm}^4$, $I_z = 9.486 \cdot 10^6\,\mathrm{cm}^4$,

$i_y = 40.1\ \mathrm{cm}$, $i_z = 77.5\ \mathrm{cm}$

The effective cross-section area will be the effective areas sum of the four walls: $A_{eff} = \sum A_{c.eff}$

### *3.1. Effective area of a stiffened wall (Fig.9)*

#### *3.1.1. Adjacent plates of stiffener*

Each plate is an internal uniform compression element ($\psi = 1;\ k_\sigma = 4$).

$$33 \cdot \varepsilon = 26.73 < \frac{c}{t} = \frac{450}{15} = 30 < 38 \cdot \varepsilon = 30.78 \Rightarrow \text{Class 2} \Rightarrow \rho_{loc} = 1$$

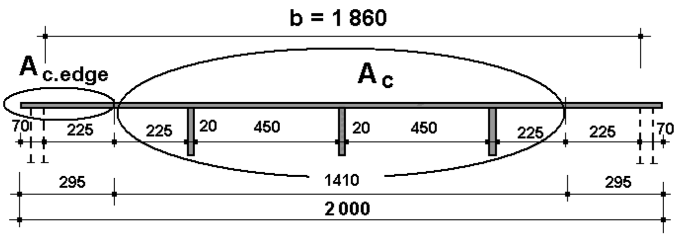



Figure 9

### *3.1.2. Stiffener*

Each plate of the stiffener is an outstand compression element .

$$9 \cdot \varepsilon = 7.29 < \frac{c}{t} = \frac{150}{20} = 7.5 < 10 \cdot \varepsilon = 0.81 \Rightarrow \text{stiffener is Class 2}$$

The stiffened panel with three longitudinal stiffeners is Class 2.
The effective area of the stiffened panel is the same with the gross area:

$$A_{c.eff.loc} = A_c = A_{sl} + \sum_c b_c t = 3 \cdot 15 \cdot 2 + (3 \cdot 45 + 3 \cdot 2) \cdot 1.5 = 301.5 \text{ cm}^2$$

$$A_{c.eff.edges} = A_{c.edges} = 2 \,\text{x}\, 29.5 \cdot 1.5 = 88.5 \text{ cm}^2$$

$$A_{c.eff} = \rho_c A_{c.eff.\,loc} + A_{c.eff.edges} = \rho_c \cdot 301.5 + 88.5 \quad [\text{cm}^2]$$

### *3.1.3.Reduction factor for overall buckling*

a) Plate type behavior
With the dimensions showed in Fig. 10 the following numerical results are obtained:

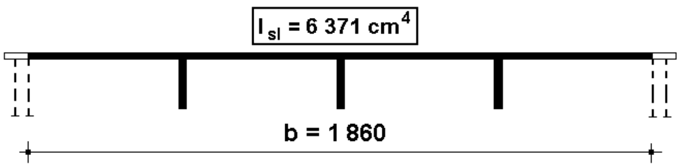



Figure 10

$$\beta_{A.c} = \frac{A_{c.eff.loc}}{A_c} = 1.0 \,;\ \psi = 1 \,;\quad \alpha = \frac{a}{b} = \frac{1000}{186} = 5.376 \,;\ a = \text{diaphragms spacing}$$

$$I_p = 57.48 \text{ cm}^4 \,;\ \gamma = 110.84 \,;\ \delta = 0.322 \,;$$

$$\alpha = 5.376 > \sqrt[4]{\gamma} = \sqrt[4]{110.84} = 3.24 \,;\quad k_{\sigma.p} = 17.44$$

The elastic critical plate buckling stress of the equivalent orthotropic plate is:

$$\sigma_{cr.p} = k_{\sigma.p} \cdot \sigma_E = 17.44 \cdot 1.9 \cdot 10^6 \left( \frac{15}{1860} \right)^2 = 2155 \text{ daN/cm}^2$$

It is obtained: $\bar{\lambda}_p = \sqrt{\dfrac{\beta_{A.c} f_y}{\sigma_{cr.p}}} = 1.28 > 0.673 \Rightarrow \rho = \dfrac{1.28 - 0.22}{1.28^2} = 0.65$

b) Column type buckling behavior
For a stiffened plate the elastic critical column buckling stress is:

$$\sigma_{cr.c} = \frac{\pi^2 \, E \, I_{sl.1}}{A_{sl.1} \cdot a^2} = \frac{\pi^2 \, 2.1 \cdot 10^6 \cdot 2008}{100.5 \cdot 1000^2} = 414 \text{ daN/cm}^2$$

The relative column slenderness will be: $\bar{\lambda}_c = \sqrt{\frac{\beta_{A.c} f_y}{\sigma_{cr.c}}} = \sqrt{\frac{1 \cdot 3550}{414}} = 2.93$,

where: $\beta_{A.c} = \frac{A_{sl.1.eff}}{A_{sl.1}} = 1.0$ (Fig. 11); $I_{sl.1} = 2008\ cm^4$; $i = 4.5\ cm$; $A_{sl.1.eff} = 100.50\ cm^2 = A_{sl.1}$

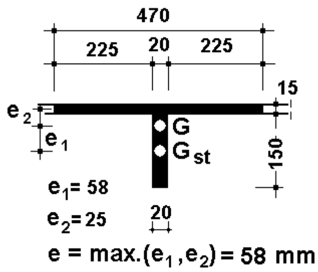



Figure 11

For stiffed plates the value of $\alpha$ should be increased to: $\alpha = \alpha' + \frac{0.09}{i/e} = 0.49 + \frac{0.09}{4.5/5.8} = 0.61$

where: $e = \max\ (e_1; e_2)$ - Fig. 11; $\alpha' = 0.49$ - for open section stiffener.

The reduction factor: $\chi_c \cong 0.1$, with: $\phi = 5.62$

### *3.1.4. The reduction factor* $\rho_c$

$$\xi = \frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1 = \frac{2155}{414} - 1 = 4.2 > 1 \quad \Rightarrow \xi = 1;$$

$$\rho_c = (\rho - \chi_c)\,\xi\,(2 - \xi) + \chi_c = (0.65 - 0.1) \cdot 1 \cdot (2 - 1) + 0.1 = 0.65 = \rho$$

The effective area of the stiffened panel is:

$$A_{c.eff} = \rho_c A_{c.eff.\,loc} + A_{c.eff.edges} = 0.65 \cdot 301.5 + 88.5 = 284.5\ cm^2$$

Effective area of the unstiffened walls:

Panel Class: $\frac{c}{t} = 25 < 33 \cdot \varepsilon = 26.73 \quad \Rightarrow$ Class 1; $A_{c.eff} = A_c = 400\ cm^2$

$$A_{eff} = \sum A_{c.eff} = 1369\ cm^2$$

### *3.2. Buckling resistance of member*

The member slenderness ratio:

$$\lambda = \max.\left\{ \lambda_y = \frac{L_{cr.y}}{i_y} = \frac{3000}{40.1} = 74.8; \lambda_z = \frac{L_{cr.z}}{i_z} = \frac{7000}{77.5} = 90.3 \right\} = 90.3$$

The non-dimensional slenderness: $\bar{\lambda} = \frac{\lambda}{\lambda_1}\sqrt{\beta_A} = \frac{90.3}{76.06}\sqrt{0.87} = 1.11$

where: $\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9 \cdot \varepsilon = 76.06$; $\beta_A = \frac{A_{eff}}{A} = \frac{1369}{1580} = 0.87$

It results: $\chi = 0.53$, with: $\phi = 1.27$

The buckling resistance of the compression member will be:

$$N_{b.Rd} = \chi \cdot A_{eff} \frac{f_y}{\gamma_{M1}} = 0.53 \cdot 1369 \frac{3550}{1.1} 10^{-2} = 23\,416 \text{ kN}$$

### *3.3. Comparative analysis*

The resistance of a similar section but with unstiffened plate, Fig. 12, is comparatively analyzed. The gross areas of the two cross-sections of members are equal.

$A = 1580 \text{ cm}^2$; $I_y = 2.696 \cdot 10^6 \text{cm}^4$; $I_z = 9.693 \cdot 10^6 \text{cm}^4$; $i_y = 41.3 \text{ cm}$; $i_z = 78.3 \text{ cm}$

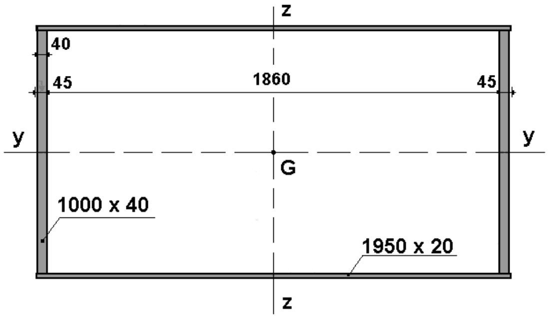



Figure 12

#### *3.3.1.Effective cross-section area*

Horizontal walls: each plate is an internal uniform compression element.

$$\frac{c}{t} = \frac{b_p}{t} = \frac{1860}{20} = 93 > 42 \cdot \varepsilon = 34.02 \Rightarrow \text{Class 4}$$

The relative plate slenderness: $\bar{\lambda}_p = \dfrac{b_p / t}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = \dfrac{93}{28.4 \cdot 0.81\sqrt{4}} = 2.02 > 0.673$

The reduction factor: $\rho = \dfrac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^2} = \dfrac{2.02 - 0.22}{2.02^2} = 0.44$

The effective width: $b_{eff} = \rho_{loc} \cdot b_p = 0.44 \cdot 1860 = 818 \text{ mm} \Rightarrow b_{edge.eff} = b_{eff} / 2 = 409 \text{ mm}$

The vertical walls are Class 1.

The Fig. 13 presents the effective area: $A_{eff} = 1072 \text{ cm}^2$; $\beta_A = 0.68$

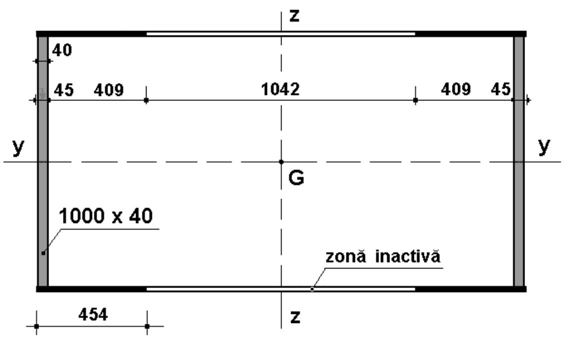



Figure 13

#### *3.3.2. Buckling resistance of unstiffened member*

The member slenderness ratio:

$$\lambda = \max.\begin{Bmatrix} \lambda_y = \dfrac{L_{cr.y}}{i_y} = \dfrac{3000}{41.3} = 72.6 \\ \lambda_z = \dfrac{L_{cr.z}}{i_z} = \dfrac{7000}{78.3} = 89.4 \end{Bmatrix} = 89.4$$

The non-dimensional slenderness: $\bar{\lambda} = \dfrac{\lambda}{\lambda_1}\sqrt{\beta_A} = \dfrac{89.4}{76.06}\sqrt{0.68} = 0.97$

It is obtained: $\chi = \dfrac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \dfrac{1}{1.10 + \sqrt{1.10^2 - 0.97^2}} = 0.62$

with: $\phi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0.5\,[1 + 0.34\,(0.97 - 0.2) + 0.97^2] = 1.10$

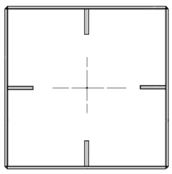
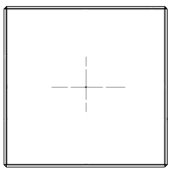
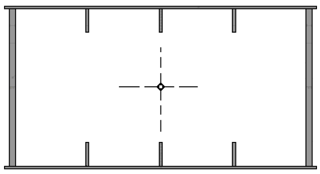
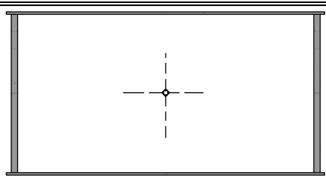
The buckling resistance of the compression member will be:

$$N_{b.Rd} = \chi \cdot A_{eff}\,\frac{f_y}{\gamma_{M1}} = 0.62 \cdot 1072\,\frac{3550}{1.1}\,10^{-2} = 21\,450\ \text{kN}$$

The efficiency grade relative to buckling resistance of the stiffened box section comparative with the unstiffened section is given by the ratio:

$$E = \frac{N_{b.Rd}^{\text{cheson rig.}}}{N_{b.Rd}^{\text{cheson nerig.}}} = \frac{23\,416}{21\,450} = 1.09 = 9\ \%$$

Table 1. The values of the parameters $\beta_A$ and E

| TYPE | CROSS SECTION | $\beta_A$ | E |
|---|---|---|---|
| $C_1$ |  | 0.75 | 1.28 |
| $C_2$ |  | 0.48 | |
| $C_3$ |  | 1.00 | 1.09 |
| $C_4$ |  | 0.68 | |

## 4. Conclusions and comments

The compression members of box cross-section type are largely used in the steel construction domain due to their high flexural and torsion rigidity.

In case of the large box section walls and high plate slenderness ratio it is recommended to dispose longitudinal stiffeners (and eventually also transversal stiffeners) with the scope to improve the element mechanical behavior by increasing the effective area of the cross-section, avoiding the local and overall buckling and implicitly to increase the load carrying capacity of member.

The values of the parameters $\beta_A$ and E for the analyzed elements are presented synthetically in Table 1, where: $\beta_A = \dfrac{A_{eff}}{A}$ and $E = \dfrac{N_{b.Rd}^{stiffened.}}{N_{b.Rd}^{unstiffened}}$

## 5. References


[1] *EN 1993-1-5. Eurocode 3*: Design of steel structures. Part 1-5: Plated structural elements

[2] *SR EN 1993-1-5.2008. Eurocod 3*: Proiectarea structurilor de oțel. Partea 1-5: Elemente structurale din plăci plane solicitate în planul lor

[3] *EN 1993-1-1. Eurocode 3*: Design of steel structures. Part 1-1: General rules and rules for buildings

[4] *SR EN 1993-1-1.2006. Eurocod 3*: Proiectarea structurilor de oțel. Partea 1-1: Reguli generale şi reguli pentru clădiri

[5] *EN 1993-2. Eurocode 3*: Design of steel structures. Part 2: Steel Bridges

[6] *SR EN 1993-2. 2007. Eurocod 3*: Proiectarea structurilor de oțel. Partea 2: Poduri de oțel

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Studies on Composite Steel Concrete Columns

CAMPIAN Cristina[1], SAV Vladut[2] and HAUPT – KARP Alina[3]

[1,2,3.] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 28 Memorandumului Str., 400114, Cluj-Napoca, Romania*

## Abstract

*The paper describes experimental tests on small-scale and full scale composite concrete encased steel columns, made at the Technical University of Cluj, Central Laboratory of Civil Engineering Faculty, as well as at laboratories of universities abroad (taken from technical papers ). The experimental results were discussed using several parameters considered appropriated, from the point of view of different design codes.*

## Rezumat

*In acest material sunt prezentate încercări experimentale pe stâlpi cu secțiune mixta oțel beton, realizate in Laboratorul Central al Facultății de Construcții din cadrul Universității Tehnice din Cluj Napoca, cât şi in laboratoarele altor instituții de cercetare (teste preluate din literatura de specialitate). Rezultatele testelor sunt comparate având ca punct de plecare anumiți parametrii considerați corespunzători din punctul de vedere a câtorva dintre normativele de proiectare existente.*

**Keywords:** Composite members, encased steel cross section, axial loading, bending loading

## 1. Introduction

On the one hand Romania is well-known as a country of high level of seismicity; on the other hand steel-concrete composite structures appear competitive today in this country by comparison with other types of structures, for example concrete structures.

Unfortunately, though the seismic performances of composite structures are generally recognized as efficient, the existing design codes are rather recent and may lack of technical background for their detailed rules (from the point of view not only of scientific knowledge but also of data base for real composite structures).

Besides, only for the design of sway composite columns under monotonic loads, it has been necessary to wait for the issue of the EN version of Eurocode 4-Part 1-1 (CEN-EUROCODE 4, 2002); so, this code gives a simplified design method, but based on second order linear elastic analysis. As a reminder, the ENV version (CEN-EUROCODE 4, 1992) was limited to the case of isolated non-sway columns.

The type of composite column investigated in the present paper is of type steel section fully encased by reinforced concrete. The strengthening of the column due to concrete encasement is clear when the column is subject to static loading, even still here some questions may appear; but under cyclic repeated loads, the deterioration of the concrete encasement may be very marked and rapid in the column critical zones so that the usual static properties of the column, namely the flexural stiffness, the ultimate resistance and the ductility may cease to be significant for a safe design. It can be remark that the load carrying capacity is reduced with increased slenderness ratio and eccentricity. Concrete strength has no obvious influence on eccentrically loaded columns.

Several studies were made on different laboratories on the aim of a better characterization of the columns behavior.

## 2. Slender columns with eccentric monotonic loading

Steel reinforced concrete structures have become a very practical and effective structural pattern for tall buildings around the world, especially in seismic active zones areas, because they provides the required stiffness to limit the lateral drift of the building to the acceptable level, and to resist the lateral seismic and wind loads very effectively. The introduction of steel rolled shapes and high strength concrete has made it possible to design columns of large slenderness.

The structural performance of slender steel reinforced concrete composite columns has recently become a major concern for design engineers.

The behavior of slender steel reinforced concrete composite columns is not yet fully understood. ACI318-05 and Chinese standard specify a moment magnifier approach for designing the slender composite columns. This approach is dependent on the effective flexural stiffness of the columns that varies due to cracking, creep, nonlinearity of the concrete, slenderness of column, and eccentricity of the axial load. The ACI equations developed for reinforced concrete columns subjected to the high axial loads were modified without any further investigation for using in steel reinforced concrete column designs.

The tests carried out by Structural Engineering Laboratory at Inner Mongolia University of Science and Technology were made on 10 slender composite columns, tested under eccentric loading conditions, for studying the effect of concrete strength, slenderness and eccentricity of the axial load. (Zhao, G *et al.*, 2009).

The geometry in transversal section is shown in the Fig. 1. The length was 2.6m, 3.2 m and 4.1 m. All the arrangements on concrete cover, diameter and distance for stirrups are in the limits of the composite standards (including Eurocode 4).

The columns were hinged at both ends and were applied with an axial or an eccentric load at both ends. The failure occurred due to the fact that compressive concrete spalled. The column failure location varied from mid-height to an extreme of 500 mm below or above mid height.The loss of the protecting cover combined with a high load level finally led to buckling of the flange of steel sections at the end of the tests. The tension flange remained elastic in specimens before maximum load, and then yielding occurred only at maximum load.

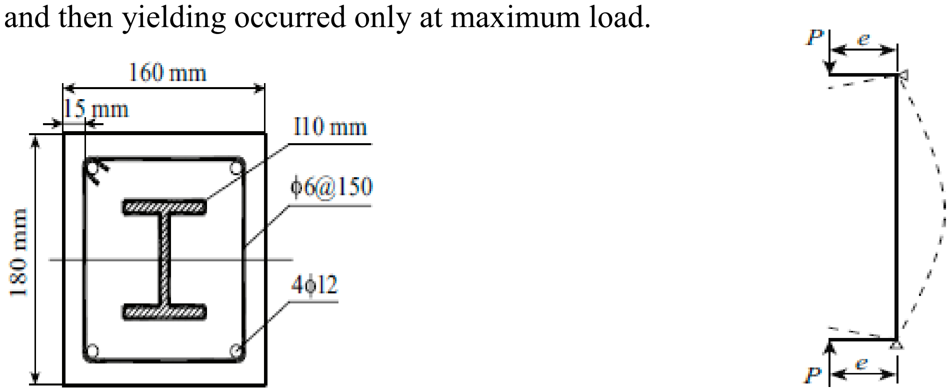



Figure 1. Geometry and details of configuration

Measured axial load versus mid-height deflection curves shown that when the load is approx. 90% the maximum load, the flexural stifness reduces rapidly, as for a force under that value, the deflections lineary increase (as shwon in Fig. 2, for different slenderness ratio).

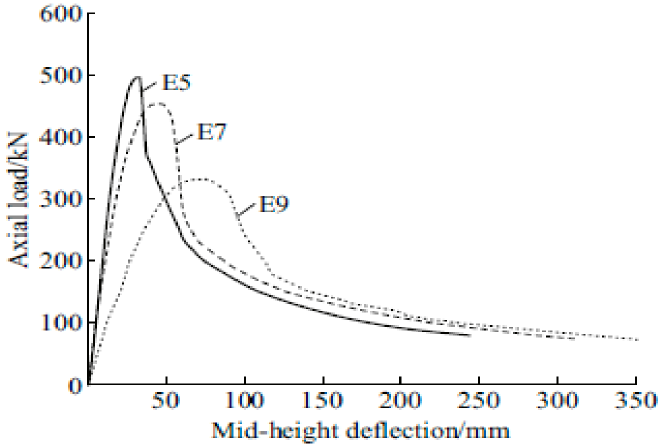



Figure 2. Measured axial load versus mid-height deflection

## 3. Monotonic and strong axis cyclic loading

The experimental program held at the Civil Engineering Faculty Cluj-Napoca, Romania, dealt with 12 columns with the same cross-section. The testing procedure was the one recommended by ECCS for characterizing the behavior of steel elements with respect to seismic action (ECCS-TWG 1.3, 1986 – using a monotonic test to calibrate the cyclic tests).

The elements were fabricated from a Romanian steel section I12 (which is quasi similar to IPE 120 section) fully covered with reinforced concrete including reinforcement. The mechanical model is that of a cantilever element. The element is subject to an axial force N in compression and to a transverse variable force H located at the free top. It may be noted that such an arrangement is similar to the one of a half-column between two floors of a building when the flexural stiffness of the floor is high enough. To ensure a suitable full restraining at the column base, the elements were ended by a sudden cross-section enlargement acting as a foundation (the flexural stiffness ratio between the element and the so-realized foundation was about 1/5). Three lengths of 2.00, 2.50 and 3.00 respectively were mould.

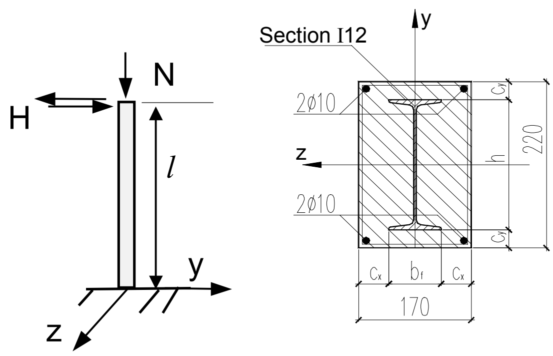
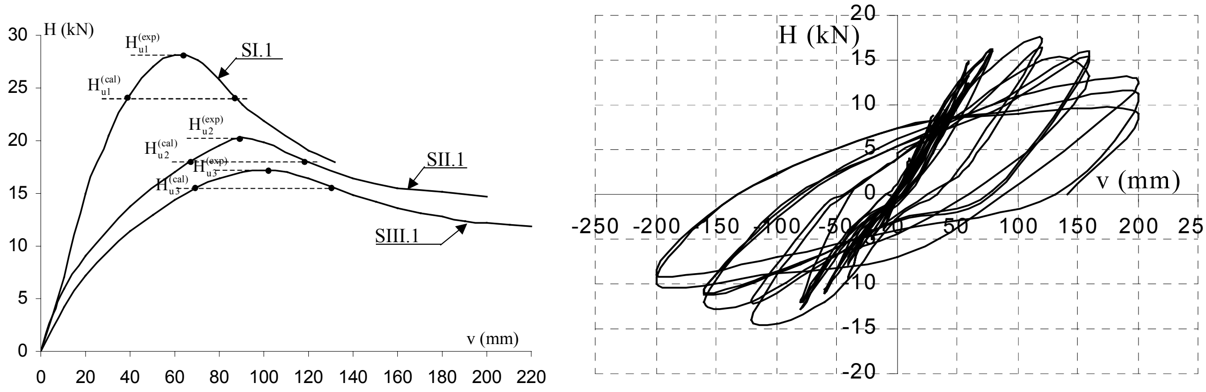



Figure 3. Mechanical model and composite cross-section of the elements

All the tests were conducted up to the maximum damage of concrete at the column bases; generally, at this stage, buckling of the longitudinal reinforcing bars occurred in the critical zone due to a quasi full deterioration of concrete encasement.

The cyclic testing of the relevant composite columns (Fig. 4.b) has shown clearly a rapid decreasing of resistance due to strong degradation of concrete in the critical zones.

Fig. 4.a shows the transverse force H-displacement v curves registered for the three monotonic tests, each curve including a clear soft branch below $H_u^{(exp)}$ due to buckling.




a) b)

Figure 4. Load-deformation relationship for members subjected to (a) monotonic and (b) cyclic load

For every cyclic test, the envelope curve (with two branches corresponding to v positive and negative) has been plotted for the best so that the curve is in tangential contact with the cycles close to their ends. When non-negligible shifting was observed for 3 repeated 3 cycles, only the first cycle has been used to define the envelope curve. Comparing to the monotonic H-v curves, it appears that the positive branch of every envelope can be superposed quasi perfectly on the corresponding monotonic curve whereas the negative branch shows some loss of resistance. This unsymmetrical tendency is well-known as general for any cyclic test, the direction of displacement first loaded being favored systematically. The increase of column slenderness seems to emphasize this tendency. To take into account the possible unsymmetrical cyclic behavior of the column, a reduction factor of about 0.9 may be applied to the maximum monotonic resistance. The reduce slenderness ratio were on the range of 0.73, 0.86 and 0.87.

# 4. Monotonic loading and strong / weak axis cyclic loading

Other tests (Hsu, H-L *et al.*, 2009) investigated the behavior of steel-concrete composite members under seismic loading. Test results shows that the strength deterioration rates increased when the members' steel strength ratios in the weak axis were increased. The tests also suggest that steel strength ratios in the strong and weak sectional directions to be set at approximately 2.2 to optimize the member's seismic performance.

The tests were conducted on twenty-four composite members, all with the same transversal section of 370x370 mm (as shown in Fig. 5). Six structural steels were used to investigate steel strength ratio effects. Also, the reinforcement was identical in all the columns.

Specimens were tested to a monotonic weak axis bending, to a combined axial load and cyclic strong axis, and a combined axial load with various magnitudes of bi-axial bending. The magnitude of the constant axial load was set to 13% of the compressive strength of the composite member, evaluated in the same way that is evaluated in the European code (EC4).

For specimens subjected to combined axial load and cyclic lateral force in the strong axis, the member's ultimate strength was reached at a drift ratio equaling 3%. Longitudinal bar buckling was observed at approximately 6% drift. In these cases, the member performances were governed by plastic hinge formation (H.-L. Hsu *et al.*, 2009).

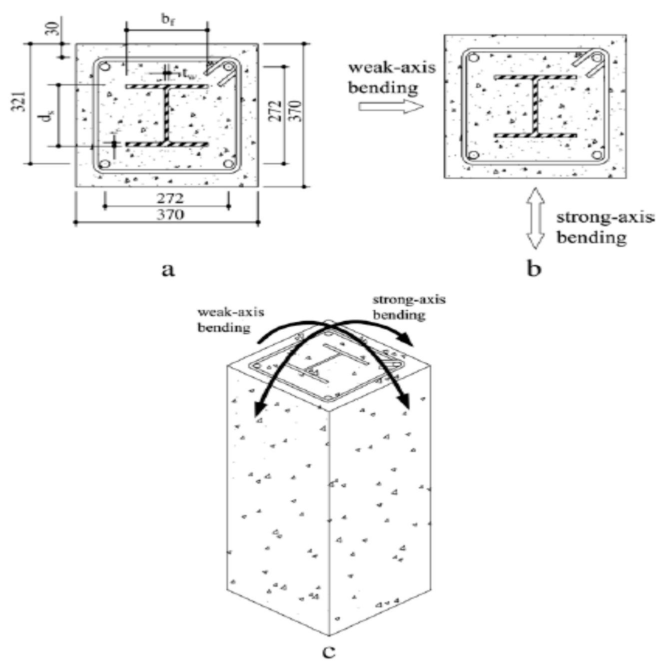



Figure 5. Specimen details: (a) Cross-sectional dimensions ; (b) definitions of load directions ;(c) 3-dimensional sketch

The load–deformation relationships of the tested specimens, shown in Fig. 6, indicate that similar strength deteriorations were exhibited when the ultimate member strengths were reached. However, the strength and the energy dissipation of the test members increased when the sections' steel strength ratios increased. These results conformed to the guidelines for beam–column designs, i.e. higher flexural strength in the load direction adopted to achieve higher member performance.

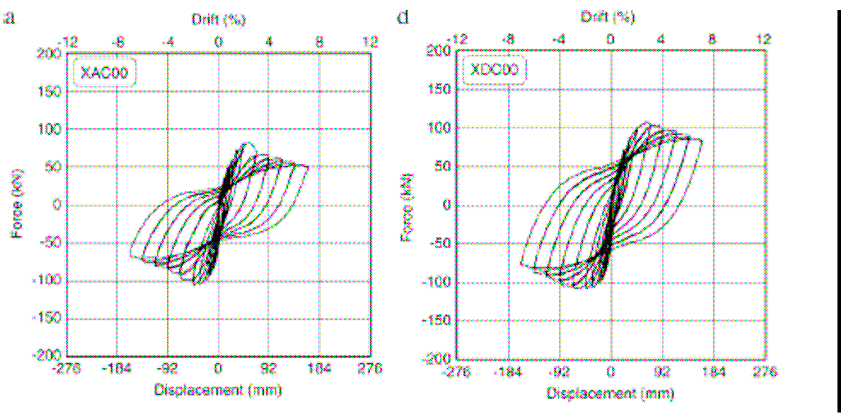



Figure 6. Load-deformation relationship for members subjected to axial load and strong axis bending

## 5. Square column with monotonic and cyclic loading

A short comment can be made regarding the tests made by (Matsui - K. Chung L *et al.*, 2007), on composite columns made by a square column subjected to a combined loading of axial force and cyclic lateral force, in the same way (Campian 2001) did. The ratio of the sectional capacity of the cross section and the axial load was 0.2 to 0.6, as in the Cluj tests were on the same range, 0.145 to 0.155.

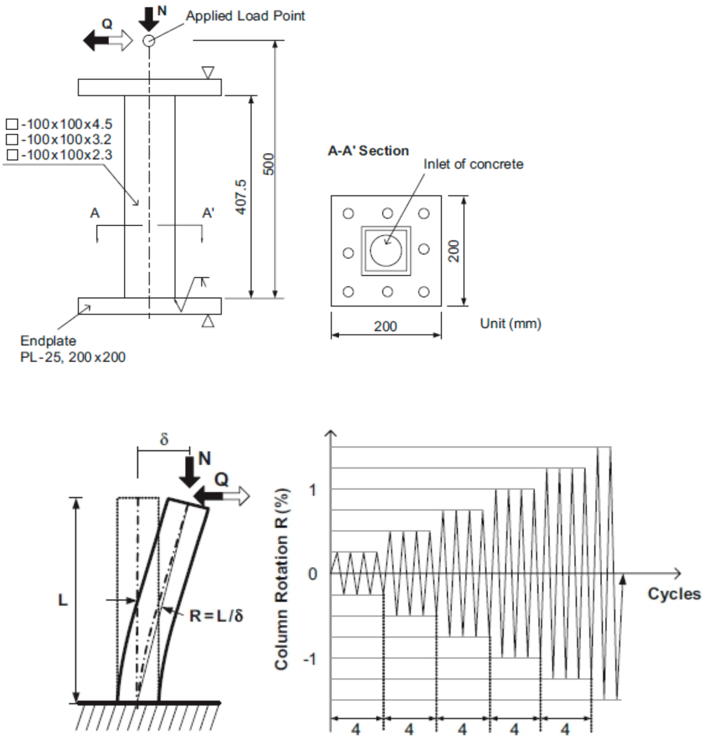
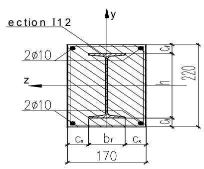
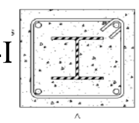
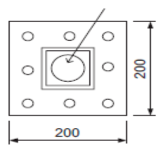



Figure 7. Speciments and testing protocol

Table 1. The ratio of the sectional capacity of the cross section and the axial load

| Review articles | Ratio Npl.Rd/Next | |
|---|---|---|
| Campian(2001) | 0.145-0.155 |  |
| H.-I  | 0.13 | |
| Matsui (1998) | 0.2-0.6 |  |

## 6. Brief overview of design provisions and comments

Basic strength design provisions for concrete encased composite columns in the USA are included in the ACI 318 Code , and the AISC-LRFD specifications.The AISC/LRFD proposed a design model with allowable stress specification. It stipulates that columns with a ratio of encased steel more than 0.04 ($A_s/A>0.04$) should be designed as composite and those with a ration less then 0.04 should be design as reinforced concrete. Although the ACI 318 Code has not applied a comparable restriction, , the same limit is implied by the seismic design provisions for composite strucures (NEHRP 1997).
Applying the limitation of AISC-LRFD to the steel area $A_s/A>0.04$ and the concrete strenght, $f_c$ <55MPa we would be in the situation of limiting the use of the code to the low-strengh sections.
The situation raise several concerns, namely that no provision are applicable to composite columns with $A_s/A>0.04$ and $f_c$ >55MPa, even though ACI318 permits the design of reinforced concrete with high-strenght concrete.
The results we made for all the columns conclude that the columns with very small contribution from the steel section are governed by the AISC rather rhan the ACI provisions.The distinction between design procedures based on the ratio of $A_s/A$ does not aknowledge differences in behaviour of the tested composite columns associated with the wide range of available concrete and steel strenghts.
Other comments can be made to nominal strenghts. The AISC-LRFD provision calculate the strenght interaction between axial and flexural effects on a bilinear intercation formula, wich has the same form as those for the steel columns. The ACI 318 provision for these type of columns are essentially the same as those for reinforced concrete columns. There are three large differences in the nominal strenghts for combined axial compression and bending calculated using the ACI 318 and the AISC-LRFD specifications. Compared with the tests, the ACi 318 provisions are slightly conservative, particulary at higher axial loads and for higher-strength concrete. In the studied cases, the ACI 318 values were up to 10% unconservative for columns with fc=110MPa. The AISC provisions are shown to be overly conservative at intermediate to high axil load levels, particulary for columns with small encased steel sections and/or where higher-strength concrete is used. For example, AISC strength prediction was over 63% conservative for the column with a steel ratio of 0.04 and fc=110Mpa. The analyses results emphasize the need to reconcile the large differences between tha ACI 318 and AISCLRFD nominal trength provision for composite columns.The presence of a large steel core provides a beneficial residual strength following concrete crushing that leads to improve ductility.
So, under severe seismic conditions, a possible collapse-avoidance design strategy could be the one who allow the concrete to deteriorate and absorb energy and to design and detail the steel core to resist the dead load of the structure and provide enough stiffnessto minimize the risk of collapse. For composite columns with higher-strenght concrete, this strategy would be to increase the maximum limit of permissible yield strength of encased steel sections beyond the current limits .
The improvement of mechanical characteristics and increasing the performances of composite columns led to new researches in our Laboratory. An experimental program was initiate having the target of studying the effects of increasing of the concrete class, using high performance concrete and ultra-high performance concrete. The experimental programe will follow some of the directions of the passed research program on composite columns. In parallel, a model is created usig a FEM program, for a computationally efficient prediction for the behavior of the composite column.

## 7. References

[1] J-M., Aribert, C., Campian, V., Pacurar (2004), Etude expérimentale et interpretation analytique du comportement statique et cyclique de poteaux mixtes acier flechis par une charge transversale ", *Revue „Construction metalliques"*, Centre Technique de la Construction metallique (CTICM), Paris, Franta, pp. 3-22.

[2] J-M., Aribert, C., Campian, V., Pacurar(2003), Monotonic and cyclic behavior of fully Encased composite columns and dissipative interpretation for seismic design", Swets&Zeitlinger B.V., Lise, The Netherlands, *Proc. of STESSA 2003*, pp. 115-122C

[3] Eurocode 4: Design of composite steel and concrete structures. Part 1-1 ; General rules and Rules for buildings ENV 1994-1-1.2004.

[4] K. Chung, J.Chung, S. Choi, (2007) Prediction of pre- and post-peak behavior of concrete Filled square steel tube columns under cyclic loads using fiber element method, *Elsevier, Thin-Walled Structures* 45,pp. 747-758

[5] Hsu, H-L, Jan, F., Juang, J-L, (2009) Performance of composite members subjected to axial an bi-axial bending, *Journal of Constructional Research* 65, pp. 869-878

[6]C., Campian, , N. Chira ,V. Sav, "Strength and ductility of concrete encased composite Columns", Turkish Constructional Steelwork Association (TUCSA), Istanbul, Turcia, *Proc. of Steel Structures: Culture & Sustainability 2010*, pp. 485-493, ISBN 978-97592461-2-9

[7] Zhao, G, Zhang, M, Li, Y. (2009), Behavior of slender steel concrete composite columns in eccentric loading, *J. Shanghai University* , Engl. edition, pp. 481-4888.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Shrinkage of Ultra-High Performance Concrete

Ioan Sosa[*1], Cornelia Magureanu[2]

[1,2] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 25 Baritiu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*The paper presents time effects regarding shrinkage of Ultra-High Performance Concrete, with steel fibers reinforcement (UHPFC) and without fibers (UHPC). The concrete composition contains mainly local materials. The shrinkage characteristics were determined in terms of early age shrinkage and long-term shrinkage. Early age shrinkage was tested on concrete without any heat curing regime and the measurements started one our after casting the concrete and lasted for 7 days. Long-term shrinkage was tested on concrete subject to five days steam treatment (T= 90ºC; UR=80%). The measurement started immediately after the finalization of the curing regime at the concrete age of 6 days and lasted more than 110 days. The results revealed the prevalence of the autogenous shrinkage in the magnitude of the overall shrinkage at early age. After thermal treatment finalization no shrinkage occurred, in stead swelling occurred in the first 130 days of measurement.*

## Rezumat

*Articolul prezintă efectul timpului asupra contracției betonului de ultra-înaltă rezistență, cu ados de fibre metalice (BUIPF) si fără ados de fibre (BUIP). Compoziția betonului este alcătuită in principal din materiale de proveniență locală. Contracția a fost determinată atât la vârste mici cât şi pe termen lung. Contracția la vârste mici a fost determinată pe un beton care nu a urmat un tratament termic, măsurătorile începând la o oră de la turnare şi durând 7 zile. Contracția pe termen lung a fost testată pe un beton supus tratmentului termic cu abur (T= 90ºC; UR=80%) timp de 5 zile. Măsurătorile au început imediat după finalizarea tratamentului termic la vârsta de 6 zile si au durat peste 110 zile. Rezultatele au relevat prelevanța contracției autogene în totalul contracției la vârste fragede. După finalizarea tratementului termic nu s-a mai observat contracție, in schimb fiind măsurate deformații din umflare.*

**Keywords:** ultra-high performance concrete, fibers, steam treatment, autogenous shrinkage, swelling.

## 1. Introduction

Ultra-high performance concrete (UHPC) is a cemeticious composite with compression strength that exceeds 150 MPa, tensile strength over 7 MPa and low porosity and. It became of world-wide interests with the publication of research of Richard and Cheyrezy in 1995 [1].
The UHPC presented in this paper was modeled by prof.Cornelia Magureanu and co-workers with the aim of a full characterization of this kind of concrete produced with locally available materials.

[*] Corresponding author: Tel./ Fax.: 0264 401 559
E-mail address: pavel.sosa@bmt.utcluj.ro

The concrete composition is submitted in order to be patented. Extensive researches regarding mechanical properties and durability of the designed concrete were previously published. In this paper the shrinkage characteristics of the concrete is analyzed.
Accordingly to CEB-FIP [2] concrete shrinkage can be subdivided into the components: plastic shrinkage (water loss in fresh concrete), chemical shrinkage, autogenous shrinkage, drying shrinkage and carbonation shrinkage. Plastic shrinkage usually doesn't occur in high and ultra strength concrete due to the low water-to-cement ratios. However, for the studied concrete, water losses in fresh state were prevented by providing concrete surfaces with anti-evaporation oils. Carbonation shrinkage due to the carbonation of concrete surfaces is also of low interests for superior concretes as proven by the previous work of Magureanu and Negrutiu [3] on high performance concrete designed with same type and same manufacturer cement as use for UHPC. Accordingly to [4] chemical shrinkage is part of chemical shrinkage together with self-desiccation and swelling of concrete and can be summarize as illustrated in Fig. 1.

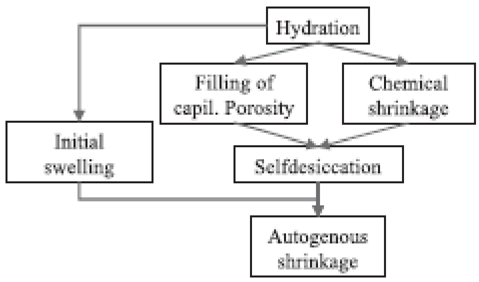



Figure 1. Proposed mechanism of autogenous shrinkage [4].

Regarding UHPC Graybeal research on total shrinkage revealed an ultimate shrinkage of 790 micro-strains (μm/m) for untreated concrete at 40 days after casting. Steam treated specimens displayed no deformations, nor shrinkage or swelling, after the treatment finalization. The studied concrete contained 2 Vol.-% steel fibers reinforcement [5]. The Interim Recommendations of SETRA [6] on UHPFC (fiber addition 2 Vol.-%) stated the same that no shrinkage occurs after steam curing and the ultimate shrinkage is 550 μm/m for water-to-cement ratios of 0.17 to 0.20.
Habel [7] research on UHPFC revealed that swelling occurs between 6 to 31 hours after water addition and than only shrinkage occurs. Swelling was also observed by other authors but only for few hours due to thermal expansion cause by thermal treatment or hydration heat [8] [9] [10]

## 2. Experimental program

### *2.1 Materials*

Shrinkage was tested using two ultra-high concrete compositions, one without fibers(UHPC) and one with a mixture of short and long steel fibers (UHPFC). The UHPFC mixture had a compressive strength over 180MPa and the UHPC mixture over 150 MPa when subject to steam curing. Further details on mechanical properties can be found in [11] [12]
The constituent materials included locally manufactured high strength Portland Cement CEM I 52.5R and imported grey silica fume with high content of $SiO_2$, commercially available as Elkem Microsilica Grade 940V. The locally used aggregates were quartz sand divided in fine sand (0-0.3), medium fine sand (0.3-0.63) and coarse sand (0.63-1.2 mm). The suitable workability of the concrete was ensured by the use of a last generation superplasticizer Glenium ACE440, produced by BASF Romania. In addition, two types of high strength fibers were used: 50% long hooked end fibers (WMS-25/0.4/4/304), with L/d=25/0.4 and 50% short straight fibers (MSF-6/0.175/5-26WB)

with L/d=15/0.2, with the minimum tensile strength of 1450 MPa and 2200 MPa, respectively. The fibers are produced by Baumbach Metall Gmbh - Germany, with the Romanian branch Baum Cas Fibers SRL-Romania. The fiber density is 7.85 g/cm$^3$. The fibers proportion was 2.55 Vol.-%.
The water-to-binder (cement+silica fume) ratio of the concretes is presented in Table 1.

Table 1**:** Water-to-binder ratios

| Composition | Water/Binder (W/B) |
|---|---|
| UHPC | 0.125 |
| UHPFC | 0.129 |

### *2.2 Testing procedure*

Shrinkage was measured in terms of early age shrinkage and long-term shrinkage on two composition: fiber reinforced concrete (UHPFC) and non-reinforced concrete (UHPC)
Early age shrinkage was tested in terms of autogenous shrinkage and total shrinkage. Autogenous shrinkage was tested using sealed specimens with no moisture exchanged between concrete and environment. The sealed conditions were provided by the use of aluminum self-adhesive foil and supplementary plastic foil. The total shrinkage was tested on un-sealed specimens that permitted moisture exchange between concrete and environment. The measurement started 1 hour after casting and lasted for 7 days. The readings were made every 30 minutes in the first 12 hours and twice a day afterwards until the concrete age of 7 days. The autogenous shrinkage setup is illustrated in Fig. 2. The setup for total shrinkage is similar except the moisture loss foils.

Figure 2. The autogenous shrinkage test setup.

The length changes were measured using two steel bars embedded 2 cm into concrete at both ends of the specimen. The length measurement devices were placed at the free ends of the steel bars. The concrete specimens did not receive any thermal treatment during the test. The room temperature was T=22ºC and relative humidity RH=60%
Long-term shrinkage was tested in terms of total shrinkage. The total shrinkage was tested on un-sealed specimens that permitted moisture exchange between concrete and environment. The measurements started at the concrete age of 6 days. Previously to the test the concrete was subject to steam curing for five days. The steam curing started 1 day after casting. The length change readings were made on daily basis in the first 30 days and weekly afterwards. The length changes were measured using steel gauges glued on the concrete surface. During the test the concrete specimens were stored in a climatic room. The room temperature was kept constant at T=20ºC and relative humidity RH=60%.

## 3. Results and discussions

### *3.1 Early age shrinkage*

Early age shrinkage results are graphically displayed in Fig. 3 for autogenous shrinkage ($\varepsilon_{cas}$) and in Fig. 4 for total shrinkage ($\varepsilon_{cs}$). The deformations are measured in micro-strains (µm/m). The figures contain both fiber reinforced concrete (UHPFC) and non-reinforced concrete (UHPC) behaviors.

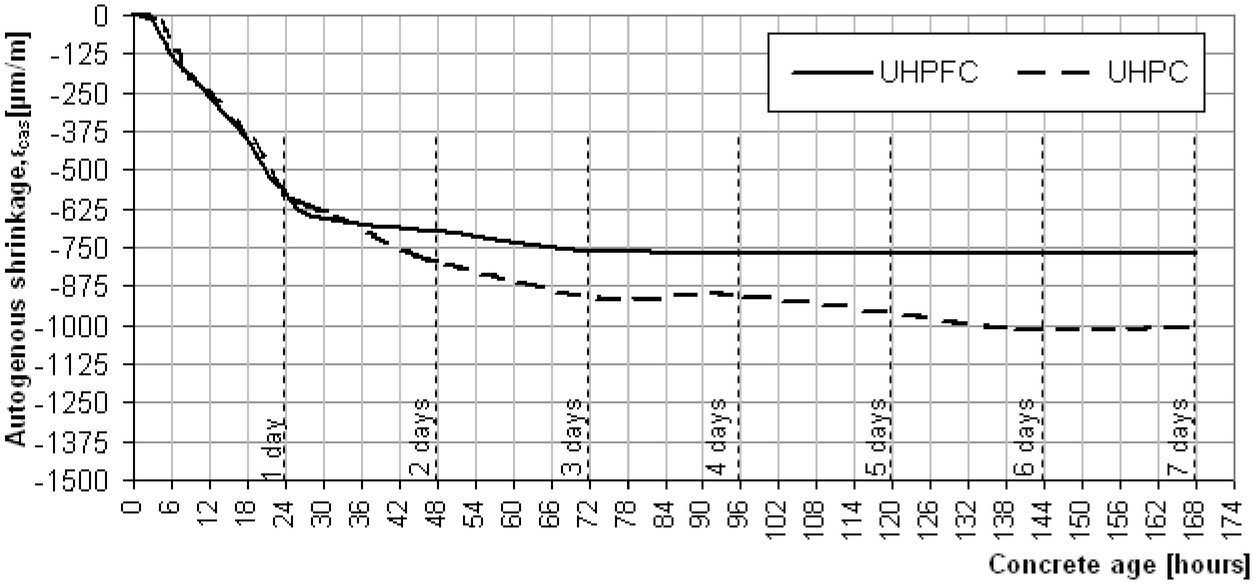



Figure 3. Autogenous shrinkage.

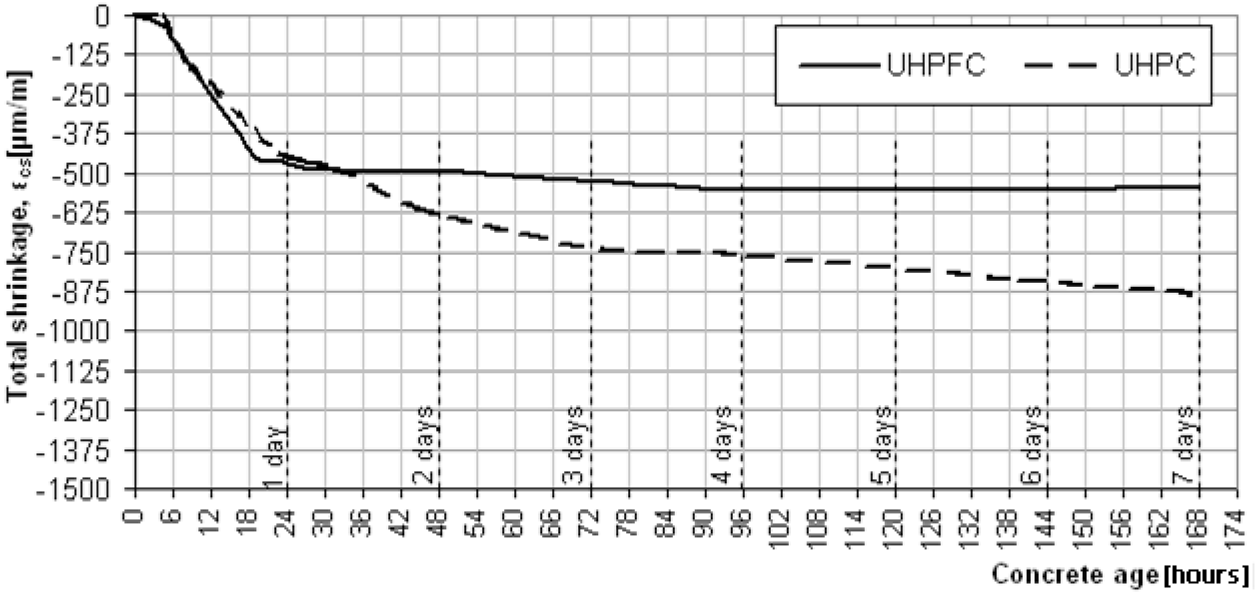



Figure 4. Total shrinkage.

The results showed that the shrinkage (total and autogenous) occurred within 4 hours after casting for both concretes and developed at high speed until 24 hours. The shrinkage had almost the same rate of 23 µm/m/hour for both UHPC and UHFPC in this interval. After 36 hours the reinforced and non-reinforced concrete shrinkages developed differently. The UHPFC displayed little increase whereas UHPC continued to display large values of shrinkage. After 3 days UHPFC had almost no shrinkage. In stead non-reinforced concrete shrinkage continued with 1.05 µm/m/hour rates until test ended at 7 days. The final autogenous shrinkages (sealed specimesn) values were $\varepsilon_{cas}$=766 µm/m for UHPFC and $\varepsilon_{cas}$=1011 µm/m for UHPC. The lower values of UHPFC are due to the

presence of fibers which oppose to concrete deformations. However concrete has to have sufficient stiffness in order to ensure the adherence with fibres that is why the deformations were similar for both concretes in the first 24 hours. The total shrinkage values (un-sealed specimens) were $\varepsilon_{cs}$=550 µm/m for UHPFC and $\varepsilon_{cs}$=888 µm/m for UHPC. This means that concrete expose to environment had no drying shrinkage ($\varepsilon_{cds}$= $\varepsilon_{cs}$-$\varepsilon_{cas)}$ but swelling. This finding is in contrast with that of other authors [8] [9] [10] which stated that only shrinkage occurs on un-sealed specimen and minor thermal expansion (see Introduction).
The results revealed that for the researched ultra-high performance concretes without any curing regime the autogenous shrinkage is the driving and only force in developing deformations at early age.

### *3.2 Long term shrinkage*

The long term deformations of both UHPC (no fibers) and UHPFC (steel fibers) are displayed in FIG. 4. The deformations were measure on un-sealed specimens subject to steam curing for 5 days prior testing.

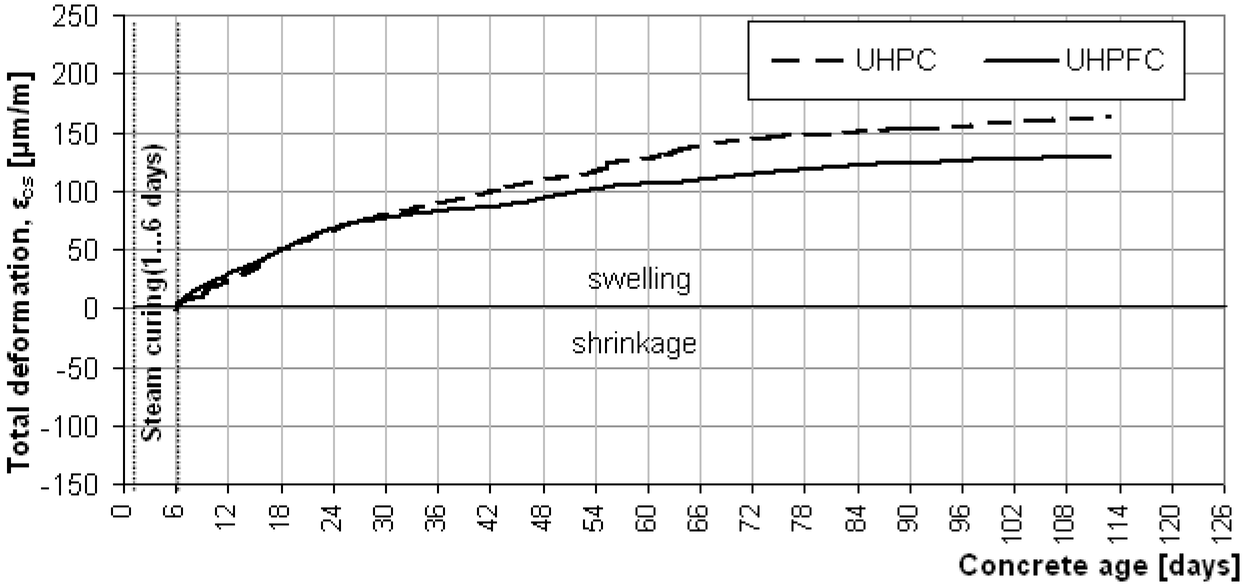



Figure 4. Total deformations for steamed cured specimens.

The results show that concrete has no shrinkage after the finalization of the steam curing. In stead swelling deformations occurred for the all period of the test (6…113 days). Once again the fiber addition limited the concrete deformations. Non-reinforced concrete specimens displayed swelling deformations with 25% greater than that of reinforced concrete. The swelling deformation seems to stabilize at approximately 70 days after casting. The final swelling values measured at 113 days were: $\varepsilon_{cds}$=130 for UHPFC and $\varepsilon_{cds}$=162 for UHPC.
The swelling deformation produced after steam curing could be attributed to the very low water-to-binder ratio (w/b=0.125) which is less than the usual ratios used for this kind of concretes. The results are in contrast with those of [7] [8] which reveal no deformation after the heat curing finalization.Swelling deformations may be favorable to the self-healing of concrete (reduction of crack openings), observed in the researches of Magureanu and Negrutiu on high-performance concretes [3].

## 4. Conclusions

The shrinkage behavior of ultra-high performance concrete is significantly different from that of ordinary concretes. The autogenous shrinkage is the dominant deformation that occurs at early concrete ages, whereas the ordinary concretes displays only minor autogenous shrinkage and large drying shrinkage. The autogenous shrinkage is reduced by 24% if steel fibers (2.55Vol.-%) are added in concrete composition. The UHPC and UHPFC didn't display any drying shrinkage. The 5 days steam curing (T=90ºC; RH=80%) consumed all concrete shrinkage, afterwards only swelling occurred. The swelling tendency of UHPC may be favorable to concrete self-healing.

## Acknowledgements

The authors would like to express their gratitude to CNCSIS for financing the research program within the PCE-PN II-IDEI Program, code 1053/2007.

## 5. References

[1] Richard P., Cheyrezy M. Reactive powder concretes with high ductility and 200-800 MPa compressive strength, **ACI SP144-24**, pp. 507-518, 1994.

[2] CEB-FIP Bulletin **No. 42**: *Constitutive modelling of high strength/high preformance concrete*, Lausanne, Switzerland, 2008.

[3] Negrutiu C, Magureanu C. *Betonul de inalta rezistenta. Durabilitate.* Cluj-Napoca: U.T. Press, 2010

[4] Barcelo L, Moranville M., Clavaud B. Autogenous shrinkage of concrete: a balance between autogenous swelling and self-desiccation, *Cement and Concrete research,* **Vol. 35**, pp. 177-183, 2005.

[5] Graybeal B. Material property characterization of ultra-high performance concrete, *US Department of Transportation,* **FHWA-HRT-06-103**, 2006.

[6] AFGC-Setra. *Ultra-high performance fiber-reinforced concrete. Interim Recommendations*, France, 2002

[7] Habel K., *Structural behaviour of elements combining ultra-high petrformance fiber reinforced concrets (UHPFRC) and reinforced concrete*, PhD Thesis **No. 3036**, pp. 20, EPFDL, Lausanne, 2004.

[8] Staquet S., Espion B., Early-age autogenous shrinkage of UHPC incorporating very fine fly ash or metakaolin in replacement of silica fume, *Proceeding of the International Symposium on Ultra-High performance Concrete*, **No. 3**, Kassel, Germany, 2004.

[9] Ma J., Orgass M., *Comparative Investigations on Ultra-High Performance Concrete with and without Coarse Aggregates*, Lacer, **No.9**, 2004.

[10] Ma J., Dietz J., *Ultra high performance self compacting concrete*, Lacer, **No.7**, 2002.

[11] Magureanu C., Sosa I., Negrutiu C., Heghes B., Physical and mechanical properties of ultra-high strength fiber reinforced cemetitious composites, *Proceeding of the FraMCoS. 7th International Conference on Fracture Mechanismsof Concrete and Concrete Structures*, Seul, South Correa, 2009

[12] Sosa I, Constatinescu H., Tibea C., Vagii V, Freeze-thaw durability of ultra-high performance concrete with and without fibers, *10th International Scientific Conference VSU'2010*, Bulgaria, 2010

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Experimental Study of the Cracking Behaviour at Bending of High Strength Concrete Beams

L.C. Letia[*1]

[1]*Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*This paper summarizes the research findings of the characteristics of high strength concrete (HSC) for flexural cracks of reinforced concrete girders. A number of 14 HSC beams with different percentage of ρ (reinforcement ratio) cast and incrementally loaded under bending. The test results showed that not all equations used to evaluate the different mechanical properties and cracking behaviour of normal concrete can be applied to HSC. In some cases, the difference between test results and theoretical results is quite important. The 14 reinforced HSC beams, prism and cubes were cured and tested at the Laboratory of Reinforced Concrete Department, of the Technical University of Cluj- Napoca.*

## Rezumat

*In aceastã lucrare sunt analizate diferite caracteristici ale betonului cu rezistențe la compresiune mai mari de 60 MPa în ceea ce priveşte fisurile din încovoiere a grinzilor armate. Sunt analizate 14 grinzi cu diferite procente de armătură. În urma cercetărilor efectuate s-a observat cã, odatã cu creşterea clasei betonului de la C60/75 la C80/95 şi asocierea acestui beton cu rezistență mai mare cu oțelul S500 (Bst500S), valorile momentului de fisurare creşte. De asemenea, se observă că relațiile ce estimează momentul de fisurare pentru betonul obişnuit nu pot fi aplicate elementelor încovoiate realizate cu betoane de înaltă rezistență. Elementele testate au fost realizate şi încercate în cadrul Laboratorului disciplinei de Beton Armat şi Precomprimat al Facultății de Construcții din cadrul Universității Tehnice Cluj-Napoca.*

**Keywords:** Experimental study, Cracking behavior, High strength concrete, Beams.

## 1. Introduction

The researches sight the cracking behaviour at bending of reinforced concrete beams. The experimental program is based on bending tests of 14 beams, having the transversal section of 125x250mm, 3200mm length and the clear span of 3000mm. For each concrete class and reinforcement ratio two beams were tested. There have been studied two concrete classes: C60/70 and C80/95 and two types of steel reinforcement PC52 and Bst500S.

[*] Corresponding author: Tel./ Fax.: 0264 594967
E-mail address: catinca.letia@bmt.utcluj.ro

# 2. Materials

## *2.1. Concrete*

Table 1. Concrete mix

| **Concrete class** | | **C60/75** | **C80/95** |
|---|---|---|---|
| **Component** | **Component characteristics** | **Dozage** | |
| Cement (C) | CEM I 52,5R | 480 kg/m$^3$ | 520 kg/m$^3$ |
| Silica fume (SF) | | 48 kg/m$^3$ | 52 kg/m$^3$ |
| Sand | 0 - 4mm | 530 kg/m$^3$ | 530 kg/m$^3$ |
| Gravel | 4 - 8mm | 530 kg/m$^3$ | 530 kg/m$^3$ |
| Gravel | 8 - 16mm | 706 kg/m$^3$ | 706 kg/m$^3$ |
| Water | | 152 l/m$^3$ | 152 l/m$^3$ |
| Superplasticizer | RAVENIT | 13,5 l/m$^3$ | 13,5 l/m$^3$ |
| W/(C+SF) | | 0,29 | 0,27 |

The concrete classes that have been tested are: C60/75 and C80/95. Table 1 presents the concrete mix for each concrete class. The compressive strength was tested on 150mm cubes. For each beam at least 3 cubes were tested at 28 days after curing and respectively when the beams were tested.


Fig. 1. Compressive strength test on cubes, in the Advantest press.

Table 2. Average compressive strength of concrete

| Beam | $f_{c,cube}$ [MPa] | $f_{ck}$ [MPa] | $f_{cd}$ [MPa] | Concrete class |
|---|---|---|---|---|
| I 1-1 | 92,40 | 63,31 | 42,20 | C60/75 |
| I 1-2 | | | | |
| I 2-1 | 85,10 | 59,55 | 39,70 | |
| I 2-2 | | | | |
| I 3-1 | 84,90 | 59,44 | 39,63 | |
| I 3-2 | | | | |
| I 4-1 | 89,90 | 62,04 | 41,36 | |
| I 4-2 | | | | |
| BH 1-1 | 102,00 | 67,93 | 45,28 | C80/95 |
| BH 1-2 | | | | |
| BH 2-1 | 101,00 | 67,46 | 44,97 | |
| BH 2-2 | | | | |
| BH 3-1 | 104,00 | 68,84 | 45,89 | |
| BH 3-2 | | | | |

Table 3. The concrete elasticity modulus and the flexural tensile strength

| Beams type | $E_c^{real}$ [MPa] | $f_{ct,fl}$ [MPa] |
|---|---|---|
| I | 42000 | 5,27 |
| BH | 45500 | 5,29 |

The elasticity modulus was determined on 100x100x300mm prisms, which were kept in similar conditions as the tested beams. At least 3 prisms were tested for each beam. The results are presented in table 3. The flexural tensile strength was tested on prisms and the results are presented in table 3.

a) the prism equipage b) the prism failure

Fig. 2. Modulus of elasticity test on prisms.

### *2.2. Steel reinforcement*

The longitudinal reinforcement diameter, of PC52 and Bst500S, was between: ø12 and ø18 mm. The stirrups were made of OB37 steel type and had the diameter of: ø6 mm. The reinforcement characteristics were determined using VEB ZD 10/90 -1976 press machine. To consider all the parameters the are influencing the beams behaviour at bending, the reinforcement mechanical coefficient is defined using the relation (1).

Table 4. The steel mechanical characteristics

| Beam type | Position | Reinforcement type | $f_t$ [MPa] | $f_y$ [MPa] | $\frac{f_y}{f_t}$ | $k = \frac{f_t}{f_y}$ | Reinforcement class |
|---|---|---|---|---|---|---|---|
| I | longitudinal | PC 52 | 554,8 | 329 | 0,706 | 1,41 | C |
| BH | longitudinal | Bst500S | 642,25 | 553 | 0,861 | 1,16 | C |

$$\omega = \omega_s - \omega' \tag{1}$$

where:
$\omega_s$ – the mechanical coefficient of the tensile reinforcement;
$\omega'$ – the mechanical coefficient of the compressed reinforcement.

Table 5. The longitudinal reinforcement type, the mechanical coefficient ($\omega$) and the reinforcement ratio (p)

| Beam | Concrete class | Reinforcement | $\omega$ [-] | p [%] |
|---|---|---|---|---|
| I1-1 | C60/75 | PC52 | 0.189 | 2.625 |
| I1-2 | | | 0.192 | 2.658 |
| I2-1 | | | 0.235 | 3.072 |
| I2-2 | | | 0.229 | 2.990 |
| I3-1 | | | 0.258 | 3.358 |
| I3-2 | | | 0.258 | 3.358 |
| I4-1 | | | 0.279 | 3.786 |
| I4-2 | | | 0.282 | 3.834 |
| BH1-1 | C80/95 | Bst500S | 0.129 | 1.346 |
| BH1-2 | | | 0.129 | 1.346 |
| BH2-1 | | | 0.179 | 1.855 |
| BH2-2 | | | 0.179 | 1.855 |
| BH3-1 | | | 0.219 | 2.312 |
| BH3-2 | | | 0.219 | 2.312 |

The coefficients $\omega_s$ and $\omega'$ are defined using the relations (2) and (3):

$$\omega_s = \frac{A_s \cdot f_{yd}}{b \cdot d \cdot f_{cd}}; \tag{2}$$

and

$$\omega' = \frac{A_{s2} \cdot f_{yd}}{b \cdot d \cdot f_{cd}}; \tag{3}$$

where:
$A_s$ – the tensile reinforcement;
$A_{s2}$ – the compressed reinforcement;
$f_{yd}$ – reinforcement yield strength;
$f_{cd}$ – the concrete design compressive strength;
b – the concrete section width;
h – the concrete section height.

## 3. Beams equipages

All the beams were tested using hydraulic testing system (WPM 262/6-1977hydraulic press of 3000 kN, having the precision class 1) and loaded with the two equal concentrated loads, F. The distance between the two concentrated loads was kept equal to $L_1$=1000 mm.

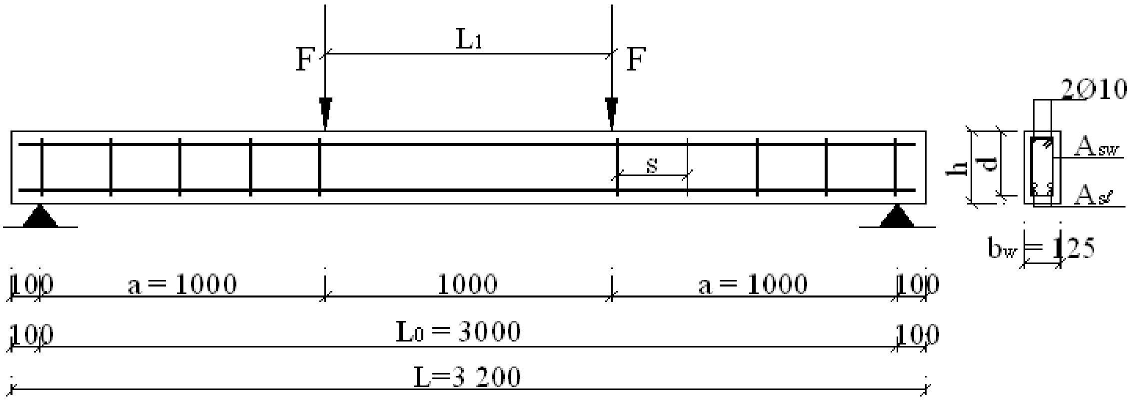



Fig.3. The loading scheme

Both ends of the beam were free to rotate and translate under load. At each load increment, the mid-span deflection and all strain reading were recorded and the developing crack patterns marked at the beam surface. The concrete maximum compressive strain is recorded at the mid-span by strain gauges glued are placed on one of the beam side in the tensile zone. The beams are submitted to a growing monotonic loading until failure. The monotonic loading is applied 10% of the failure calculated force. The loading scheme is presented in Fig. 3.


Fig. 4. The beams test

At each monotonic load growing the crack trajectory, width and high are recorded. The crack width is recorded on the direction of the longitudinal reinforcement centroid, using a magnifying glass, having the precision of 0,1mm. The crack high was measured perpendicular to the longitudinal reinforcement

## 4. Beams tests results

Table 6. The cracking bending moment, $M_{cr}^{EC2}$, according to EC2 SR EN 1992- 1-1:2004[25]

| BEAM | Concrete class | $f_{ctm}^{EC2}$ [MPa] | $b_{real}$ [mm] | $h_{real}$ [mm] | W [$mm^3$] | $M_{cr}^{EC2}=f_{ctm}*W$ |
|---|---|---|---|---|---|---|
| I1-1 | C60/70 | 4.4 | 130 | 240 | 1.25 | 5.49 |
| I1-2 | | | 125 | 245 | 1.25 | 5.50 |

| | | | | | | |
|---|---|---|---|---|---|---|
| I2-1 | | | 130 | 240 | 1.25 | 5.49 |
| I2-2 | | | 130 | 245 | 1.30 | 5.72 |
| I3-1 | | | 130 | 245 | 1.30 | 5.72 |
| I3-2 | | | 130 | 245 | 1.30 | 5.72 |
| I4-1 | | | 130 | 245 | 1.30 | 5.72 |
| I4-2 | | | 125 | 250 | 1.30 | 5.73 |
| BH1-1 | C80/95 | 4.8 | 125 | 250 | 1.30 | 6.25 |
| BH1-2 | | | 125 | 250 | 1.30 | 6.25 |
| BH2-1 | | | 125 | 250 | 1.30 | 6.25 |
| BH2-2 | | | 125 | 250 | 1.30 | 6.25 |
| BH3-1 | | | 125 | 250 | 1.30 | 6.25 |
| BH3-2 | | | 125 | 250 | 1.30 | 6.25 |

Table 7. The design cracking moment ($M_{cr}^{EC2}$, $M_{cr}^{ACI}$) and the experimental cracking bending moment, $M_{cr}^{exp}$.

| **Beam** | **ω [–]** | **p [%]** | **Reinforcement** | **Concrete class** | **$M_{cr}^{exp}$ [kN*m]** | **$M_{cr}^{EC2}$ [kN*m]** | **$M_{cr}^{EC2}/M_{cr}^{exp}$** | **$M_{cr}^{ACI}$ [kN*m]** | **$M_{cr}^{ACI}/M_{cr}^{exp}$** |
|---|---|---|---|---|---|---|---|---|---|
| **I1** | 0,19 | 2,60 | PC52 | C60/75 | 8.00 | 5.99 | 0.75 | 3.45 | 0.43 |
| **I2** | 0,22 | 3,00 | | | 5.00 | 5.49 | 1.10 | 3.53 | 0.71 |
| **I3** | 0,26 | 3,36 | | | 5.00 | 5.72 | 1.14 | 3.77 | 0.75 |
| **I4** | 0,28 | 3,80 | | | 5.00 | 5.72 | 1.14 | 3.93 | 0.79 |
| **BH1** | 0,13 | 1,35 | Bst500S | C80/95 | 8.00 | 6.25 | 0.78 | 3.44 | 0.43 |
| **BH2** | 0,18 | 1,86 | | | 8.00 | 6.25 | 0.78 | 3.54 | 0.44 |
| **BH3** | 0,22 | 2,31 | | | 8.00 | 6.25 | 0.78 | 3.73 | 0.47 |

## 4. Conclusions

The tests showed an increase of the cracking moment with the concrete classe. That means an increase of the bending moment when the first crack, having the crack width of 0,1mm, was detected. The tests lead to following conclusions:

- at the elements having the concrete class C60//75 and the reinforcement PC52 the experimental values of the cracking moment, considered when the crack width was 0,1mm, $M_{cr}^{exp}$ inferior with approximately 10-15% to the design crack width determined according to SR EN 1992-1-1:2004. This inconsistency can be attributing to the use of PC52 reinforcement type and not the S500 (Bst500S). The first crack was detected at approximately 8% of the experimental failure load.
- at the elements having the concrete class C80/95 and the reinforcement Bst500S, the design values according to SR EN 1992-1-1:2004[25] this observation underlines the very good behaviour of the high strength concrete (class C80/95), respectively the increasing of the cracking moment. This very good behaviour is due to the increase of the flexural tensile strength of the class C80/95 respect to the concrete class C60/75. The first crack was detected at approximately 15% of the experimental failure load. When using the ACI 2005 relations to determine the cracking moment, the obtained values are overestimating the value of the experimental bending moment, when the first crack was detected, up to two times.

## 5. References


1. EUROCODE 2: Design of concrete structures – Part 1: General rules and rules for buildings - 2004
2. ACI COMMITTEE 363. "Review of ACI Code for Possible Revisions for High Strength Concrete" (ACI 362 R- 92) American Concrete Institute, Detroit, 1992
3. ACI COMMITTEE 318. "Building Code Requirements for Structural Concrete" ACI 318-2005
4. STAS 10107/0-90 – Constructii Civile si Industriale. Calculul si alcatuirea elementelor structurale din beton, beton armat si beton precomprimat
5. MAGUREANU, C. Reinforced Steel - Concrete Bond of High Strength Concrete Proc. Of the first "fib"Congress 2002. Vol. 7. pp. 241- 246
6. MAGUREANU, C, HEGHES, B. Experimental study on ductility reinforced concrete beams using high- strength concrete. The 2-nd International "fib" Congress- Naples-2006.
7. Leția, L.-C. Cracking Behavior at Bending and Mechanical Properties of High Strength/ High Performance Concrete, Section:Civil Engineering-Architecture, vol.II, Acta Technica Napocensis, 2008, pp. 107-114..

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Cracking Behavior at Bending of Singly Reinforced Concrete Beams

L.C. Letia[*1]

[1]*Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*When studying the crack behaviour of singly reinforced concrete beams under external loads, there are determined the stress, respectively the forces that are recorded in the transversal section. The stages that have to be studied are: the bending moment when the first recorded crack appeared and when the tension concrete strength is reached, $M_{cr}$; the crack stabilization; the ultimate bending moment, $M_u$.*

## Rezumat

*In această lucrare sunt analizate starea de eforturi în secțiunea nefisurată şi în momentul fisurării, la apariția primei fisuri vizibile. Relația momentului de fisurare poate fi scrisă analizând trei prespective: modulul de rupere al betonului, fr; momentul de inerție pentru secțiunea fisurată, Icr; utilizând legea de variație a curburii elementului, M-Φ. De asemenea pentru determinarea practică a momentului de fisurare, lucrarea prezintă relațiile de calcul recomandate în normele europene şî nord americane.*

**Keywords:** Experimental study, Cracking behavior, Reinforced concrete beams.

## 1. Introduction

When studying the crack behaviour of singly reinforced concrete beams under external loads, there are determined the stress, respectively the forces that are recorded in the transversal section.
The stages that have to be studied are:

I. The bending moment when the first recorded crack appeared and when the tension concrete strength is reached, $M_{cr}$
II. The crack stabilization
III. The ultimate bending moment, $M_u$.

## 2. Hypothesis regarding the stress evaluation

To write the equilibrium equations the following hypothesis are taken in account:

- The concrete and the steel are considered homogenous and isotropic materials
- the concrete shrinkage, due to the temperature changes, is neglected;
- the strain-stress relations are considered according to EC2 (SR EN 1992-1-1:2004);

[*] Corresponding author: Tel./ Fax.: 0264 594967
E-mail address: catinca.letia@bmt.utcluj.ro

- the concrete and steel bond varies parabolicaly as presented in Fig. 1.;
- the Bernoulli hypothesis regarding the plane sections that remain plane after the appearance of the first crack and after cracking, is considered to be valid;
- there is a compatibility between the concrete and steel strain.

## 3. Forces acting in the uncracked transversal section

In this stage the stress and strain in the transversal section are smaller that the necessary one to form the first crack. The concrete tension stress, $f_t$, has not reached the value of the rupture modulus, $f_r$.

The bending moment, M, can be estimated taking in account $\varepsilon_{c1}$ and $x_1$.

The equilibrium of the horizontal forces can be written as it follows:

$$A_{sc} \cdot f_{sc} + F_c = F_t + A_{st} \cdot f_{sl} \quad (1)$$

The bending moment equiation toward the neutral axis is:

$$M = F_c \cdot x_{c1} + A_{sc} \cdot f_{sc}(x_1 - d_2) + F_t \cdot x_{t1} + A_{st} \cdot f_{s\lambda} \cdot (d - x_1) \quad (2)$$

where:
$A_{st}$ – the tensioned steel reinforcement aria;
$f_{s\ell}$ – the steel tension stress;
$x_{c1}$, $x_{t1}$ – the position of the compression, respectively the tension resultant force;
$A_{sc}$ – the compressed steel reinforcement aria, (if there is);
$f_{sc}$ – the steel stress, in the compressed reinforcement;
$d_2$ – the distance between the compressed reinforcement and the compressed concrete face;
$F_c$, $F_t$ – the resulting compressive, respectively tension concrete force.

M- the bending moment;
$\varepsilon_{c1}$- the maximum compression strain;
d- the effective depth of the transversal section;
$x_1$- the copression block heigh, the neutral axis position.

The concrete compression resulting force will be:

$$F_c = \int_{y=0}^{y=x_1} b \cdot \int(\varepsilon)dy \quad (3)$$

It was taken in account that: $\int(\varepsilon) = \sigma_c$, where $\sigma_c$ will be determined using the following relation:

Hognestad (1951)[60]

$$\sigma = f_c' \cdot \left[ 2 \cdot \left( \frac{\varepsilon}{\varepsilon_{c0}} \right) - \left( \frac{\varepsilon}{\varepsilon_{c0}} \right)^2 \right] \quad (4)$$

where: $f_c$'- the cylinder compressive strength;
$\varepsilon$- the strain that corresponds to $\sigma$ stress;
$\varepsilon_{c0}$- the strain that corresponds to the maximum stress;

$$\varepsilon_{c0} = 2 \cdot \frac{f_c'}{E_{it}}; \quad (5)$$

$\varepsilon_{c0} \le 0{,}0038$.
$E_{it}$- the tangent modulus of elasticity.

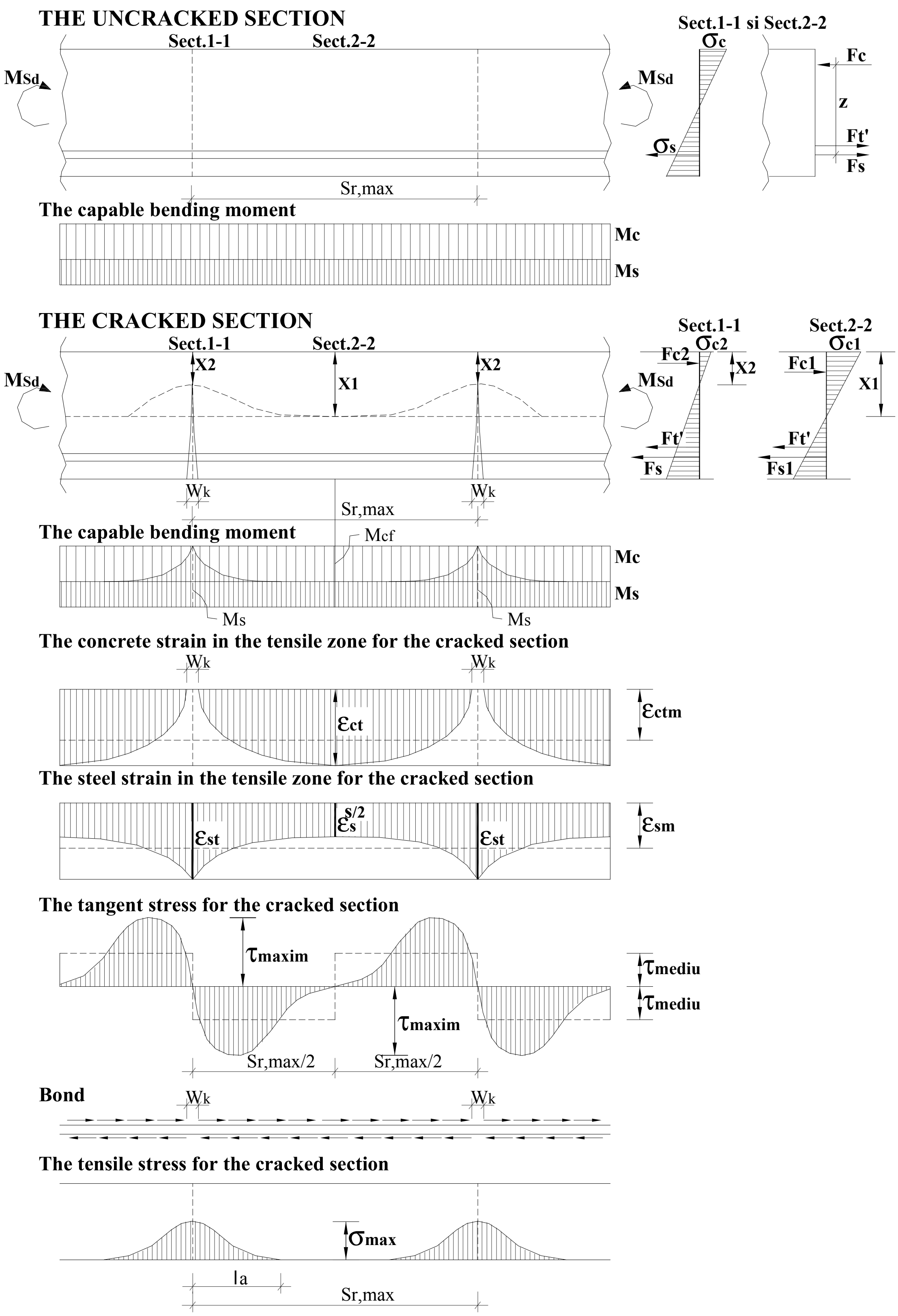



Fig. 1 Stress and strain befor and after cracking

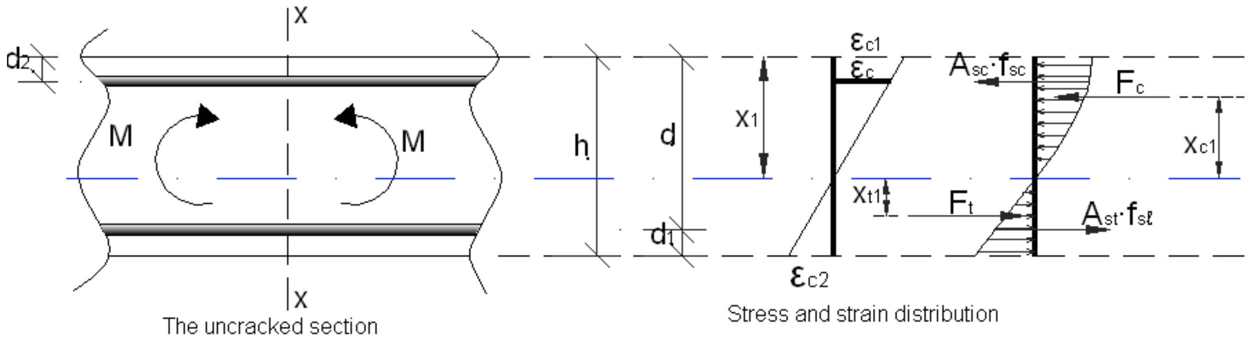



Fig. 2 The stress and strain distribution in the uncracked transversal section- stage I

In the first case, when $0 \leq \varepsilon_c \leq \varepsilon_{c2}$, $F_c$ becomes:

$$F_c = \int_{y=0}^{y=x_1} b \cdot f_{cd} \cdot \left[2 \cdot \frac{\varepsilon}{\varepsilon_{c0}} - \left(\frac{\varepsilon}{\varepsilon_{c0}}\right)^2\right] dy \tag{6}$$

It will be considered:

$$\varepsilon_c = \frac{y}{x_1} \varepsilon_{c1} \tag{7}$$

$$F_c = \int_{y=0}^{y=x_1} b \cdot f_{cd} \cdot \left[2 \cdot \frac{y}{x_1 \cdot \varepsilon_{c0}} \cdot \varepsilon_{c1} - \left(\frac{y}{x_1 \cdot \varepsilon_{c0}} \cdot \varepsilon_{c1}\right)^2\right] dy \tag{8}$$

It follows:

$$F_c = b \cdot f_{cd} \cdot x_1 \cdot \left[\frac{1}{2} - \frac{1}{3} \cdot \left(\frac{\varepsilon_{c1}}{\varepsilon_{c0}}\right)^2\right] \tag{9}$$

The concrete tension resulting force will be:

$$F_t = \int_{y_1=0}^{y_1=h-x_1} \left(b \cdot E_{ci} \cdot \frac{\varepsilon_{c1}}{x_1} y_1\right) dy \tag{10}$$

where:
$y_1$ – the distance from the neutral axis to the extreme tensile fiber of the beam;
$E_{ci}$ – the modulus of elasticity of the concrete in tension modulul;
h – the transversal section heigh of concrete.

$$F_t = b \cdot E_{ci} \cdot \frac{\varepsilon_{c1}}{x_1} \cdot \frac{(h - x_1)^2}{2} \tag{11}$$

The tension stress:

$$f_{s\lambda} = E_s \left[\frac{d - x_1}{x_1}\right] \cdot \varepsilon_{c1} \tag{12}$$

The compression stress:

$$f_{sc} = E_s \left[\frac{x_1 - d_2}{x_1}\right] \cdot \varepsilon_{c1} \tag{13}$$

where:
$E_s$ – the steel modulus of elasticity.

## 4. The cracking moment of the singly reinforced concrete beams

Generraly, the cracking moment can be written using the expression of the concrete modulus of rupture.

$$x_1 = \left[\frac{\varepsilon_{c1}}{\varepsilon_{c1} + \varepsilon_{c2}}\right] \cdot h \tag{14}$$

where, the tension strain $\varepsilon_{c2}$ when the first crack appears is:

$$\varepsilon_{c2} = \frac{f_r}{E_{ci}} \tag{15}$$

The cracking moment, $M_{cr}$, becomes:

$$M_{cr} = F_c \cdot x_{c1} + A_{sc} \cdot f_{sc}(x_1 - c) + F_t \cdot x_{t1} + A_{st} \cdot f_{s\lambda}(d - x_1) \tag{16}$$

It will be considered $A_{sc} \cdot f_{sc}(x_1 - c)$, when there is reinforcement in the compressed zone.
where:
c – the concrete cover.

$$M_{cr} = \frac{f_r \cdot I_g}{h - x_1} \tag{17}$$

$I_g$- the transversal section moment of inertia.

The moment of inertia of a cracked section can be determined as it follows:
- Considering the modulus of deformation for concrete made with regular aggregates :

$$E_c' = \frac{0,8}{1 + 0,5 \cdot \overline{\varphi} \cdot \nu} \cdot E_c \tag{18}$$

where:

$$\nu = \frac{M_{ld}}{M_{Ed}} \tag{19}$$

$\overline{\varphi}$ - the maximum strain time related concrete characteristics.

According to the EC2 (SR EN 1992-1-1:2004):

$$n_e = \frac{E_s}{E_c'} \text{ - for regular cases} \tag{20}$$

$n_e = \dfrac{E_s}{\psi E_c'}$ - when the contribution of the tensioned concrete is taken in account throught the coefficient $\psi$

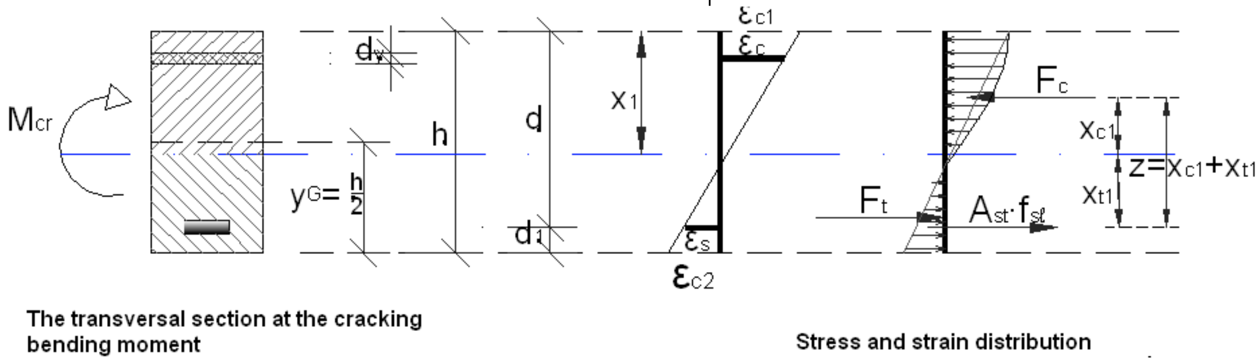



Fig. 3. The stress at the cracking moment

The resulting concrete stress:

$$F_c = \int_{y=0}^{y=x_1} b \cdot \sigma_{cy} \cdot y dy \tag{21}$$

$$F_c = \frac{\sigma_c}{x_1} \int_{y=0}^{y=x_1} b \cdot y dy \tag{22}$$

$$F_c = \frac{\sigma_c}{x_1} b \cdot \left( \frac{x_1}{2} \right)^2 \tag{23}$$

$$F_c = \frac{\sigma_c}{x_1} \underbrace{b \cdot \left( \frac{x_1}{2} \right)^2}_{S_{b,net}} \tag{24}$$

The $F_c$ positio to the neutral axis:

$$x_{c1} = \frac{\int_{y=0}^{y=x_1} b \cdot \sigma_{cy} \cdot y dy}{F_c} \tag{25}$$

$$x_{c1} = \frac{\frac{\sigma_c}{x_1} \int_{y=0}^{y=x_1} b \cdot y^2 dy}{\frac{\sigma_c}{x_1} S_{c,net}} \tag{26}$$

$$x_{c1} = \frac{I_{c,net}}{S_{c,net}} \tag{27}$$

$I_{c,net} = I_c$ the moment of inertia of the compressed zone.

$$z = d - x_1 + x_{c1} \tag{28}$$

It follows:

$$F_t - F_c = 0 \Rightarrow A_{st} \cdot f_{s\lambda} = \frac{\sigma_c}{x_1} \cdot S_{c,net}\,; \qquad \sigma_s = F_{s\lambda} \tag{29}$$

The bending moment equation:

$$M_{Ed} = F_c \cdot z \tag{30}$$

$$M_{Ed} = \frac{\sigma_c}{x_1} \cdot S_{c,net} \cdot z \tag{31}$$

The compatibility ecuations:

$$\frac{\varepsilon_s}{\varepsilon_c} = \frac{d - x_1}{x_1} \tag{32}$$

$$\sigma_s = n_e \cdot \sigma_c \cdot \frac{d - x_1}{x_1} \tag{33}$$

It is known that:

$$z = d - x_1 + \frac{I_{c,net}}{S_{c,net}} \tag{34}$$

Replacing z:

$$M_{Ed} = \frac{\sigma_c}{x_1} \cdot \left[S_{c,net} \cdot (d - x_1) + I_{c,net}\right] \tag{35}$$

But:

$$S_c = A_{st} \cdot n_e \cdot (d - x_1) \tag{36}$$

$$I_{cr} = A_{st} \cdot n_e \cdot (d - x_1)^2 + I_{c,net} \tag{38}$$

where:

$$I_{c,net} = \frac{b \cdot x_1^3}{3} \tag{39}$$

In this case, the moment of inertia will be:

$$I_{ci} = \frac{b \cdot x_1^3}{3} + A_{st} \cdot n_e \cdot d^2 - 2 \cdot A_{st} \cdot n_e \cdot d + A_{st} \cdot n_e \cdot x_1^2 \tag{40}$$

The cracking moment:

$$M_{cr} = \frac{\sigma_c}{x_1} \cdot \left[\underbrace{A_{st} \cdot n_e \cdot (d - x_1)^2 + \frac{b \cdot x_1^3}{3}}_{I_{cr}}\right] \tag{41}$$

To determine the cracking moment it can be used the elastic deformation, M – ø.

$$\phi = \frac{1}{\rho} = \frac{M^{II}}{EI} \tag{42}$$

$$I = \frac{M^{II}}{\phi E_c} \tag{43}$$

where:

$$\frac{1}{\rho} = \frac{\varepsilon_c}{x_1} \tag{44}$$

$$\phi = \frac{\varepsilon_c}{x_1} \tag{45}$$

It follows:

$$I_{cr} = \frac{M^{II} \cdot x_1}{\varepsilon_c \cdot E_c} \tag{46}$$

The cracking moment will be:

$$M_{cr} = \frac{I_{cr} \cdot \varepsilon_c \cdot E_c}{x_1} \tag{47}$$

## 4. Conclusions

EC2(SR EN 1992-1-1:2004) recomands the following relation to determine the cracking moment:

$$M_{cr} = f_{ctm} \cdot W = f_{ctm} \cdot \frac{b \cdot h^2}{6} \tag{48}$$

where:
$f_{ctm}$ – the mean value of axial tensile strength of concrete.

$$f_{ctm} = 0{,}3 \cdot f_{ck}^{2/3} \text{ for concrete classes } \leq C(50/60) \tag{49}$$

$$f_{ctm} = 2{,}12\ln(1)+(fck+8)/10) \text{ for concrete classes } > C(50/60) \tag{50}$$

To determine the cracking moment in the Canadian Codes CSA[85] and the North American Codes ACI2005(2008)[105] the following relation is recomended:

$$M_{cr} = \frac{f_r \cdot I_g}{\underbrace{h - x_1}_{y_t}} \tag{51}$$

where:
$f_r$ – concrete modulus f rupture, according to CSA[85] and ACI[105];
$I_g$ – the moment of inertia of the uncracked section;
$y_t$ – the distance from the neutral axis to the extreme tensile fiber of the beam.

## 5. References


1. EUROCODE 2: Design of concrete structures – Part 1: General rules and rules for buildings - 2004
2. ACI COMMITTEE 363. "Review of ACI Code for Possible Revisions for High Strength Concrete" (ACI 362 R- 92) American Concrete Institute, Detroit, 1992
3. ACI COMMITTEE 318. "Building Code Requirements for Structural Concrete" ACI 318-2005
4. STAS 10107/0-90 – Constructii Civile si Industriale. Calculul si alcatuirea elementelor structurale din beton, beton armat si beton precomprimat
5. MAGUREANU, C. Reinforced Steel - Concrete Bond of High Strength Concrete Proc. Of the first "fib"Congress 2002. Vol. 7. pp. 241- 246
6. MAGUREANU, C, HEGHES, B. Experimental study on ductility reinforced concrete beams using high- strength concrete. The 2-nd International "fib" Congress- Naples- 2006.7.
7. Leția, L.-C. Cracking Behaviour at Bending and Mechanical Properties of High Strength/ High Performance Concrete, Section:Civil Engineering-Architecture, vol.II, Acta Technica Napocensis, 2008, pp. 107-114..7.
8. ACI COMMITTEE 318. "Building Code Requirements for Structural Concrete" ACI 318-2008
9. Lawrance H. Martin, John A. Purkiss- " Concrete Design to EN 1992" Second Edition, Ed. ELSEVIER, 2006
10. E. Hognestad- "A study of combined bending and axial load in reinforced concrete member", Buletin Series No. 399, University of Illinois, Engineering Experimental Station, Champaine, Illinois, 1951, 128 pp.
11. R. Piyasena- " Crack Spacing, Crack Width and Tension Stiffening Effect in Reinforced Concrete Beams and One-Way Slabs", A thesis in fulfilment of the requirements for the award of the degree of Doctor of Philosophy, Griffith University Goald Coast Campus, Nov., 2002
12. I. Nicula, T. Oneț- " Beton armat", Editura Didactică şi Pedagogică, Bucuresti, 1982.

13. I. Tertea,T. Oneț, M. Beuran, V. Păcurar- " Proiectarea betonului armat", Editura Didactică şi Pedagogică, Bucuresti, 1975.

14. Maghsoudi and Akbarzadeh Bengar – " Ductility of HSC members", Iranian Journal of Science & Technology, Transaction B, Engineering, Vol. 31, No. B2, pp. 209-223, Printed in The Islamic Republic of Iran, 2007

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# The High Performance Concrete Elements Calculus Using Eurocode 2

R. Olar [*1], T. Oneţ[2]

*[1,2] Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*This paper presents the general calculus algorithms, very useful taking into account that the actual automatic programs used in structural designing are limited to C50/60 concrete classes. A comparative analysis regarding the utilisation of different concrete classes to multistored structures is detailed in the authors PhD-thesis.*

**Keywords:** High performance concrete, Eurocode 2, Automatic programs, Structural design.

## 1. Introduction

Following the european design rules, the structure must assure:

- adequated performances on all possible actions,
- durability,
- the damages values under exceptional actions must be controlled.

To optimize the building's elements design process using the new materials and using the european design rules, we must have the proper design algorithms to control all the parameters involved. Hereby, in this paper we present the design algorithms for the high performance concrete beams and columns reinforced with S500H steel.

The design characteristics for concrete and reinforcements are gived in the tables 1 and 2.

Table 1. The concrete design characteristics.

| | | C12 /15 | C16 /20 | C20 /25 | C25 /30 | C30 /37 | C35 /45 | C40 /50 | C45 /55 | C50 /60 | C55 /67 | C60 /75 | C70 /85 | C80 /95 | C90 /105 | C100 /115 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $f_{ck}$ | [N/mm$^2$] | 12 | 16 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 | 70 | 80 | 90 | 100 |
| $f_{ctm}$ | [N/mm$^2$] | 1,6 | 1,9 | 2,2 | 2,6 | 2,9 | 3,2 | 3,5 | 3,8 | 4,1 | 4,2 | 4,4 | 4,6 | 4,8 | 5,0 | 5,2 |
| $f_{ctk0,05}$ | [N /m$^2$] | 1,1 | 1,3 | 1,5 | 1,8 | 2,0 | 2,2 | 2,5 | 2,7 | 2,9 | 3,0 | 3,1 | 3,2 | 3,4 | 3,5 | 3,7 |
| $f_{ctk0,95}$ | | 2,1 | 2,5 | 2,9 | 3,3 | 3,8 | 4,2 | 4,6 | 4,9 | 5,3 | 5,5 | 5,7 | 6,0 | 6,3 | 6,6 | 6.8 |
| $E_{cm}$ | [kN/mm$^2$] | **27** | **29** | **30** | **31** | **33** | 34 | 35 | 36 | 37 | 38 | 39 | 41 | 42 | 44 | 45 |

[*] Corresponding author: Tel./ Fax.: 0264 594967

Table 2. The reinforcements design characteristics.

| S500H | S1 |
|---|---|
| $f_{yk}$ [N/mm$^2$] | 500.00 |
| $f_{yd}$ [N/mm$^2$] | 434.78 |
| E [kN/mm$^2$] | 200.00 |
| $\gamma_s$ [daN/m$^3$] | 7850.00 |
| $\varepsilon_s$ [‰] | 2.20 |

## 2. Bended rectangular sections simple reinforced

For the simple reinforced bended section, EC2 give the following relations:

$\lambda=0.8$ for $f_{ck}\leq 50$MPa (1)

$\lambda=0.8-(f_{ck}-50)/400$ for $50<f_{ck}\leq 90$MPa (2)

and

$\eta=1.0$ for $f_{ck}\leq 50$MPa (3)

$\eta=1.0-(f_{ck}-50)/200$ for $50<f_{ck}\leq 90$MPa (4)

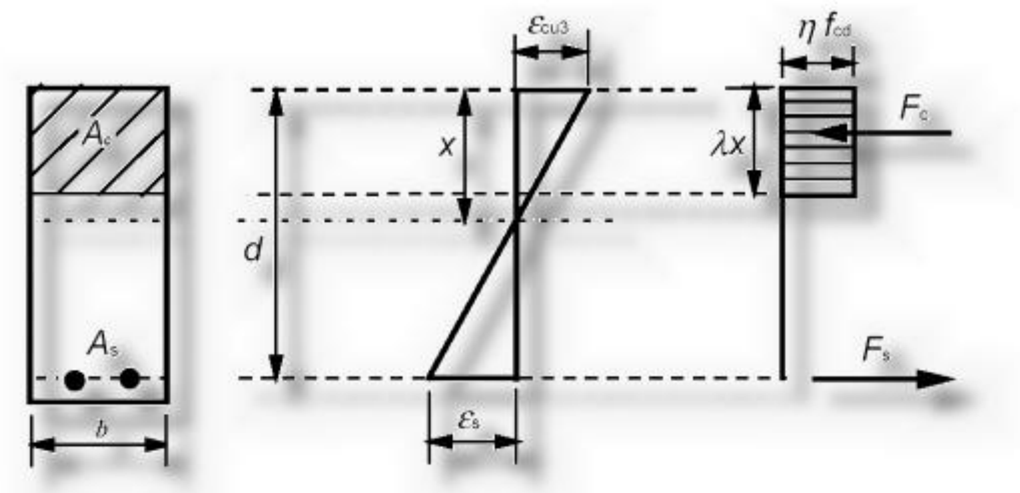



Fig. 1. Simple reinforced bended section.

The equations at equilibrium state are:

$$M = F_c \cdot z = F_s \cdot z \quad (5)$$

where: $z = d - \lambda \cdot x/2$ represent the resultant forces lever arm of $F_c$ şi $F_s$.

$$F_c = \lambda \cdot x \cdot \eta \cdot f_{cd} \cdot b \quad (6)$$

and $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c$ .

The coefficients values of λ and η are gived in the following table:

Table 3. The values of λ and η as concrete classes function.

| Clasa beton | C12 /15 | C16 /20 | C20 /25 | C25 /30 | C35 /45 | C40 /50 | C45 /55 | C50 /60 | C55 /67 | C60 /75 | C70 /85 | C80 /95 | C90 /105 | C100 /115 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| λ | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.79 | 0.78 | 0.75 | 0.73 | 0.70 | 0.68 |
| η | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 0.98 | 0.95 | 0.90 | 0.85 | 0.80 | 0.75 |

The usual values are: $\lambda = 0.80$, $\eta = 1$, $\alpha_{cc} = 0.85$ and $\gamma_c = 1.5$ it is obtained:

$M = 0.567 \cdot f_{ck} \cdot b \cdot s \cdot z$ (7)

where $s = \lambda \cdot x$ or $s = 2 \cdot (z - d)$.

$M = 1.134 \cdot f_{ck} \cdot b \cdot (d - z) \cdot z$

(8)

If $K = M/b{\cdot}d^2{\cdot}f_{ck}$ the equation is:

$(z/d)^2 - (z/d) + K/1.134 = 0$

Solving the equation we obtain:

$$z = d \cdot \left[0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right] \tag{9}$$

In equation 5:

$F_s = (f_{yk}/\gamma_s){\cdot}A_s = 0.87{\cdot}f_{yk}{\cdot}A_s$ , for $\gamma_s$=1.15 and

$$A_s = \frac{M}{0.87 \cdot f_{yk} \cdot z} \tag{10}$$

For the general situation, we obtain:

$$z = d \cdot \left[0.5 + \sqrt{0.25 - \frac{K}{2 \cdot \eta \cdot \frac{\alpha_{cc}}{\gamma_c}}}\right] = \frac{d}{2} \cdot \left[1 + \sqrt{\left(1 - 2 \cdot \frac{\gamma_c}{\eta \cdot \alpha_{cc}} \cdot K\right)}\right] \tag{11}$$

$$\frac{\varepsilon_s}{\varepsilon_{cu3}} = \frac{d - x}{x} \tag{12}$$

or
$$x = \frac{d}{1 + \frac{\varepsilon_s}{\varepsilon_{cu3}}} \tag{13}$$

In the ultimate limit state, the concrete compression strain has the following values:

Tabel 4. The concrete strain values for different classes

| **Clasa beton** | **C12 /15** | **C16 /20** | **C20 /25** | **C25 /30** | **C30 /37** | **C35 /45** | **C40 /50** | **C45 /55** | **C50 /60** | **C55 /67** | **C60 /75** | **C70 /85** | **C80 /95** | **C90 /105** | **C100 /115** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **$\varepsilon_{cu3}$ (‰)** | 3.5 | 3.5 | 3.5 | 3.5 | 3.5 | 3.5 | 3.5 | 3.5 | 3.5 | 3.1 | 2.9 | 2.7 | 2.6 | 2.6 | 2.6 |

The neutral axis position is limited by the values from the table 5:

Tabel 5. Limitarea poziției axei neutre în funcție de clasa betonului

| **Clasa beton** | **C12 /15** | **C16 /20** | **C20 /25** | **C25 /30** | **C30 /37** | **C35 /45** | **C40 /50** | **C45 /55** | **C50 /60** | **C55 /67** | **C60 /75** | **C70 /85** | **C80 /95** | **C90 /105** | **C100 /115** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **$x_{bal}$** | 0.45·d | 0.45·d | 0.45·d | 0.45·d | 0.45·d | 0.45·d | 0.35·d | 0.35·d | 0.35·d | 0.35·d | 0.34·d | 0.33·d | 0.32·d | 0.31·d | 0.30·d |

$$M_{bal} = K_{bal} \cdot f_{ck} \cdot b \cdot d^2 \tag{14}$$

$$K_{bal} = 2 \cdot \eta \cdot \frac{\alpha_{cc}}{\gamma_c} \cdot \left(1 - \frac{z_{bal}}{d}\right) \cdot \frac{z_{bal}}{d} \tag{15}$$

$$z_{bal} = d - \frac{\lambda \cdot x_{bal}}{2} \tag{16}$$

Tabel 6. The values for $z_{bal}=z_{lim}$ şi $K_{bal}=K_{lim}$

| Clasa beton | C12 /15 | C20 /25 | C25 /30 | C30 /37 | C35 /45 | C40 /50 | C45 /55 | C50 /60 | C55 /67 | C60 /75 | C70 /85 | C80 /95 | C90 /105 | C100 /115 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $z_{bal}$ | 0.82·d | 0.82·d | 0.82·d | 0.82·d | 0.82·d | 0.86·d | 0.86·d | 0.86·d | 0.862·d | 0.866·d | 0.876·d | 0.884·d | 0.892·d | 0.900·d |
| $K_{bal}$ | 0.167 | 0.167 | 0.167 | 0.167 | 0.167 | 0.137 | 0.137 | 0.137 | 0.130 | 0.123 | 0.111 | 0.099 | 0.087 | 0.077 |

Tabel 7. The usual values for $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c$

| Clasa beton | C12 /15 | C16 /20 | C20 /25 | C25 /30 | C30 /37 | C35 /45 | C40 /50 | C45 /55 | C50 /60 | C55 /67 | C60 /75 | C70 /85 | C80 /95 | C90 /105 | C100 /115 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $f_{cd}$ | 6.80 | 9.07 | 11.33 | 14.17 | 17.00 | 19.83 | 22.67 | 25.50 | 28.33 | 31.17 | 34.00 | 39.67 | 45.33 | 51.00 | 56.67 |

For the general case, when we know: M, b, d, $f_{yk}$ and $f_{ck}$ and we must calculate $A_s$, we must follow this steps:

- $K = \frac{M}{b \cdot d^2 \cdot f_{ck}}$,
- $K \le K_{bal}$
- $z = d \cdot \left[0.5 + \sqrt{0.25 - \frac{K}{2 \cdot \eta \cdot \frac{\alpha_{cc}}{\gamma_c}}}\right]$,
- $A_s = \frac{M}{0.87 \cdot f_{yk} \cdot z}$,
- $A_s^{eff}$.

## 3. Conclusions

The relations presented above permits the designing of high performance concrete elements using Eurocode 2 rules, for bending action. It can be observed that the calculus relations are different for the high performance concrete from that of normal concrete. This general calculus algorithms are very useful taking into account that the actual automatic programs used in structural designing are limited to C50/60 concrete classes. A comparative analysis regarding the utilisation of different concrete classes to multistored structures is detailed in the authors PhD-thesis.

## 4. References

[1] R.OLAR – "High Performance Concrete – The Design of Structural Elements", The 4th International Ph.D Symposium in Civil Engineering,vol.II, p.88, September 2002, München, Germany.
[2] R.OLAR, T.ONEȚ - "High performance concrete structural elements behaviour, design and composition – peculiarities" Zilele Academice Timişene 22-23 mai 2003. Simpozionul de materiale, elemente şi structuri compozite pentru construcții. p.19-22. Timişoara 2003
[3] T.ONEȚ, R.OLAR - "Beton Armat", Editura U.T.Pres, Cluj-Napoca 2003, republicată în 2004.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# The Multistoried Structures Realised From High Performance Concrete

R. Olar [*1], T. Oneț[2]

[1,2] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*In this paper are presented the aspects regarding the high performance concrete utilisation on tall buildings and the peculiarities of the Eurocode 2 rules utilisation in the design process.*

**Keywords:** High performance concrete, Eurocode 2, Tall building, Structural design.

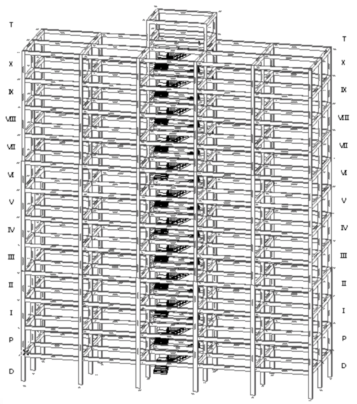
## 1. Introduction

Four tall frame structures realised from concrete of C16/20, C50/60, C90/105 and C100/115 classes and reinforcement S500H. The designing used code was Eurocode2. For a proper results comparation, the same structures geometry was used, the same external loadings, the same compartmentation, the same layer for walls and floors. The structures weights are different and, implicit, the seismic actions are differed. The static and structural design was done for the structures with the following characteristics: two bays, five openings each of them 6.00m, and ten stories with 3.90m height. The zone choosed was the city Cluj-Napoca.

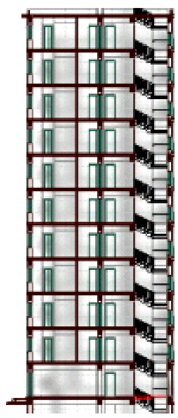
## 2. The structures geometry and shape

In figure 1 is represented the basic structural shape in axonometrical representation, as cross-section and drawing.

a)



b)



c)

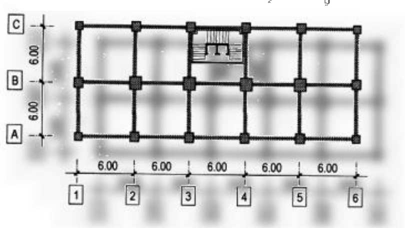



Fig.1. The structures geometry: a) axonometry, b) cross section, c) drawing.

* Corresponding author: Tel./ Fax.: 0264 594967

The seismic zone for Cluj-Napoca is F.

## 3. The materials mechanical properties

The design characteristics of used materials to the analysed structures are gived, as in Eurocode 2, in table 1 for the concrete and in table 2 for the steel:

Table 1. The concrete characteristics.

| | C16/ 20 | C50/ 60 | C90/ 105 | C100/ 115 |
|---|---|---|---|---|
| $f_{ck}$ [N/mm$^2$] | 16.00 | 50.00 | 90.00 | 100.00 |
| $f_{cd}$ [N/mm$^2$] | 10.67 | 33.33 | 60.00 | 66.67 |
| $f_{ctm}$ [N/mm$^2$] | **1.60** | 4.10 | 5.04 | 5.23 |
| $E_{cm}$ [kN/mm$^2$] | **29.00** | 37.00 | 44.00 | 45.00 |
| G [kN/mm$^2$] | **11.44** | 14.91 | 17.45 | 17.97 |
| $\varepsilon_{cu3}$ [‰] | **3.50** | 3.50 | 2.60 | 2.60 |

Table 2. The steel characteristics.

| S500H | S1 | S2 | S3 | S4 |
|---|---|---|---|---|
| $f_{yk}$ [N/mm$^2$] | 500.00 | 500.00 | 500.00 | 500.00 |
| $f_{yd}$ [N/mm$^2$] | 434.78 | 434.78 | 434.78 | 434.78 |
| E [kN/mm$^2$] | 200.00 | 200.00 | 200.00 | 200.00 |
| $\gamma_s$ [daN/m$^3$] | 7850.00 | 7850.00 | 7850.00 | 7850.00 |
| $\varepsilon_s$ [‰] | 2.20 | 2.20 | 2.20 | 2.20 |

## 4. The cross-section dimensions of the structural elements.

After the structural calculus, we obtain the dimensions for the frame elements (beams and columns). The following concluding observations are obtained:

- the cross-sections ratio of the beams for the structures indicate: a diminution with 8.33% at S2 structure than S1 structure, a diminution with 24.24% at S3 structure than S2 structure and a diminution with 10% at S4 structure than S3 structure,
- the corner columns cross-section at P-I-II levels are with 64% smallers at S2 than S1, with 20.98% smallers at S3 than S2 and with 23.43% smaller at S4 than S3,
- the corner columns cross-sections at III-IV-V-VI levels are with 65.97% smallers at S2 than S1, they are equals at S2 and S3, and with 26.53% smallers at S4 than S3,
- the corner columns cross-sections at VII-VIII-IX-X are with 55.55% smallers at S2 than S1, and equals at S2, S3 and S4,
- the marginal columns cross-sections at P-I-II are with 65.39% smallers at S2 than S1, with 19% smallers at S3 than S2 and with 20.98% smallers at S4 than S3,

- the marginal columns cross-sections at III-IV-V-VI are with 58.67% smallers at S2 than S1, with 20.98% smallers at S3 than S2 and with 23.43% smallers at S4 than S3,
- the marginal columns cross-sections at VII-VIII-IX-X are with 51% smallers at S2 than S1, with 26.43% smallers at S3 than S2 and they are equals at S3 and S4,
- the internal columns cross-sections at P-I-II are with 64% smallers at S2 than S1, with 30.55% smallers at S3 than S2 and with 19% smallers at S4 than S3,
- the internal columns cross-sections at III-IV-V-VI are with 65.39% smallers at S2 than S1, with 36% smallers at S3 than S2 and with 23.43% smallers at S4 than S3,
- the internal columns cross-sections at VII-VIII-IX-X are with 55.55% smallers at S2 than S1, with 43.75% smallers at S3 than S2 and they are equals at S3 and S4.

In general, the columns cross-section reductions for the choosed structures are gived by the following values:

- for corner columns: with 62.98% smallers at S2 than S1, with 8.75% smallers at S3 than S2 and with 18.23% smallers at S4 than S3,
- for marginal columns: with 60.01% smallers at S2 than S1, with 21.58% smallers at S3 than S2 and with 17.26% smallers at S4 than S3,
- for internal columns: with 62.89% smallers at S2 than S1, with 35.66% smallers at S3 than S2 and with 16.71% smallers at S4 than S3.

As a consequence of resistance elements cross-sections reduction, the total building weights are reduced as well. Thus, the S2 weight is with 13.29% smaller than S1, the one of S3 is 6.45% smaller than S2 and with 2.70% the one of S4 structure than S3. A direct effect of this weights reduction is the seismic forces reduction for the buildings.

## 5. Concluding remarks.

Based on the obtained results the followings concluding remarqs can be maked:

- the concrete quantities used indicate a decreasing with 34.35% at S2 than S1, with 24.07% at S3 than S2 and a decreasing with 12.36% at S4 than S3,
- the steel quantities used for the beams longitudinal reinforcement shown a decreasing with 13.07% at S2 than S1, a decreasing with 8.74% at S3 than S2 and an increase with 6.01% at S4 than S3,
- the steel quantities used for the columns longitudinal reinforcement shown a decreasing with 6.11% at S2 than S1, a decreasing with 7.32% at S3 than S2 and an increase with 5.93% at S4 than S3,
- the steel quantities used for the beams transversal reinforcement shown a decreasing with 2.35% at S2 than S1, a decreasing with 14.26% at S3 than S2 and a decreasing with 5.46% at S4 than S3,

- the steel quantities used for the columns transversal reinforcement shown a decreasing with 40.98% at S2 than S1, a decreasing with 12.52% at S3 than S2 and a decreasing with 10.32% at S4 than S3,
- generally, the total steel quantities used for the beams and columns reinforcement shown a decreasing with 12.57% S2 than S1, a decreasing with 8.98% at S3 than S2 and an increasing with 3.19% at S4 than S3.

Analysing this final comparative dates we can observe that the S3 and S4 structures are the most atractive by the material economy point of view. This fact, added to the practical space gained by the structural elements dimensions reductions, recommend the high performance concrete to be used to the multistored structures in the Cluj-Napoca area.

## 6. REFERENCES

[1] R.OLAR, T.ONEȚ - "High performance concrete structural elements behaviour, design and composition – peculiarities" Zilele Academice Timişene 22-23 mai 2003. Simpozionul de materiale, elemente şi structuri compozite pentru construcții. p.19-22. Timişoara 2003

[2] T.ONEȚ, R.OLAR - "Beton Armat", Editura U.T.Pres, Cluj-Napoca 2003, republicată în 2004.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Yielding Criteria Used in Geotechnical Engineering Problems

Iulia C. Molnar[*1], A. Popa[2] , O.C.Muresan[3]

[1,2,3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*The paper represents a synthesis of the main yielding criteria used in geotechnical design. The paper describes the main yielding criteria, the yielding surfaces equations and presents a graphical representation for each of the yielding criteria discussed. The main criteria approached are: Mohr-Coulomb Criteria, which relies on a line defined by the Coulomb failure stress and the stress circles of Mohr. The field of failure is given by the cohesion and internal friction angle. The Mohr-Coulomb criterion is based on the assumption that the phenomenon of macroscopic plastic yielding is, essentially, the result of frictional sliding between material particles. Drucker-Prager criterion is a smooth approximation to the Mohr-Coulomb law. The Drucker-Prager criterion states that plastic yielding begins when the invariant of the deviatoric stress and the hydrostatic stress reach a critical combination. Represented in the principal stress space, the yield locus of this criterion is a circular cone whose axis is the hydrostatic line. The first critical state models for describing the behavior of soft soils such as clay, Original Cam-Clay and Modified Cam-Clay were formulated by researchers from Cambridge University. These models describe very important aspects of soil behavior like strength, compression and dilatancy and also critical states.*

## Rezumat

*Lucrarea reprezinta o sinteza a celor mai importante criterii de rupere folosite in probleme de inginerie geotehnica.In lucrare sunt descrise principalele criterii de rupere, ecuatiile suprafetelor de rupere precum si reprezentarea grafica a acestora. Criteriile de rupere abordate sunt urmatoarele: Criteriul de rupere Mohr-Coulomb care se bazeaza pe o dreapta definita de Coulomb si cercurile lui Mohr date de eforturile principale. Cedarea este guvernata de coeziune si unghiul frecarii interne. Criteriul Drucker-Prager este o aproximare fina a modelului Mohr-Coulomb.Conform acestui criteriu, cedarea plastica incepe atunci cand invariantul tensorului deviator al tensiunii si efortul hidrostatic ajung la o combinatie critica. Reprezentand in spatiul eforturilor principale suprafata de cedare Drucker-Prager aceasta este un con a carui axa este axa hidrostatica. Primele modele de stare critica care descriu comportarea argilelor, Cam-Clay Original si Cam-Clay Modificat, au fost formulate de cercetatori de la Universitatea Cambridge. Aceste modele descriu aspecte foarte importante ale pamanturilor cum sunt rezistenta, modificarea de volum care apare datorita forfecarii precum si starile critice.*

**Keywords:** yielding criteria, yielding surface, yielding surface equation

[*] Corresponding author: Molnar Iulia-Consuela: Tel.:0040744 875 191
E-mail address: iulia.molnar@cif.utcluj.ro

## 1. General aspects regarding soil failure

Among the many geotechnical problems that are raised by the actual rate of construction works, an important issue is to ensure the stability of construction in time often having difficult soil conditions.

Behaviour of soil under the action of external loads is characterized by strength and deformability parameters which are dependent on the nature of soil, the intensity and duration of action , the size and shape of the foundation.

Under the action of loads which are transmitted from the construction, in the foundation soil will arise normal and tangential tensions. Foundations will present vertical deformations called settlement of foundations and also horizontal deformations and rotations.

Knowledge of stress and deformation of the foundation soil is necessary to determine the bearing capacity of soil and also to ensure stability of construction during its life.

The distinctive feature of the behavior of soil loaded with sufficient small loads in order to cause linear deformations is that the soil is a linearly deformable medium.

Linear deformation of the soil has a reversible part and also an irreversible part. The reversible part is characterized by the elastic deformation of solid particles, by the elastic deformation of pore water. The irreversible part is characterized by the irreversible deformation of connected water films and solid particles and also by the irreversible deformations given by structural changes, rollings and slidings.

When an external force is applied on a massif and its value exceeds a critical value plastic areas cover a large part of the massif. At the appearance of plastic zones there is a tendency for continuous sliding of particles. Because of this, inside the massif will develop sliding surfaces. Along this surfaces parts from the massif will detach, phenomenon that will lead to collapse.

If the extension of plastic areas is relatively small, in this case we can successfully apply the computational methods of theory of elasticity. For linear deformations areas study we use the methods of theory of elasticity because of the availability of the fundamental relations between tensions and deformations. Using these assumptions we can determine for different loading situations, the distribution of tensions and deformations inside the areas with linear deformations for soil.

When the entire soil massif or at least a good part of it is in critical state, when plastic areas cover a big part of the soil massif we can successfully apply the theory of plasticity. The study of critical state areas can be realized with computational methods of theory of plasticity.

Using different plasticity criteria we can define the stress-strain state that makes the transition from the elastic state to the plastic state. There are different plasticity criteria but the most common ones are those that refer to tensions. According to the most simple tension criteria, the plastic state begins when one of the principal tensions reaches a determined value $\sigma$.

A soil massif is in limit equilibrium state if it is in the point of losing stability.There is also the possibility to have different limit states areas in the soil massif without being in the situation of losing stability [9].

The deformation manner of the soil can be characterized by the following stages: under the action of the external loads that are sufficiently small, the soils present deformations that are directly proportional with the efforts. If the external loads are higher in value, there will be important structural changes in the interior of the soil through the fact that the soil particles change their relative position through rolling and especially sliding, fact that begins to have a continuous character. This is the moment in which plastic deformations appear, being characterized by resulting deformations after a constant effort. When the external loads grow, the areas presenting plastic deformations from the interior of the soil develop progressively.

Because the collapse of a massif has the same stages as other construction materials, we use hypothesis like isotropy and homogeneity, and it emerges the problem of finding a criterion that could define the yielding and the yielding resistance of the soils.

On this basis, it is necessary to have an analysis of different yielding criteria used for the construction materials and the possibility to apply them in the case of soil behavior [11].

## 2. Yield Criteria used in geotechnical engineering problems

### *2.1 Mohr-Coulomb Yield Criteria*

In geotechnical engineering Mohr-Coulomb criteria is used to define the shear strength of soils at different effective stresses. Coulomb's friction hypothesis is used to determine the combination of shear and normal stress that will cause the fracture of the material. Mohr's circle is used to determine which principle stresses that will produce this combination of shear and normal stress, and the angle of plane in which this will occur.

A material failing according to Coulomb's friction hypothesis will show the displacement introduced at failure forming an angle to the line of fracture equal to the angle of friction. This makes the strength of the material determinable by comparing the external mechanical work introduced by the displacement and the external load with the internal mechanical work introduced by the strain and the stress at the line of failure. According to the conservation of energy principle the sum of these must be zero and this will make it possible to calculate the failure load of the soil [3].

According to this criterion, the yield along a plane appears because of a critical combination of normal and tangential tension. The relation between the normal and tangential tension and the shear plane can be written according to Coulomb like this: $\tau = \sigma \cdot tg\phi + c$

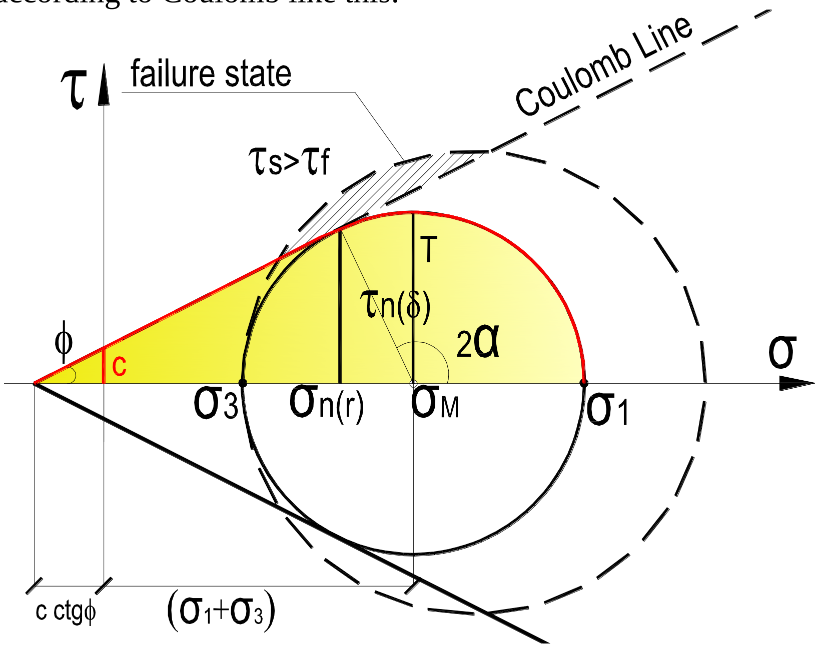



Fig.2.1.1 Mohr-Coulomb Yielding Criterion

The condition of yielding: $F = (\sigma_1 - \sigma_3)^2 - \sin^2 \phi(\sigma_1 + \sigma_3 + 2cctg\phi)^2 = 0$

There are the following 3 situations:

a) Mohr circle placed under the intrinsic line (the tensions $\sigma_1$ and $\sigma_3$ maintain the soil in equilibrium).

b) Mohr circle is tangential to the line of Coulomb (the tensions $\sigma_1$ and $\sigma_3$ induce a limit condition of tension).

c) Mohr circle intersects the line of Coulomb (respectively the tensions $\sigma_1$ and $\sigma_3$ induce an impossible limit condition – of abruption, yield of the soil).

In order to find out in which of these situations is a sample from the soil massif we have to follow this stages:

– the determination of the state of tension elements (normal and tangential tensions)
– the calculus of the main tensions $\sigma_1$ and $\sigma_3$
– the construction of Mohr circle
– the analysis of Mohr circle position in relation with the intrinsic line.[1]

Example: $\tau = \sigma' \cdot tg\phi' + c'$

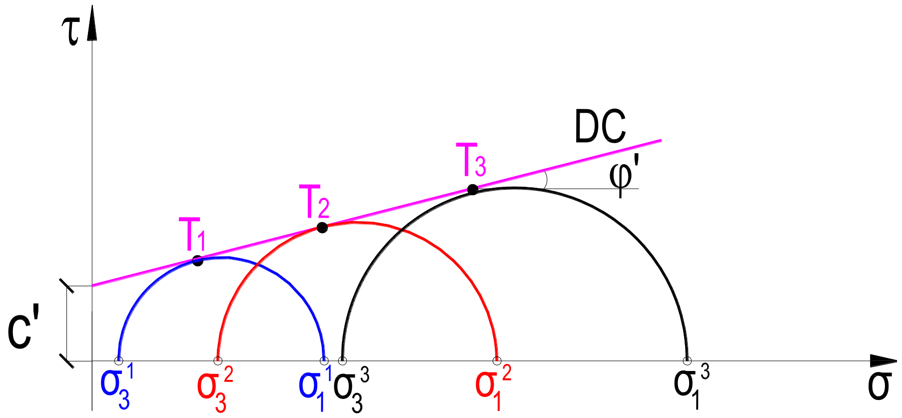



Fig. 2.1.2 The determination of effective parameters (φ',c') by the triaxial test in a C-D open system (consolidated - drained) using the model Mohr-Coulomb

The Mohr-Coulomb yield criterion can be expressed in the (p;q), respectively the (p';q') system. For a non-cohesive or cohesive soil the yield criterion has the graphical expression in which q plays the role of τ, p plays the role of σ and M the role of tgφ. The correspondent of Coulomb line is in this case "The critical state line" (CSL) with the equation: q=Mc·p. This critical state line, as well as the intrinsic line, divides the plane in the domain of impossible tensions state (yielding) and the domain of possible tension state (equilibrium). The tension state from a cylindrical sample are represented by the points Mi having the coordinates σi, τi, which, depending of their position indicate the state of the soil under the actions of (σi, τi). In order to find the coefficient Mc which determines the position of the CSL line we use the plastic yield condition – the critical state given by the Mohr-Coulomb criterion [2] :

$$\left(\frac{\sigma_3}{\sigma_1}\right)_f = \frac{1 - sin\phi_{sc}}{1 + sin\phi_{sc}} \tag{1}$$

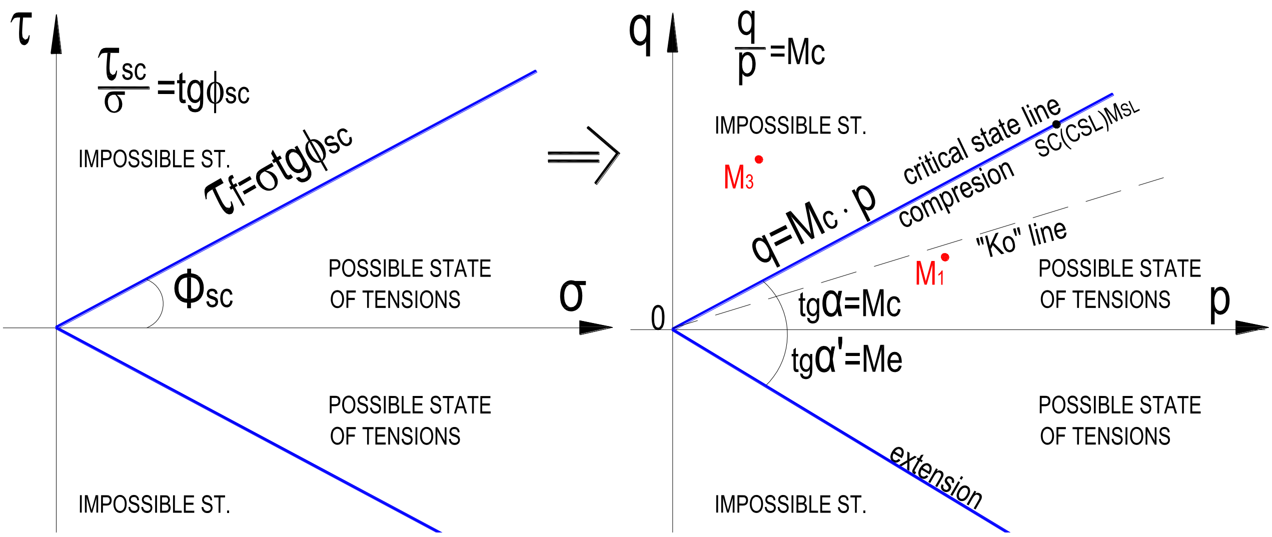



Fig.2.1.3 the Mohr-Coulomb criterion in a modified representation (p,q)

The advantage of this representation (p,q) is that the stress path, respectively the critical state line is determined.

### *2.2 Drucker-Prager Yield Criteria*

Pressure-dependent materials such as soil require advanced models to predict yielding. The Drucker-Prager criterion consists in a change of von Mises criterion by introducing an additional term which depends on the pressure. The Drucker-Prager Model (sometimes known as the von Mises extended model) was proposed to be a fine approximation of the Mohr-Coulomb model [3].

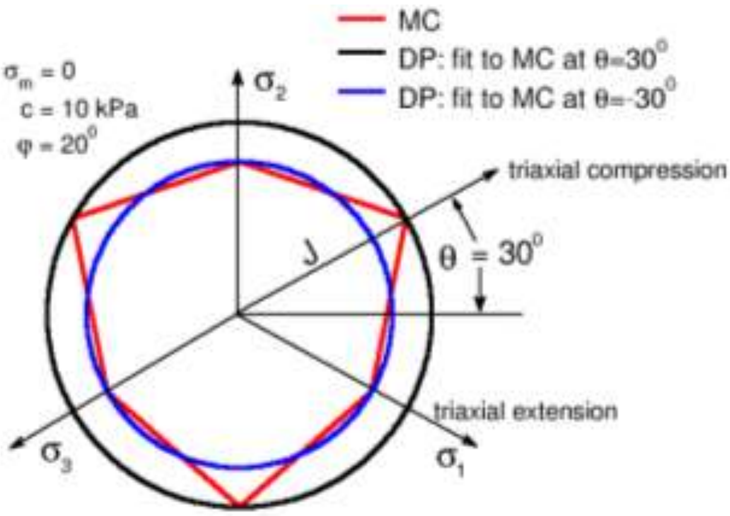



Fig.2.2.1 Mohr-Coulomb and Drucker-Prager yielding surfaces

The model is defined by the yielding surface also called as the Drucker-Prager yielding surface. In the principal stress space $(\sigma_1, \sigma_2, \sigma_3)$ the yielding surface is cylindrical cone whose axis is the hydrostatic axis. The hydrostatic axis makes the same angle with each of the coordinate axis. All the points from the hydrostatic axis have the coordinates $\sigma_1 = \sigma_2 = \sigma_3$ [14].

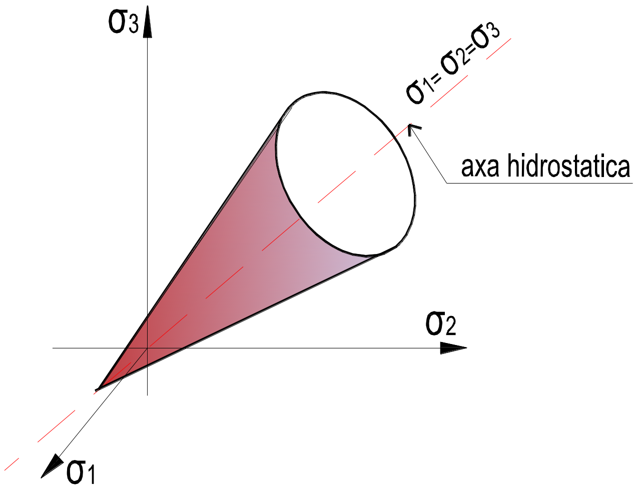



Fig.2.2.2 Drucker-Prager yielding surface in principal stress space

The Drucker – Prager yielding criterion is a model that determines if the material has yield or if it entered in the plastic flow phenomenon. This criterion was introduced in order to solve the problem of soil plastic deformation.

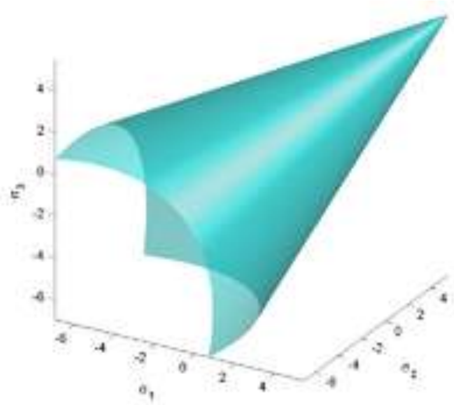


Fig.2.2.3 The Drucker-Prager yield surface of the principal stress for c=2 and φ=-20° in 3D space

The yield criterion has the following form: $\sqrt{J_2} = A + B\ I_1$, where I1 is the first invariant of the stress tensor, J2 is the second invariant of deviatoric stress tensor, A and B are material constants.

Depending on the equivalent stress and the hydrostatic tension, the Drucker-Prager yield criterion can be also written: $\sigma_e = a + b\ \sigma_m$, where $\sigma_e$ is the equivalent stress, $\sigma_m$ is the hidrostatic tension.

The representation of Drucker-Prager criterion depending on the principal stresses σ1, σ2, σ3 is the following:

$$\sqrt{\frac{1}{6}[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2]} = A + B\ (\sigma_1+\sigma_2+\sigma_3)\ . \tag{2}$$

The Mohr-Coulomb hexagonal yielding surface is mathematically indicated only when one of the six facets is indicated for usage purposes. If this information is not known in advance, the hexagon corners may cause considerable difficulties and complications in obtaining a numerical solution. Thus, considering the Drucker-Prager yield surface as a fine approximation of the Mohr-Coulomb yield surface, this surface can be defined considering the cohesion and the internal friction angle, as these parameters are used for the determination of the Mohr-Coulomb yield surface. If we assume that the Drucker–Prager yield surface circumscribes the Mohr–Coulomb yield surface then the expressions for *A* and *B* are [3]:

$$A = \frac{6c\cos\phi}{\sqrt{3}(3+sin\phi)}\ ;\quad B = \frac{2\sin\phi}{\sqrt{3}(3+sin\phi)} \tag{3}$$

If the Drucker–Prager yield surface inscribes the Mohr–Coulomb yield surface then the expressions for *A* and *B* are:

$$A = \frac{6c\cos\phi}{\sqrt{3}(3-sin\phi)}\ ;\quad B = \frac{2\sin\phi}{\sqrt{3}(3-sin\phi)} \tag{4}$$

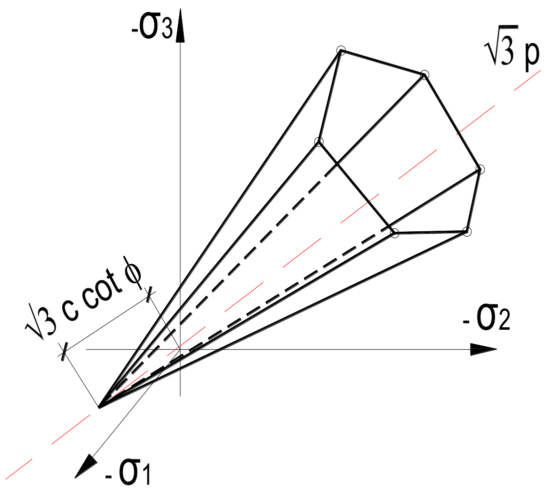



Fig. 2.2.4 The Mohr-Coulomb yield surface.
(p- hydrostatic stress tensor)

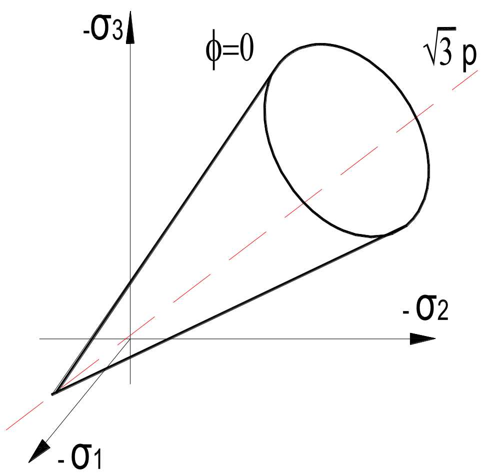



Fig. 2.2.5 The Drucker-Prager yield surface
(p- hydrostatic stress tensor)

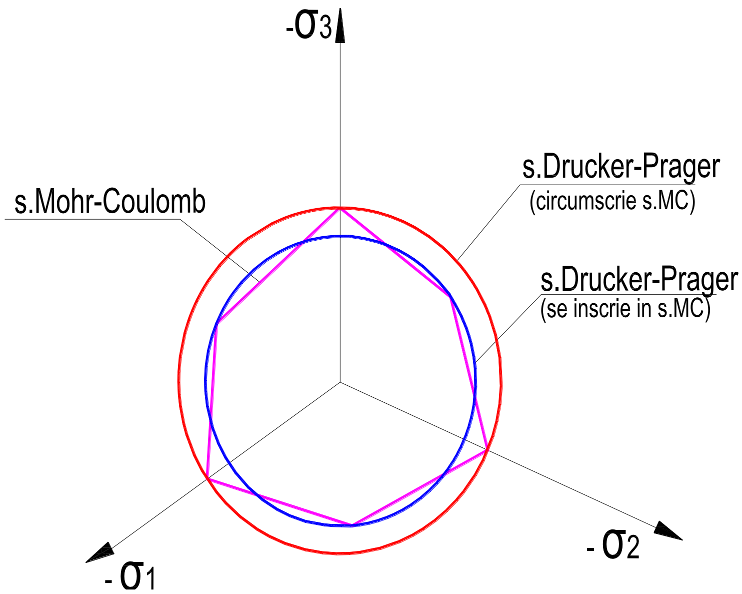



Fig.2.2.6 The Mohr-Coulomb and Drucker-Prager surface in π plane

### *2.3 Cam-Clay Yield Criteria*

At almost the same time with the developing of Con-Cap models, Roscoe (1958) introduced the concept of critical state in soil mechanics. Cam-Clay models relies on the concept of critical state and it was presented by Roscoe and Schofield (1963) and Schofield and Wroth (1968) [12]. The main idea of the Cam-Clay model (Roscoe) or the Critical state model (CSM) is that all soils will yield after a unique yield surface (critical) in the (q, p, e) space, surface that is limited by the critical state line (CSL). The model includes the volume variations and the associated stress states (q, p, e) in defining the yield criterion of soil, as opposed to the Mohr-Coulomb criterion that defines the yield moment considering the stress states (σ; τ or p; q).

The Cam-Clay model or the CSM model considers that, in order to get to a critical state, the soil sample must suffer a compression state defined through the void ratio, only a certain state of stress, characterized by medium stress states (p, q) and also by a certain stress state is considered to be insufficient for the sample to yield by the Mohr-Coulomb model. For the same stress state (σ; τ or p; q), several (n) hardening states (ei, vi) are possible when the sample does not yield if the void ratio is under the critical yield state [2]. In order to analyze the state of a soil inside the semi-space or the semi-plane situated at a certain depth and is under a certain primary state and/or under an active (Δσ) stress, it is necessary to establish a critical yield state, meaning the critical state line (CSL) in space, in the coordinate system (p, q, e or υ).

In the Cam-Clay model, at yielding, usually is adopetd Mc=qf si $d\varepsilon_v^p = 0$ resulting the following relationship for the external work:

$$dW^p = pd\varepsilon_v^p + qd\varepsilon_q^p = Mcd\varepsilon_q^p \tag{5}$$

According to the Cam-Clay yielding criterion, the critical yield surface has the following equation:

$$F(p;q) = (p')^2 - p' + \frac{q^2}{M_c^2} = 0 \tag{6}$$

Considering that the yielding surface is the same with the plastic potential surface we can write the flow rule [12] :

$$\frac{d\varepsilon_p^p}{d\varepsilon_q^p} = \frac{\partial F / \partial p'}{\partial F / \partial q} = M - \eta ; \tag{7}$$

where $\eta = \dfrac{q}{p'}$;

The critical state line (CSL) segment OB2 and the ellipse curve $A_2B_2$ divide the domain in a possible area (point $M_1$) where for any combination of values (p, q) the soil has an elastic behaviour and an impossible area where the soil has a plastic behaviour. For the points on the interface of the two areas, on the ellipse segment that defines the initial flow potential surface it indicates that any combination of values (p, q) that represents the coordinates of the (B) points initiate the yield phenomenon starting with the top resistance and residual resistance, corresponding to the critical state. [2]

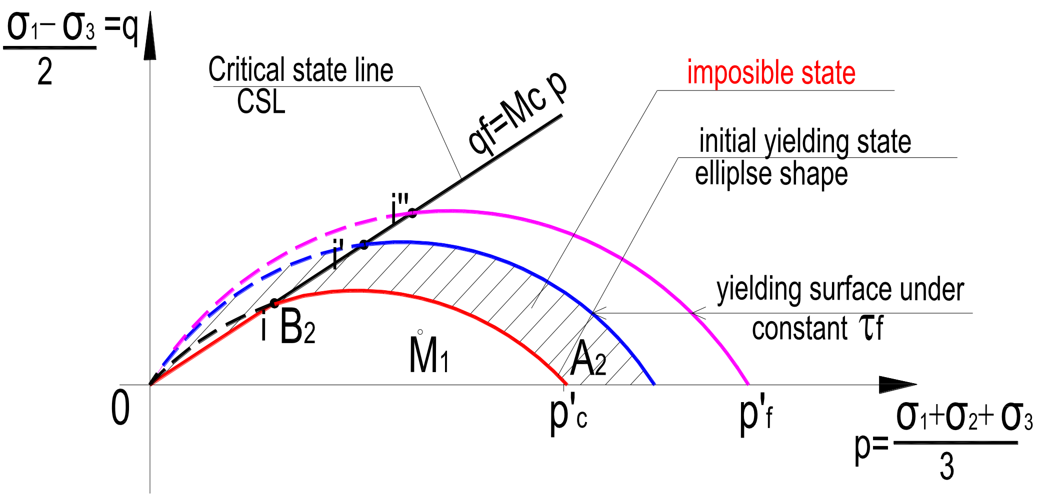



Fig.2.3.1 The approximation of potential yielding surfaces in the (p, q) plane through a line made up of ellipse segments and line segments.

Because of the fact the calculated deformations using Original Cam-Clay Model were bigger than the real ones Roscoe and Burland (1968) modified the yielding surface. This is the way that Modified Cam-Clay model was introduced.

The only difference between the Original Cam-Clay Model and the Modified Cam-Clay Model is the energy equation used in both models which leads to some differences in the relations between the tensions ratio and the increment of deformations ratio.
The following expression is the mechanical work in Modified Cam-Clay model:

$$dW^p = pd\varepsilon_p^p + qd\varepsilon_q^p = p\sqrt{(d\varepsilon_p^p)^2 + (Md\varepsilon_q^p)^2} \tag{8}$$

The yielding surface has an elliptical shape with the following equation [12]:

$$F(p;q) = (p')^2 - p'p_0' + \frac{q^2}{M_c^2} = 0 \tag{9}$$

In p-q space, the Cam-Clay yield surface is a logarithmic curve while the Modified Cam-Clay yield surface plots as an elliptical curve.

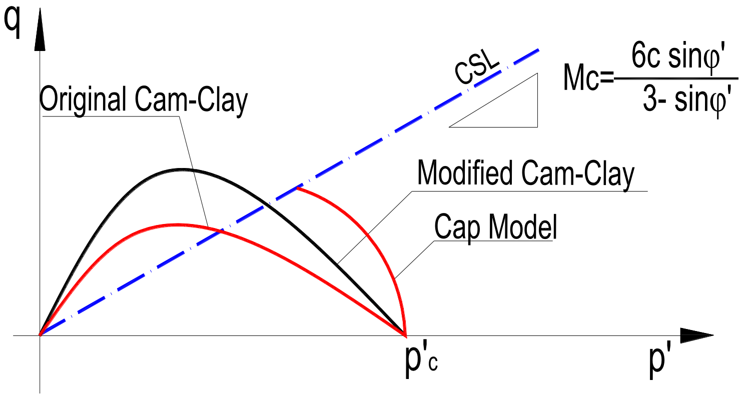



Fig.2.3.2 Original and Modified Cam-Clay Yielding Surfaces. Graphical representation

Both models, Original Cam-Clay Model and Modified Cam-Clay Model describe three important aspects of soil behaviour: strength, compression and dilatancy which are the change of volume that occurs with shearing and critical states in which soil elements can experience unlimited deformations without any changes in stress or volume.

## 3. Concluding remarks

Several representative failure criteria have been reviewed for representing the strength of soils as engineering materials. Initially the Coulomb model is described. Soil behaviour in terms of effective stresses is considered and it is shown that the well known Coulomb failure condition can be extended to give the Mohr-Coulomb and Drucker-Prager models. These conical failure criteria are appropriate for soils with both frictional and cohesive components of shear strength. By introducing the critical state framework for soil behavior and the basic assumptions of the Cam Clay and modified Cam-Clay models, some modifications to these basic formulations are then discussed. Some aspects on the importance of the shape of the plastic potential, in the deviatoric plane, on plane strain behaviour are also revealed. Future work is envisaged to investigate theoretically and numerically various advanced constitutive models able to take into account limited tension capacity, anisotropy of the yield surfaces, kinematic plasticity and strain softening behaviour under undrained conditions.

## Acknowledgements
This paper was supported by the project "Doctoral studies in engineering sciences for developing the knowledge based society-SIDOC" contract no. POSDRU/88/1.5/S/60078, project co-funded from European Social Fund through Sectorial Operational Program Human Resources 2007-2013.

## 4. References


[1] Popa A., *Geotehnică ş i fundaţ ii* Inst.Politehnic Cluj,1981.

[2] Stanciu A., Lungu I., *Fundatii* Ed.Tehnica Bucuresti, 2006.

[3] EA de Souza Neto, D Periæ, DRJ Owen,” Computational Methods for Plasticity: Theory and Applications”,John Wiley and Sons, 2008.

[4] D.C. Drucker, W. Prager, *“Soil mechanics and plastic analysis for limit design”*,1952.

[5] V.Navarro.,B. Garcia *“Optimisation procedure for choosing Cam clay parameters”*, Computers and Geotechnics, 2007.

[6] S.Andrei, I.Antonescu., *Geotehnică ş i fundaţ ii* Ed.Tehnica Bucuresti, 1980.

[7] M.Paunescu, V.Pop, T.Silion , *Geotehnică ş i fundaţ ii* Ed.Tehnica Bucuresti, 1982.

[8] N.Maior, M.Paunescu., *Geotehnică ş i fundaţ ii* Ed.Tehnica Bucuresti, 1973.

[9] F.Gall *“Teza de doctorat- Studiu theoretic si experimental privind folosirea modelelor matematice in evaluarea deformatiilor terenului de fundare”* Iasi, 2003.

[10] I.Gh.*Nas “Teza de doctorat- Contributii la studiul pamanturilor argiloase, legate de modificari interne ce intervin in timp sub actiunea diferitilor agenti si in special a apei”* Bucuresti, 1978.

[11] C.Albert *“Teza de doctorat- Contributii privind influenta rezistentei la forfecare asupra capacitatii portante a terenului de fundare”* Iasi, 2000.

[12] M.Aniculaesi *“Teza de doctorat- Contributii privind interpretarea comportarii terenului de fundare folosind modelul starii critice”* Iasi, 2009.

[13] P.W.Mayne,M.R.Coop, *“Geomaterial behaviour and testing”*17th International Conference on Soil Mechanics and Geotechnical Engineering Alexandria,Egypt;,pp.2777-2872,2009.

[14] D.Rakic, M.Zivkovic, R.Slavkovic.*Journal of the Serbian Society for Computational Mechanics*, Vol.2, pp.80-89, 2008.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Finite Element Analyses of a Multi-Propped Diaphragm Wall

T. A. Hulpus*[1], A. Popa[2]

*[1,2] Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*Rising demand for commercial, residential and industrial needs has driven the architect to consider underground structures in their design. Diaphragm walls have been widely used as primary structural elements for supporting deep excavations in urban area due to their structural advantages. The design of retaining walls and support systems for deep basement construction requires careful analysis, design and monitoring of performance. This is especially critical for deep basement construction in urban areas where the need for space and high land prices justify the deep basement construction The walls should be designed to have high stiffness to comply with strict specifications on the limitation of ground movement induced by excavations in congested urban areas.*

## Rezumat

*Creşterea cererii pentru nevoi comerciale, rezidenţiale şi industriale a condus arhitectul să ia în considerare structurile subterane din punct de vedere al proiectării. Pereţii mulaţi au fost folosiţi pe scară largă ca elemente primare structurale pentru sprijinirea excavaţiilor adânci în mediul urban, datorită avantajelor lor structurale. Calculul sistemelor de sprijin pentru construcţiile subterane necesită o atentă analiză, proiectare şi monitorizare a performanţei. Acest lucru este deosebit de important pentru construcţia de subsoluri adânci în zonele urbane, unde nevoia de spaţiu şi preţurile terenurilor mare justifică construirea de subsoluri adânci. Pereţii ar trebui să fie concepuţi pentru a avea rigiditate mare pentru a se conforma cu specificaţiile stricte cu privire la limitarea deplasării terenului induse de excavaţii în zonele urbane aglomerate.*

**Keywords:** deep excavation, diaphragm wall, finite element method, GFAS, excavation stage

## 1. Introduction

Traditional empirical methods such as those suggested by Peck (1969), Clough & O' Rourke (1990), and Mayne and Kulhawy (1982) cannot always provide reasonable prediction on the deformation pattern in complex modern construction in terms of construction sequence, tie back system and development of pore water pressure. There are many other analytical methods of prediction proposed in the literature for the complex problem but these analyses are not sensitive to the construction procedure, overall stability and movement's pattern in the adjacent soil and effects on adjacent structures.

Finite element model provides an alternative solution to the analysis of diaphragm wall because it provides the possibility to evaluate the deformation pattern developed during the excavation and

* Trifan-Alexandru Hulpus: Tel./ Fax.: 00-40-745-831133
E-mail address: trifan.hulpus@cif.utcluj.ro

the subsequent construction process. Rapid advancement in sophisticated computer industry and the better understanding of constitutive model to represent real soil behavior have made FEM a promising design tool for deep excavation problems.

The use of finite element method for geotechnical analysis requires an accurate modeling of the soil behavior. Many constitutive models have been developed to represent the soil behavior over the past decade, for example, the elastic-perfectly plastic model and hyperbolic model. The selection of the constitutive model to be used in finite element analysis has to consider several issues such as type of constitutive models, construction sequences, soil profile, and wall stiffness prior to an analysis. The selection of the most suitable constitutive model is very crucial in ensuring a safe and economical design.

One of the major considerations in designing diaphragm wall is its influence on adjacent properties that may impact the safety and economic aspect of the project. Numerical analysis using finite element and/or finite difference models can provide information on all design requirements (Mana & Clough 1981, Clough et. al. 1989) such as the pattern and extent of ground settlement at the back of retaining wall, wall displacement, bending moment and shear forces induced to the wall. However, these analyses do have limitations that may results in catastrophic failure if the parameters used in the analysis are incorrectly defined or not accurate.

Diaphragm walls are frequently used as permanent walls of underground car parks due to its structural advantage of being stiff and water-tight as compare to the other wall types. Occasionally, diaphragm walls are given extra strength and stiffness by post-tensioning high strength pre-stressing strands (Catalano et. al., 1994). However, it is not suitable to be used in highly collapsible soil during trenching (Gue, 1998).

The overall stability of retaining walls is often evaluated using the limit equilibrium method of analysis where the conditions of failure are postulated, and a factor of safety is applied to prevent its occurrence. This is to ensure the provision of sufficient embedment depth to prevent overturning of the wall and to ensure overall slope stability. For excavation in soft ground, the Strength Factor Method as recommended by Padfield and Mair (1984) can be used to determine the penetration depth of the wall only. Limit equilibrium slope stability analysis is also carried out to check for both potential circular and non-circular slip failure using Bishop's simplified method and Spencer's method respectively. The Factor of Safety (FOS) adopted is 1.2 for short term and less critical structures while an FOS of 1.4 is adopted for long term or high risk to life structures. If a finite element computer program is available, it can also be used to carry out the check on overall stability. Figure 1 shows the examples of overall stability that need to be checked in design.

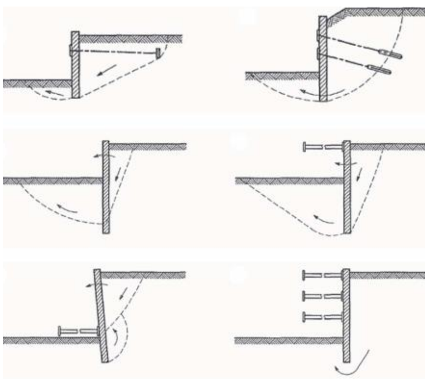


Figure 1. Examples of overall stability failures (extracted from EN1997-1:2004)

## 2. Finite element method (FEM)

The finite element methods is based on the following steps [1]

- Element discretisation:

This is the process of modelling the geometry of the problem under investigation by an assemblage of small regions, termed finite elements. These elements have nodes defined on the element boundaries, or within the element.

- Primary variable approximation:

A primary variable must be selected (e.g. displacements, stresses etc.) and rules as to how It should vary over a finite element established. This variation is expressed in terms of nodal values. In geotechnical engineering it is usual to adopt dlsplacements as the primary variable.

- Element equations:

Use of an appropriate variational principle (e.g. Minimum potential energy) to derive element equations:

$$[K_E]\{\Delta d_E\} = \{\Delta R_E\} \tag{1}$$

where $[K_E]$ is the element stiffness matrix , $\{\Delta d_E\}$, is the vector of incremental element nodal displacements and $\{\Delta R_E\}$ is the vector of incremental element nodal forces.

- Global equations:

Combine element equations to form global equations

$$[K_G]\{\Delta d_G\} = \{\Delta R_G\} \tag{2}$$

where $[K_G]$ is the element stiffness matrix , $\{\Delta d_G\}$, is the vector of incremental element nodal displacements and $\{\Delta R_G\}$ is the vector of incremental element nodal forces.

- Boundary conditions:

Formulate boundary conditions and modify global equations. Loadings (e.g. line and point loads, pressures and body forces) affect $\{\Delta R_G\}$ while the displacements affect $\{\Delta d_G\}$

- Solve the global equations:

The global Equations (2) are in the form of a large number of simultaneous equations. These are solved to obtain the displacements $\{\Delta d_G\}$ at all the nodes. From these nodal displacements secondary quantities, such as stresses and strains, are evaluated.

## 3. The structure

Using finite element method a diaphragm wall of a multi-storey building with 3 underground levels and 10 floors above de ground has been analyzed. The building is in Sibiu city from Romania.

In terms of geological characteristics there were executed geotechnical study and some dynamic penetration super heavy tests (DPSH). Additional were determined soil parameters in Mohr-Coulomb soil model. (Table 1) .

Tablel 1 Values of soil parameteres in Mohr-Coulomb model

| Layer | $\varphi'_k$ [0] | $c'_k$ [kN/m$^2$] | $\psi$[0] | E [kN/m$^2$] | $\nu$ | $\gamma/\gamma_{sat}$ [kN/m$^3$] |
|---|---|---|---|---|---|---|
| (1) fill | 15 | 0.1 | - | 2290 | 0.30 | 17/18 |
| (2) gravely sand | 38 | 0.1 | 8 | 22200 | 0.30 | 19/20 |
| (3) sandly gravel | 38 | 0.1 | 8 | 51000 | 0.30 | 19/20 |

Surcharge load is $q_1 = 10kN/m^2$ and the surcharge load given by an adjacent building placed at ~ 7.35 from the retaining wall is $q_2 = 200kN/m^2$ (Fig 2)

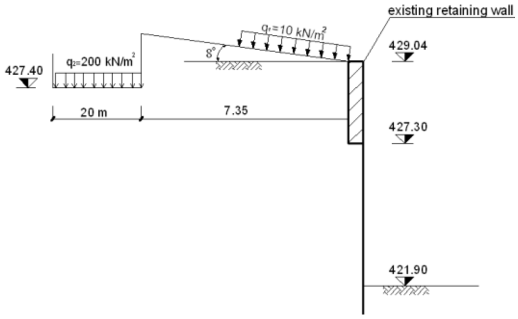



Figure 2. Geometric scheme of the excavation retaining wall

Execution of the retaining wall has been done with „top-down” consctruction method, in different stages. The stages of excavation are presented below in Table 2 and Figure 3

Table 2 Phases of execution of the underground structure

| Phase | Activity description |
|---|---|
| 1. | Open excavation between 427.30 and 424.60 level (second underground level) |
| 2. | Execution of the slab above third underground level, base slab level 424.60 |
| 3. | Further excavation below the slab of the third underground level, between 424.60 and 420.05 level |
| 4. | Execution of the slab above the second underground level and the execution of the raft. 421.90 footing level |
| 5. | Execution of the first underground level wall between 427.30 and 0.00 (430.00) level |

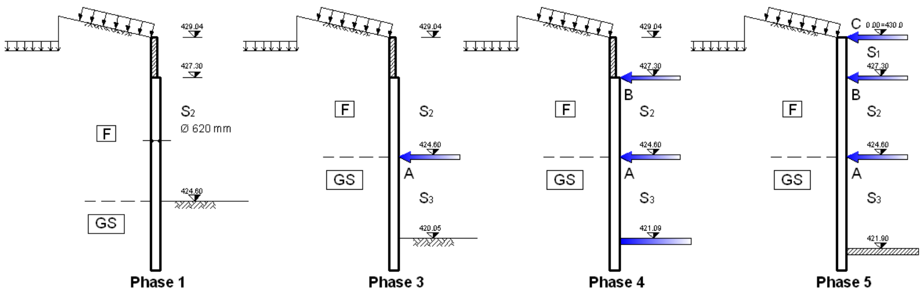



Figure 2. Retaining wall stages of construction

For execution of the retaining wall was chosen the solution with secant piles wall, $d = 620mm$ diameter placed at the distance $a = 900mm$ between their axes.

Water level is at -6.20 m depth from ground level (419.50 m level) with possibility of lifting the level with 0.50 m (420.00 level)

Secant pile wall will be executed in primary piles (concrete piles) class of concrete will be C 6/7.5 and secondary piles (reinforced concrete) class of concrete C 30/37, with the parameters : $E = 31GPa$, $\nu = 0.18$, $\gamma = 25\ kN/m^3$, $d = 0.6m$

Excavation support of each construction stages was made by reinforced concrete slabs with 25 cm thickness, class of concrete C 20/25.

## 4. Calculations

Numerical analysis of the structural model using Finite Element Method (FEM) includes - apart from the diaphragm wall under examination - also the interacting soil and objects in wall environment. The choice of the soil constitutive model is the basic element of FEM analysis. Substantial number of models can be mentioned (Gryczmański 1995), from which the most often applied in geotechnics are elastic-ideal plastic models with associate law of flow and isotropic plasticity surface (e.g. Mohr-, Tresca, Huber - Mieses Hencky, Drucker - Prager) and elastic-plastic models with isotropic strain hardening of volumetric kind (e.g. Cam-Clay, Modified Cam-Clay). Depending on the soil model adopted, different parameters are needed during analysis. In engineering practice the most popular is the elastic-ideal plastic model with Mohr-Coulomb plasticity surface, because of its simplicity and small number of model parameters ($\varphi$, c, E, $\nu$), which can be determined on the basis of laboratory investigation, or in-situ.

Finite element method (FEM) utilizing computer program "GFAS", was used to simulate the basement excavation. GFAS is a finite element package that has been developed specifically for the analysis of deformation and stability analysis in geotechnical engineering problems [2,3,4]. GFAS is an easy-to-use yet powerful geotechnical-engineering tool for the linear and nonlinear analysis of homogenous or non-homogenous structures in which soil models are used to simulate the soil behavior. The analysis procedures are fully automated and based on robust numerical procedures.

GFAS allows for automatic generation of structured and unstructured 2D finite element meshes with options for global and local mesh refinements. Quadratic 8-node and 6-node triangular elements are available to model the deformations and stresses in the soil. The program offers a wide range of support modeling such as liners, anchors and geotextile elements. The beam-column elements in either Bernoulli or Timoshenko theory are incorporated in the code and enabled the user to create complex finite element models in which both plane and line elements interact each other. Besides the stress analysis option the program includes the steady state flow analysis built right into the general program. The water pore pressures are automatically incorporated into the finite element stress analysis. Both initial strain and initial stress methods are incorporated in the program and the nonlinear equilibrium path is determined using incremental iterative procedures under the *force or arc-length* control criterions combined with the full Newton-Raphson approach.

The *Staged Analysis* option can be used to carry out staged construction analysis. During the Staged construction analysis, the loads are increased from 0 to 1, for each stage of construction. As soon as the load parameter reaches the value of 1.0, the constructions stage is completed and the analysis of the current phase is completed, and go the next phase of the construction. If a staged construction calculations finishes while the load factor is smaller than 1.0, the program will stop the analysis. The most likely reason for not finishing a construction stage is that a failure mechanism has occurred. This actually means that no stress distribution can be achieved to satisfy the failure criterion and global equilibrium. Non-convergence within a user-specified number of iterations in finite element program is taken as a suitable indicator of slope failure and is joined by an increase in the displacements. Usually the value of the maximum nodal displacement just after slope failure has a big jump compared to the one before failure.

The program can give information about the deformations at working stress levels and is able to monitor progressive failure including overall shear failure. The present release can be applied only for two-dimensional plain-strain problems. Either the *Mohr-Coulomb* or *Von-Misses* constitutive models can be used to describe the soil or rock material properties.

The Mohr-Coulomb constitutive model is used to describe the soil material behavior. The Mohr-Coulomb criterion relates the shear strength of the material to the cohesion, normal stress and angle of internal friction of the material. For Mohr- Coulomb material model, six material properties are required (Table 3) the friction angle $\Phi$, cohesion C, dilation angle $\Psi$, Young's modulus E, Poisson's ratio $\nu$, unit weight of soil $\gamma$.

Table 3 Mohr-Coulomb parameters with their standard units

| | PARAMETER | UNIT |
|---|---|---|
| E | Young's Modulus | [ $kN/m^2$ ] |
| $\nu$ | Poisson's ratio | [-] |
| $\varphi$ | Friction angle | [ ° ] |
| C | Cohesion | [ $kN/m^2$ ] |
| $\psi$ | Dilatancy angle | [ ° ] |

Plane strain analysis is the most straightforward of the finite element approaches described above, and allows good representation of the pile group configuration and geometry, without being unduly complicated. The equivalent sheet-pile walls were modeled with beam-column elements and the soil strata were represented by 6 nodded triangular elements of elastic-plastic Mohr- Coulomb models. Soil structure interaction was specified by the use of elastic-plastic Mohr-Coulomb model.

The walls and slabs elements have been modeled as a single layer liner (i.e column and beam-column element) with elasto-plastic behaviour, as shown in the Figure 3. The axial and bending moment capacities are described through basic equilibrium, compatibility and material nonlinear constitutive equations (i.e. micro model formulation).

Figure 3 shows the deformation mesh at final stage of excavation where a smaller mesh was used for soil near the excavation pit and bigger mesh used for soil beyond the influence zone. The smaller mesh used for the influence zone is important to produce more accurate result.

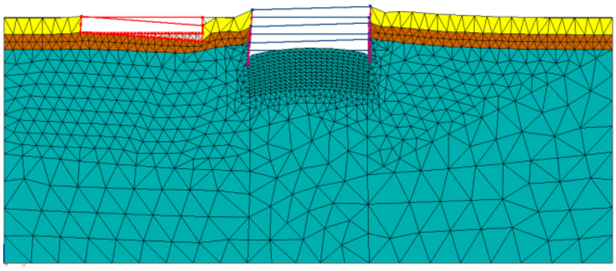
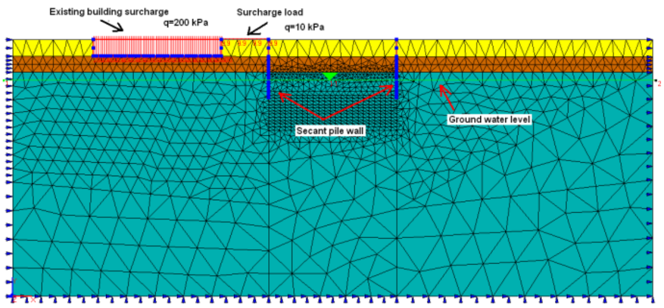
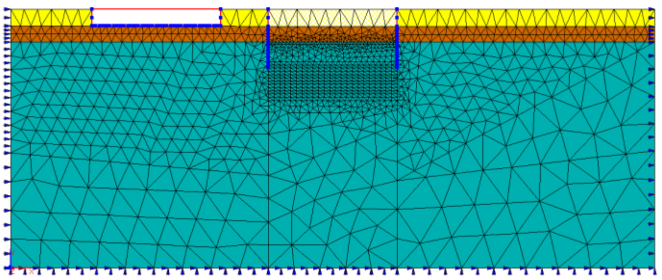


Figure 3. Discretization of finite element mesh

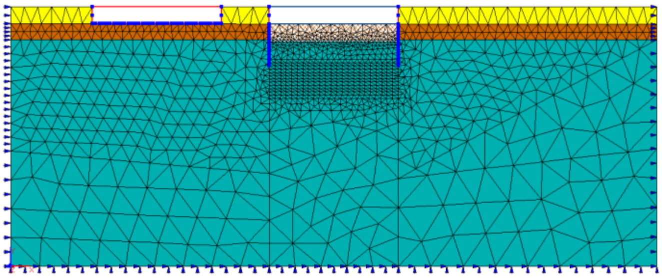
The construction sequence is simulated in four stages as outlined in Table 2  Figure 4 shows the sequence of excavation and installation of strutting to the diaphragm wall.



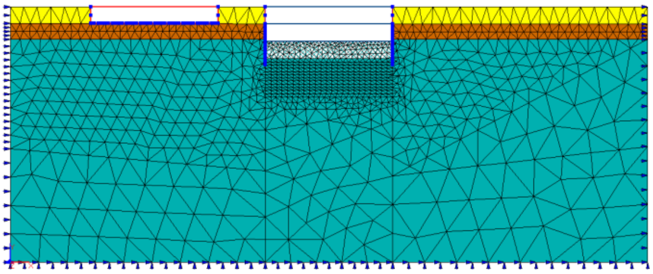

Stage 1



Stage 2



Stage 3



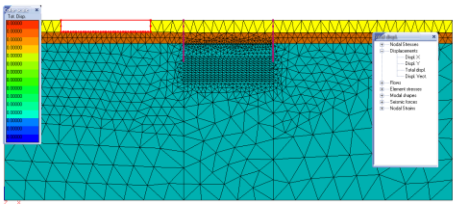
Stage 4

Figure 4. Sequence of excavation and installation of strutting to the diaphragm wall

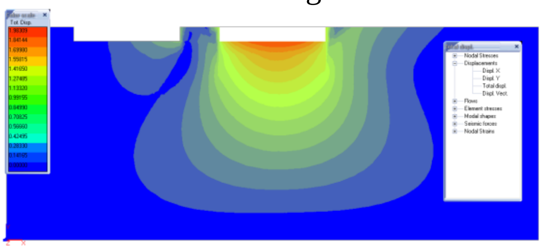
## 4. Results

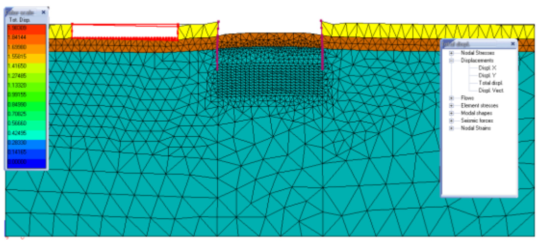
One of the major considerations in designing diaphragm wall is its influence on adjacent properties that may impact the safety and economic aspect of the project. Numerical analysis using finite element and/or finite difference models can provide information on all design requirements (Mana & Clough 1981, Clough et. al. 1989) such as the pattern and extent of ground settlement at the back of retaining wall, wall displacement, bending moment and shear forces induced to the wall. However, these analyses do have limitations that may results in catastrophic failure if the parameters used in the analysis are incorrectly defined or not accurate.

Figure 5 shows the displacements of the structure and the ground during all construction stages.

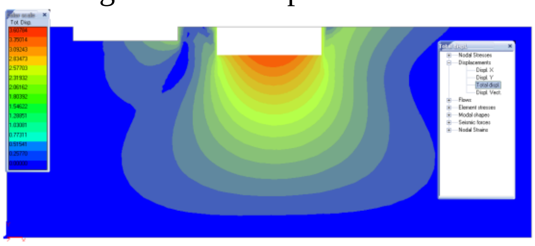


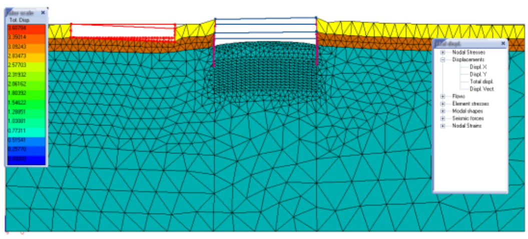
Stage 1



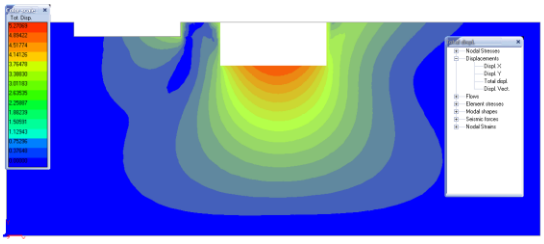


Stage 2 : total displacements deflected shape

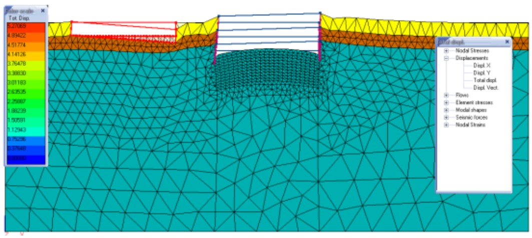




Stage 3: total displacements deflected shape



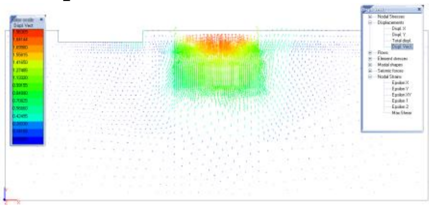
Stage 4: total displacements



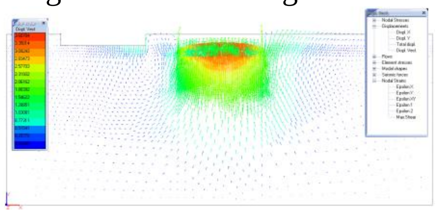
deflected shape

Figure 5. Total displacements of the structure and adjacent ground on construction stages

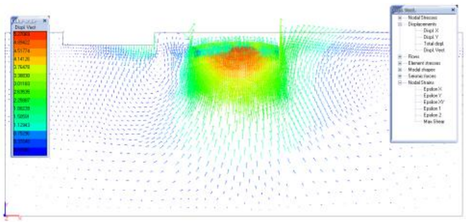
Also the displacements vectors on each construction stage are shown in Figure 6



Stage 2



Stage 3



Stage 4

The displacements resulting from these loadings were reset to zero to correspond with conditions in the field prior to placement of the wall

Report in nonlinear analysis is shown in Figure 7:

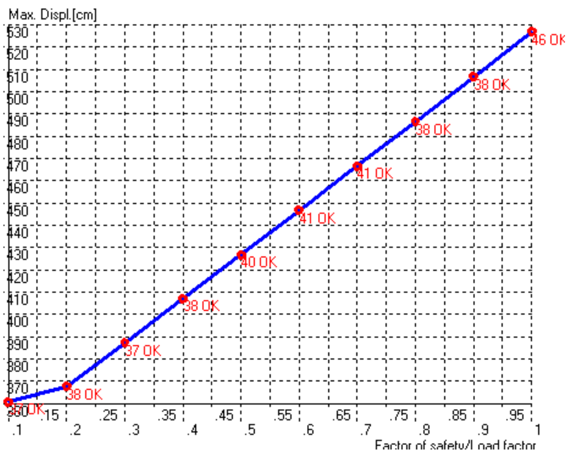
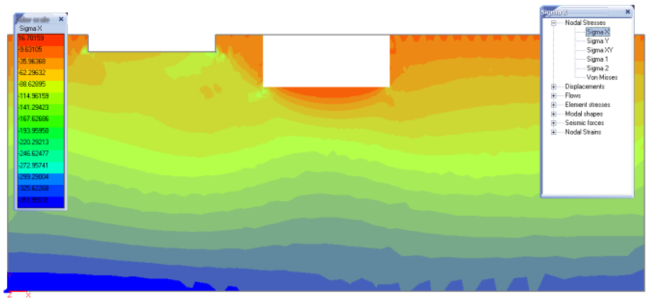
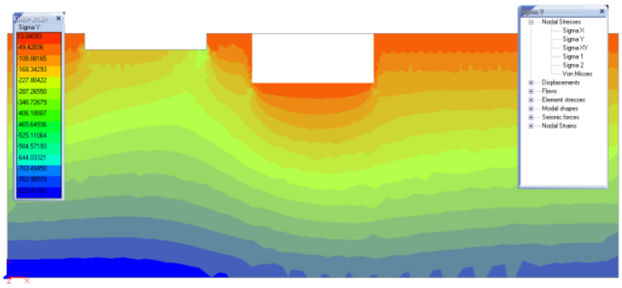



Figure 7. Graph of nonlinear analysis report

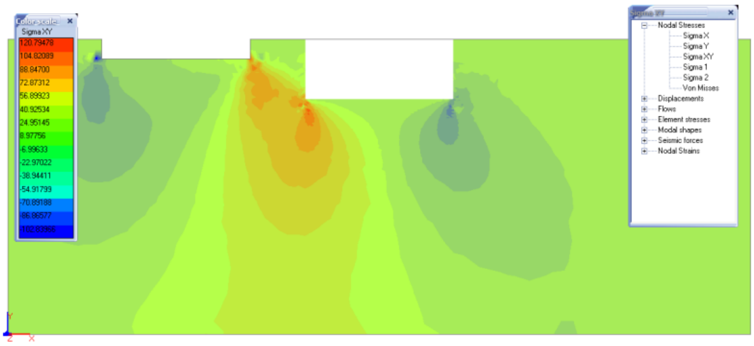
Nevertheless the presence of adjacent structures would modify the state of initial stresses within the ground and could have significant influence on wall behaviour, due to limitation of data provided; simplification of the model in two dimensional plain analyses was undertaken (Figure 8)



a)



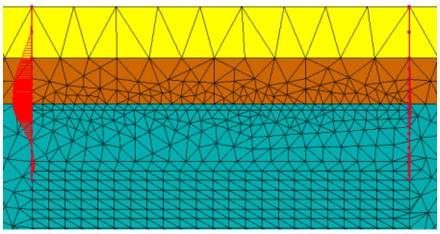
b)



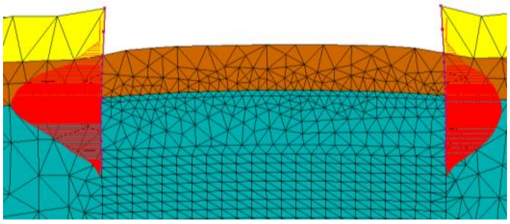
c)

Figure 8. Nodal stresses in the final stage of excavation
a) Sigma X; b) Sigma Y; c) Sigma XY

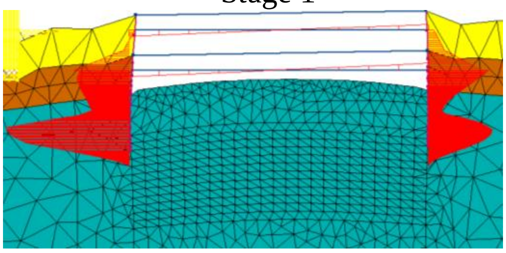
The variation of bending bending moments and shear forces on the wall on each construction stage are shown in Figure 9 and Figure 10



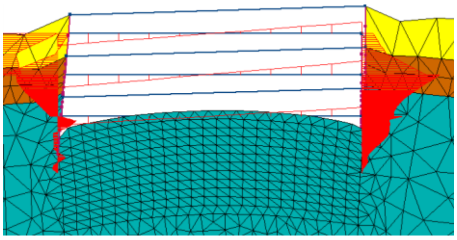
Stage 1



Stage 2

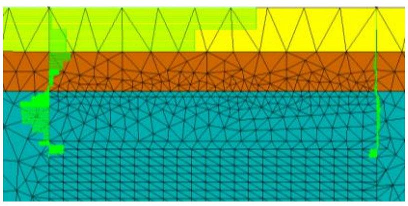


Stage 3

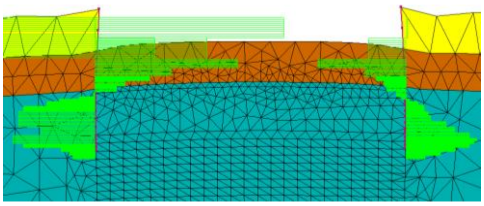


Stage 4

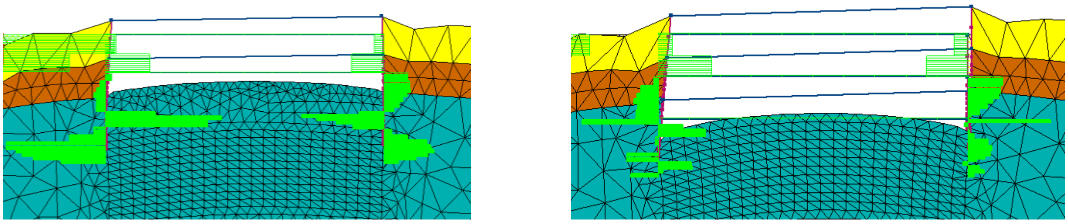
Figure 9. Bending moments diagrams on stages construction



Stage 1



Stage 2



Stage 3 Stage 4

Fig. 10 Shear forces diagrams on stages construction

## 5. Conclusions

Finite element method (FEM) have proved to be of significant help in estimating wall and ground deformations at each stage of excavation. Although significant progress has been made in numerical modelling of deep excavations, some refinements in the selection of appropriate constitutive models which have a significant influence on the results in terms of the overall deformation behaviour and the selection of in-situ stiffness parameters still need to be addressed.

The risk associated with deep basement construction works is high as failures of retaining wall or support systems will be catastrophic and will affect surrounding areas.

As such, the design of retaining walls and support systems for deep basement construction works requires careful consideration of soil-structure interaction and this is usually accomplished using the finite element method (FEM).

However, the use of the finite element method (FEM) requires proper understanding of the limitations associated with the method and also proper modelling of the structures in order to make a representative analysis.

## Acknowledgements

I would like to express my appreciation to Professor Cosmin G. Chiorean for his guidance, support and encouragement throughout the course of this project.

## 6. References

[1] David M. Potts, Lidija Zdravkovic, *Finite element analyses in geotechnical engineering*, Thomas Telford Publishing 1999.

[2] Geostru software, GFAS Finite Element Systems for Geotechnical aplications, Theoretical and User Manual 2010 www.geostru.com .

[3] Chiorean C.G. GFAS Theoretical and User manual, http://bavaria.utcluj.ro/~ccosmin, 2009.

[4] Chiorean C.G., Aplicarea metodei elementelor finite in calculul geotehnic, Curs online, http://bavaria.utcluj.ro/~ccosmin, 2009.

[5] NG HUAT HOE, *Numerical modeling of diaphragm wall in Kuala Lumpur limestone* formation, PhD Thesis 2007.

[6] A.V. Skorikov, D.E. Razvodovsky, I.V. Kolybin, *Behavior of plate foundation in deep excavation beneath 32-storey building in Moscow* Geotechnical Aspects of Underground Construction in Soft Ground Proceedings of the 5th International Symposium TC28. Amsterdam, the Netherlands, 15-17 June 2005

[7] F. H. Lee, *Application of Large Three-Dimensional Finite Element Analyses to Practical Problems,* The 12th International Conference of International Association for Computer Methods and Advances in Geomechanics (IACMAG) 1-6 October, 2008 Goa, India.

[8] Clough, G.W. & O'Rourke, T.D. 1990. *Constructed Induced Movements of In-situ walls.* Proc. ASCE Specialty Conference, Cornell. ASCE Geo Special Publication 25.

[9] Y. C. Tan & C. M. Chow, *Design of Retaining Wall and Support Systems for Deep Basement Construction – A Malaysian Experience*

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# FEM Modeling and Analysis of Precast Large Panels Joints

Alexandru CHIRA*[1]

[1] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*This paper describes a dynamic nonlinear finite element analysis that was made on a precast large panel joint, wich simulates an experimental test on a particular joint. This study is carried out with Abaqus v.6.10 finite element software. The three dimensional model has the exact configuration of the tested specimen as presented in the paper. Several finite element models of the joint were analysed until a model could be validated with the experimental results. The validated numerical model was used to propose additional models in order to obtain a better behavior on the precast large panel the joint.*

## Rezumat

*Această lucrare prezintă o analiză dinamică neliniară, folosind metoda elementelor finite asupra unei îmbinări din panouri mari prefabricate,ce simulează o încercare experimentală. Analiza a fost făcută cu ajutorul programului de element finit Abaqus v.6.10. Modelul tridimensional are aceeaşi configurație cu specimenele testate în experiment după cum sunt prezentate în lucrare. Mai multe modele au fost analizate până la validarea cu rezultatele exeperimentale.Modelul validat a fost folosit pentru propunerea unor noi modele având ca scop obținere unei comportări mai bune a îmbinării din panouri mari prefabricate.*

**Keywords:** precast large panel joint;dynamic explicit nonlinear analysis; finite element analysis; friction; solid elements; beam elements

## 1. Introduction

The aim of this paper is to get an accurate response by modelling a large precast panel joint with finite element method although it implies a high computational cost. At first the modelling of the structure was conducted using only solid elements to avoid possible D.O.F's errors but the amount of time needed for an analysis lead to finding an alternative, wich was a model with solid and beam elements. The hard part of validating the model was the lack of data on the material properties so the stress-strain curves were used from the available literature. The study was made using the finite element code software Abaqus v6.10[2]. The model used in the analysis has the exact configuration as the experimental specimens[1].

## 2. The experimental model

The experimental configuration with its dimensions and reinforcements is presented in figure 1,2

* PhD Student, Tel./ Fax.:+40766619063
E-mail address: alexandru.chira@cif.utcluj.ro

and 3[1]. The A and B parts are precast concrete and the other parts are monolith concrete with a reinforcement grid of Φ6/10,OB37 and independent bars of Φ12,PC52. The B part is subjected to a vertical load starting with an incremental load of 50 kN until it reaches 400 kN, then the incremental load changes to 25 kN until a limit of 725 kN.

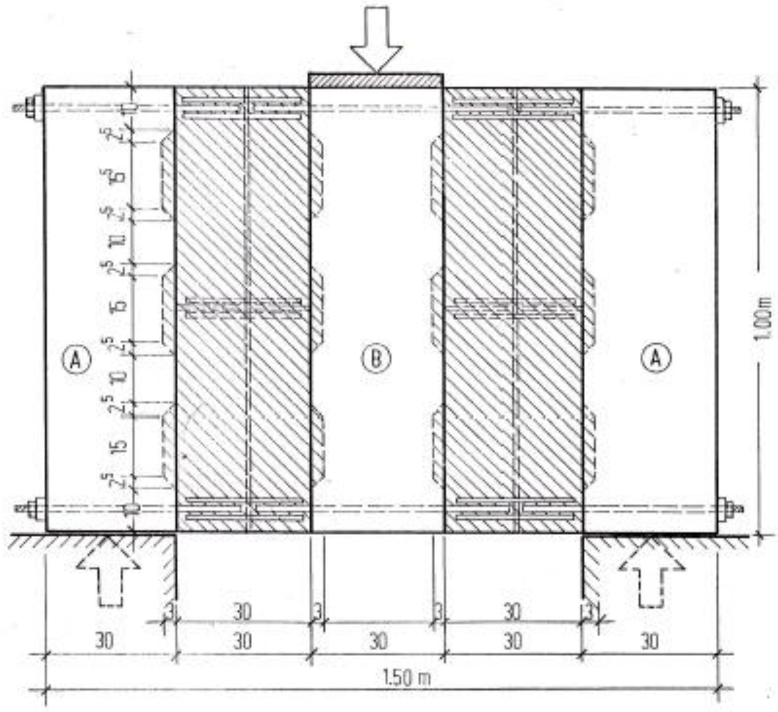



Figure 1. Experimental model.

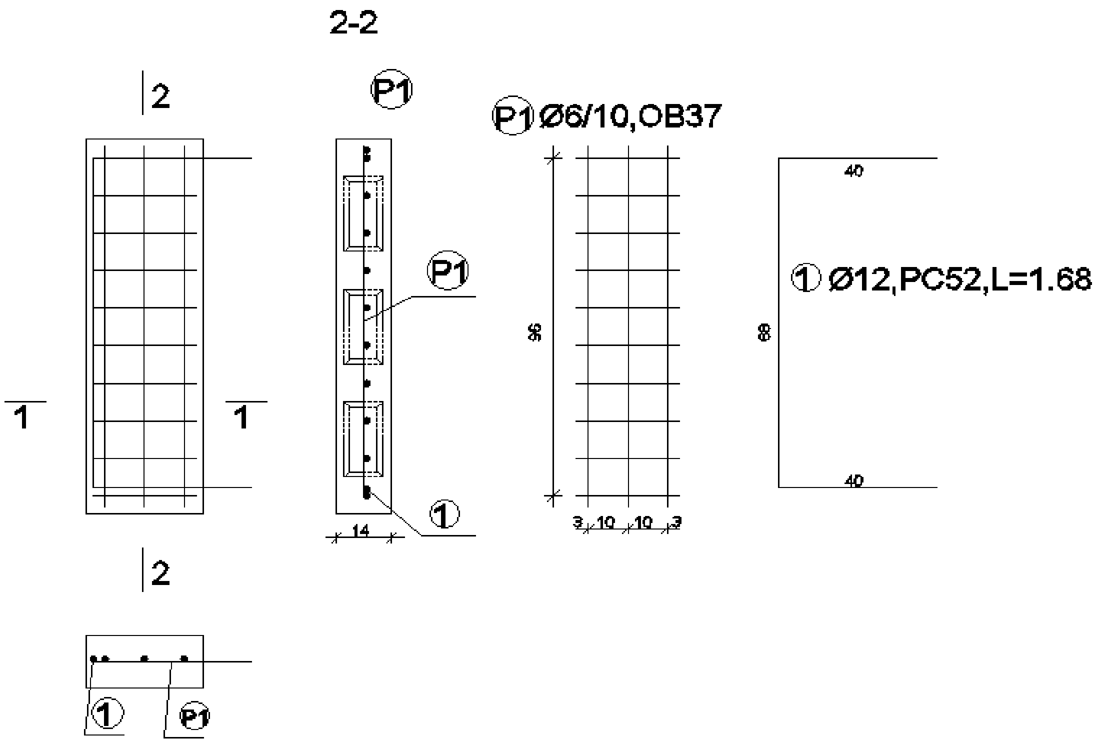



Figure 2.,,A'' element reinforcement.

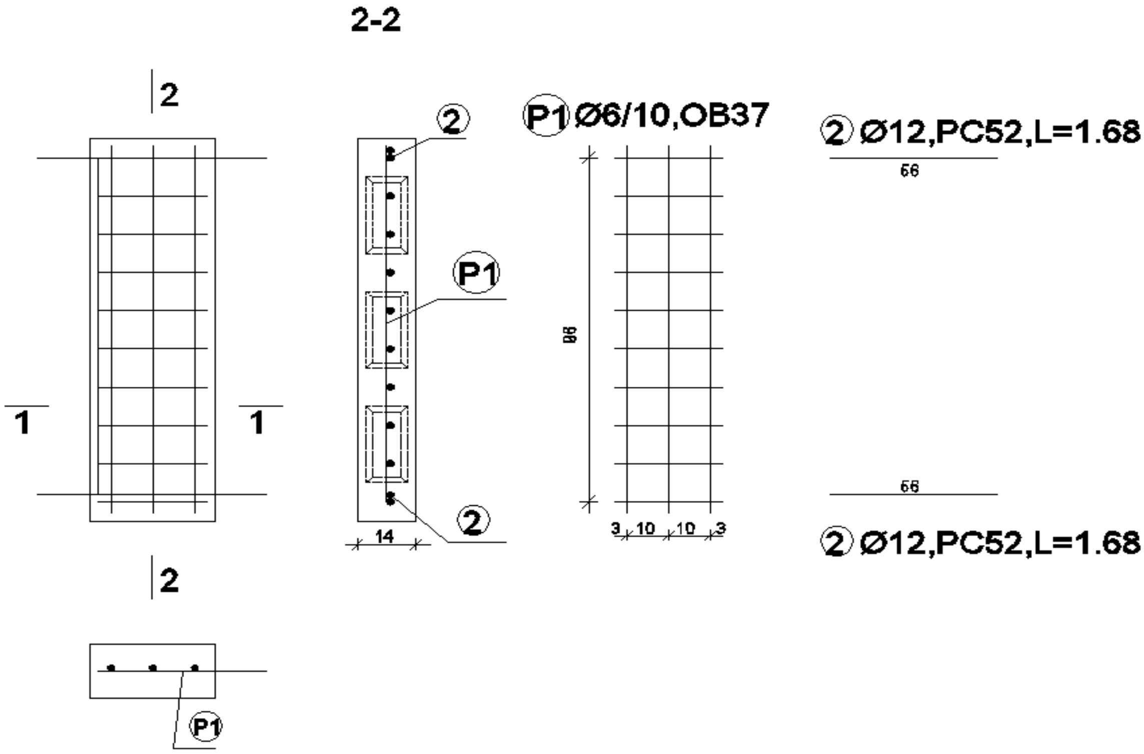



Figure 3.„B'' element reinforcement.

# 3. Finite element model

## *3.1 Geometry and definition of the elements*

The precast and monolith concrete were modeled using C3D8R 8-node solid elements and the reinforcement with B31 beam elements that are embedded in the solid. At first the model was using shell elements with rebar layers wich has the benefit of a low computational cost but it doesn't allow many changes in the spacing and geometry of the rebars. A tet mesh technique was used because of the geometry of the assembly. Three rigid parts were used in the model,for constraints and for applying the load. The assembly of the parts and the mesh are presented in figure 4.

## *3.2 Interactions and boundary conditions*

The interaction between the concrete parts was a surface to surface contact in a general contact property using normal and tangential penalty behaviour with a friction coefficient of 0.3. The reinforcememt is embedded in the solid wich is the host so that the translational degrees of freedom of the reinforcement are following the solid ones. Boundary conditions(encastre) were attached to the reference points of the lateral rigid to wich the parts have a tie constraint. A rigid part was used in the same way with a tie constraint for the middle part wich is subjected to the vertical load,so that the load is transmitted uniform from the rigid to the concrete part.

## *3.3 Materials*

For the concrete material was used a CDM model(concrete damaged plasticity) and an elastic plastic one for the reinforcement.
The properties of the concrete material[1] are:
-E=242880 daN/cm$^2$(Young's modulus)

-fcd=293 daN/cm$^2$ (compressive strength)
-fctm=20.5 daN/cm$^2$(tension strength)
The properties of the reinforcement material are:
-E=2100000 daN/cm$^2$(Young's modulus)
-fyd=3000 daN/cm$^2$

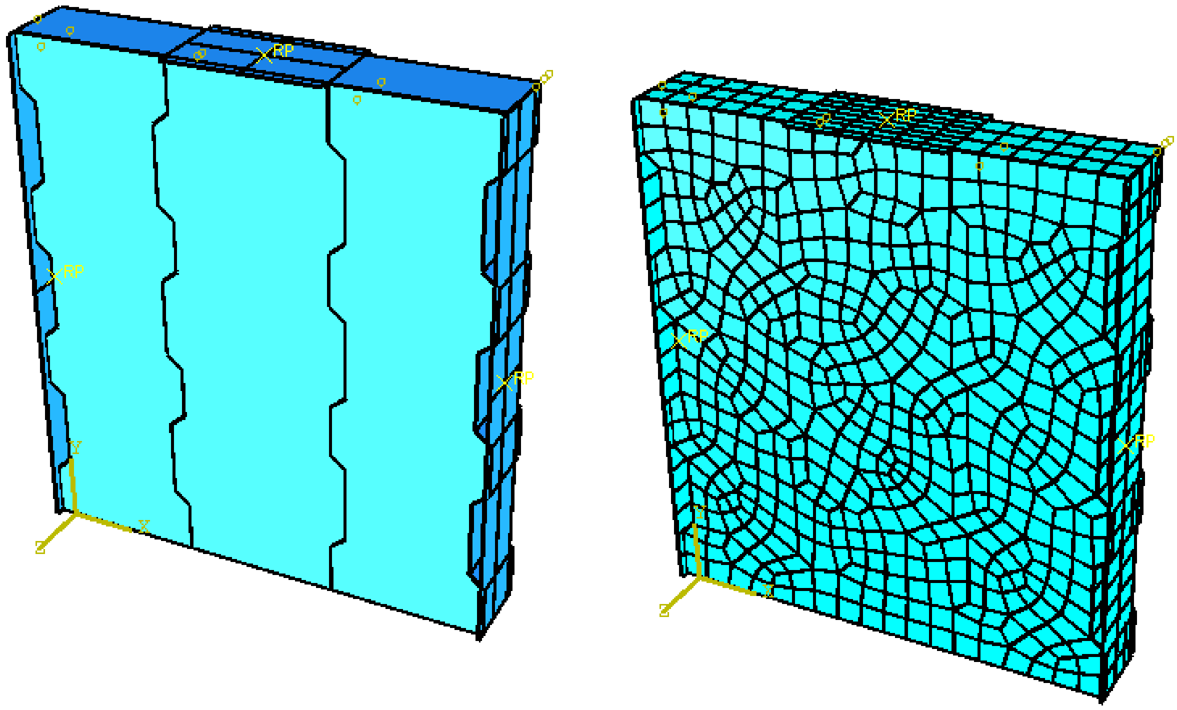


Figure 4.Assembly and mesh of the model

.

### *3.4 Analysis*

A nonlinear dynamic explicit analysis was made using 2 steps. In the initial step the boundary conditions and the contacts are set and in the second the incremental vertical load is applied on the reference point of the rigid part. The amplitude of the force is very important as it was observed from the many analysis that were conducted until validation,so that for a value of 0.3 from the total time of 1 was the maximum amplitude of the force.

### *3.5 Results and model validation*

The model was calibrated using a force-displacement curve obtained from the experimental test[1] as presented in figure 5 where the equivalent results to our model were the RV-II from the three curves in the chart The behaviour of the finite element model  is similar to the experimental one as it could be seen in figure 6(force –displacement curve) and in figure 7(deformed variable). In figure 11 an equivalent plastic strain results of the model was presented so it could be compared with the failure mode of the experimental model(figure 10,RV-II part). A new model was proposed for the same analysis adding one reinforcement(Φ12,PC52) in the middle of the central part( B part) to observe the differences(figure 12). Von Mises stresses are as well presented at the reach of the maximum amplitude,in  figure 8 and figure 9.

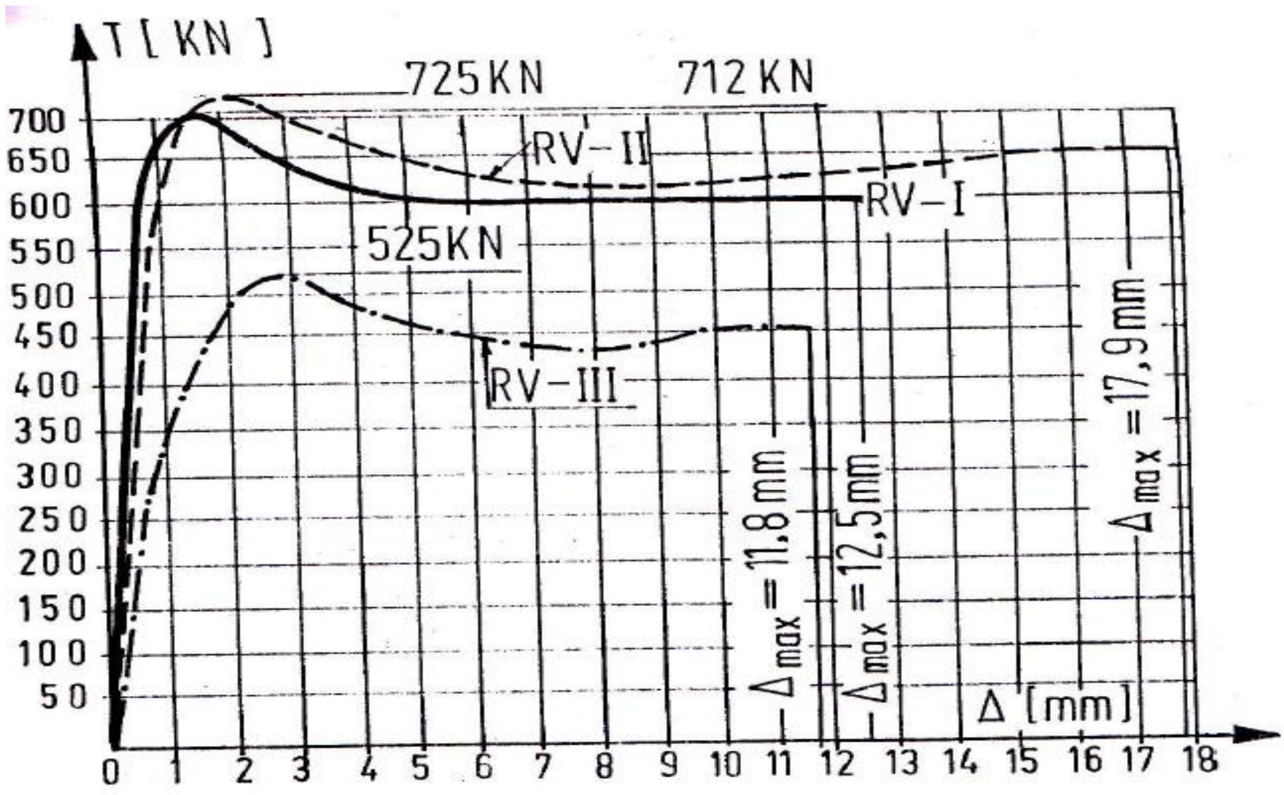



Figure 5.Experimental results of the Force-Displacement diagram

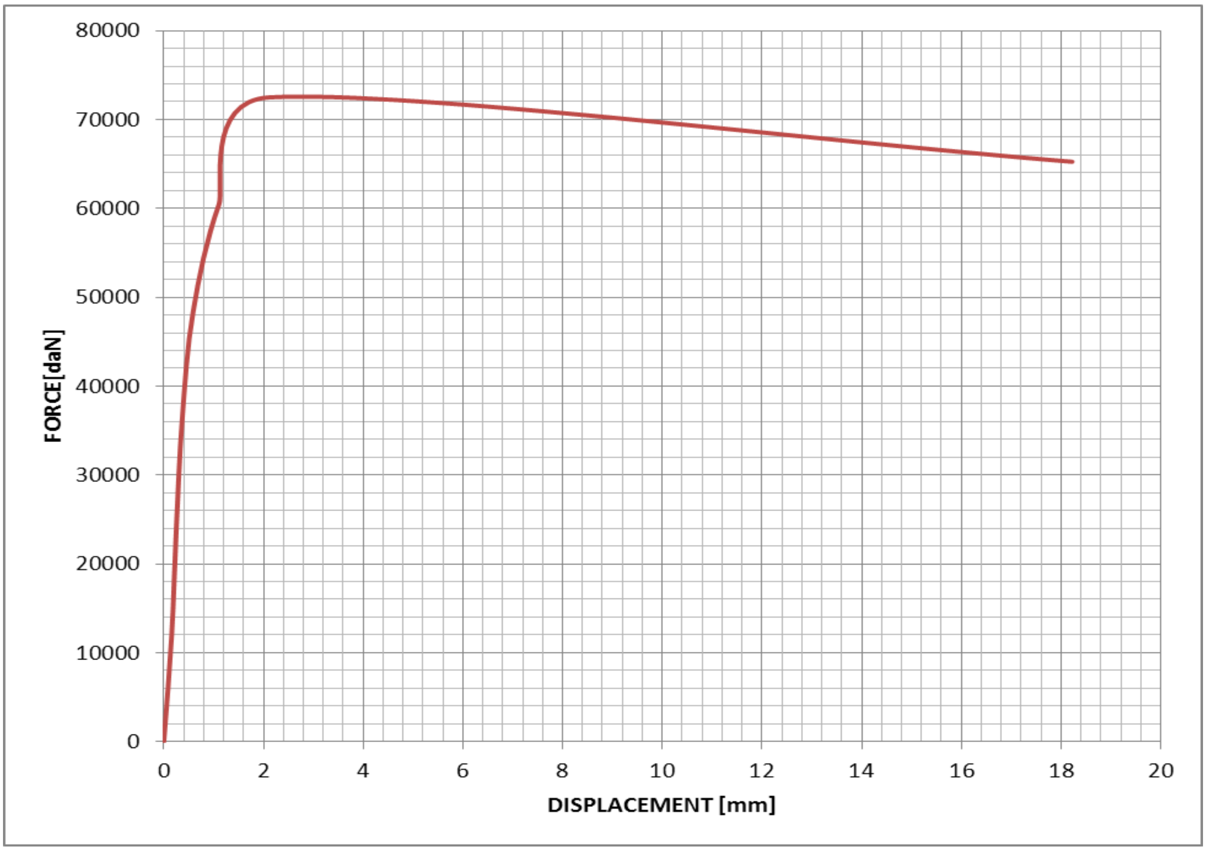



Figure 6.Finite element model results of the Force-Displacement diagram

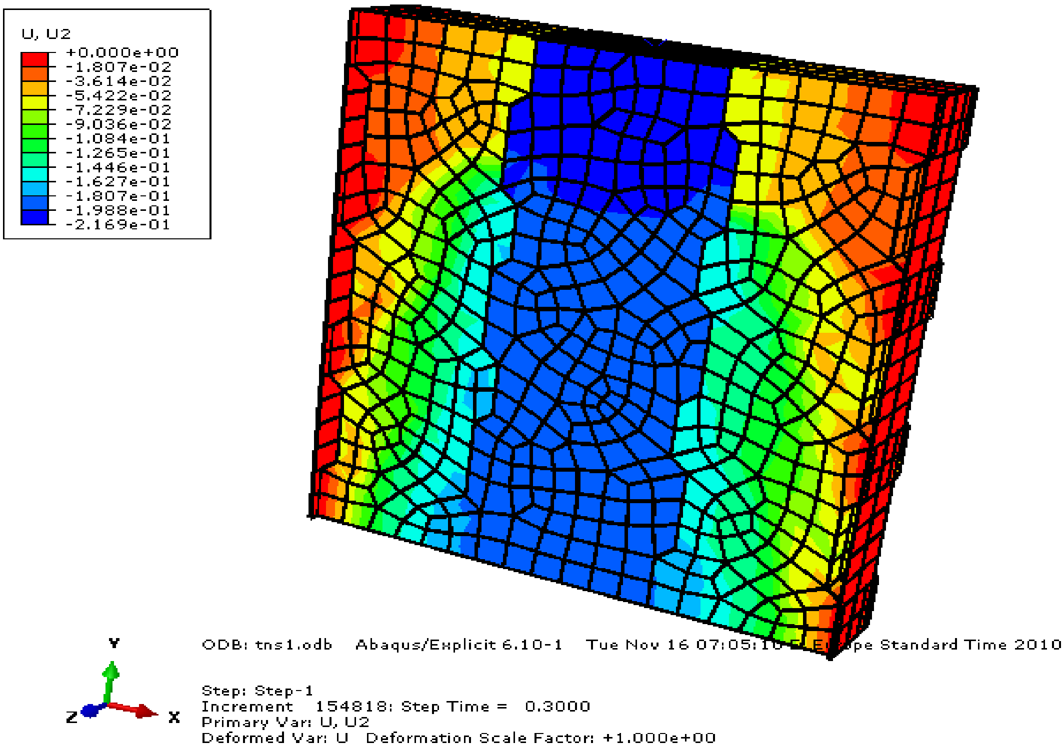



Figure 7.Finite element model results of the Deformed U2 variable

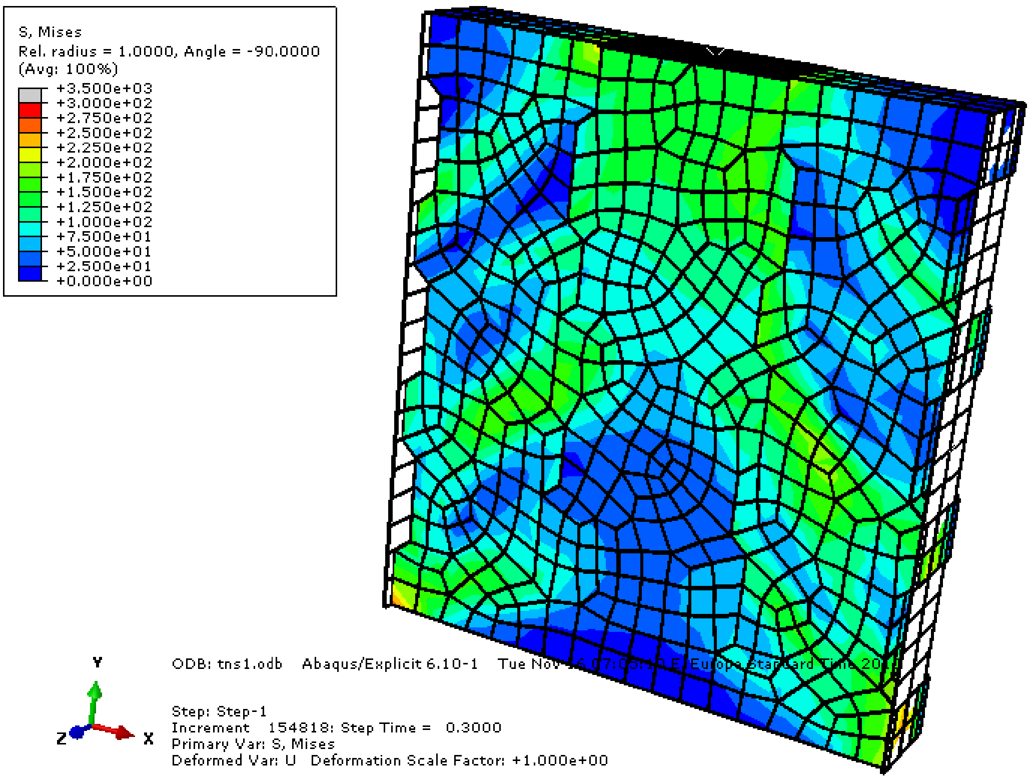



Figure 8.Finite element model results of the Von Mises stress

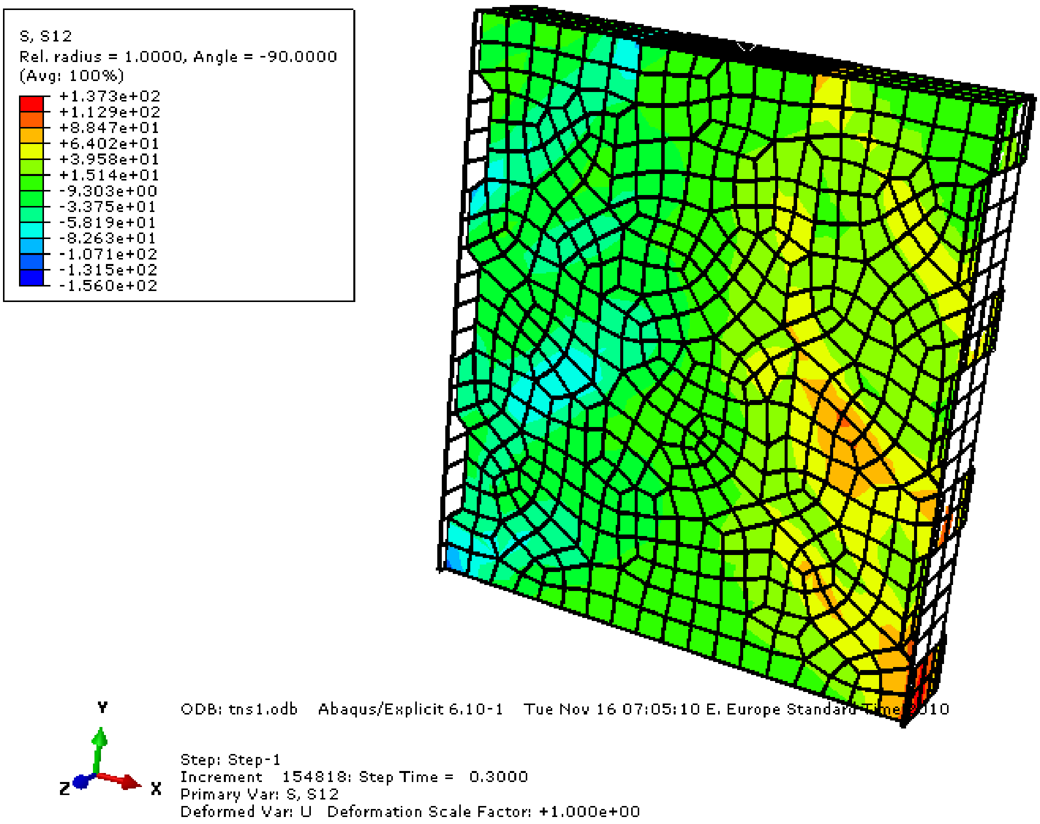



Figure 9.Finite element model results of the Von Mises stress,S12

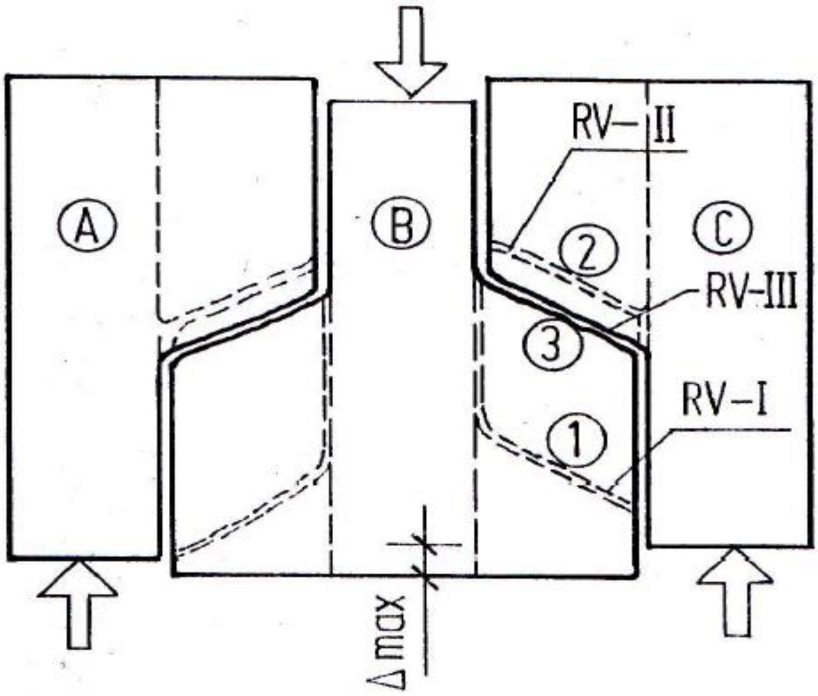



Figure 10.Failure mode experimental model(RV-II)

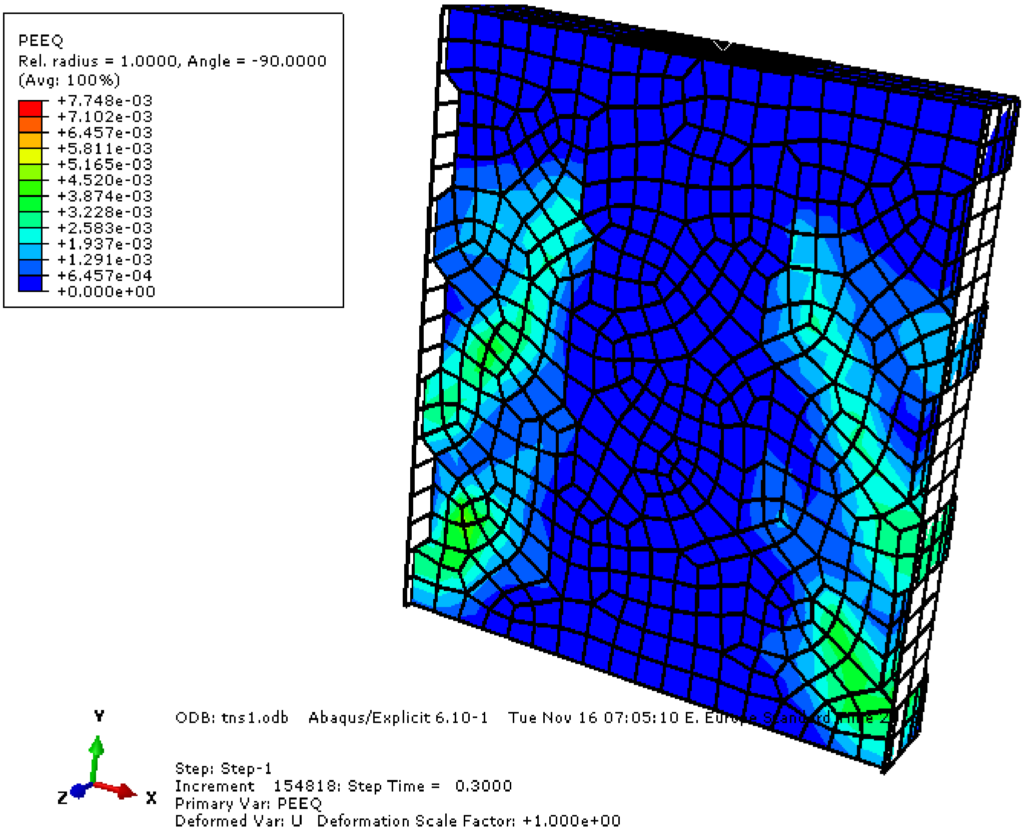



Figure 11.Finite element model results of the PEEQ,plastic equivalent strain

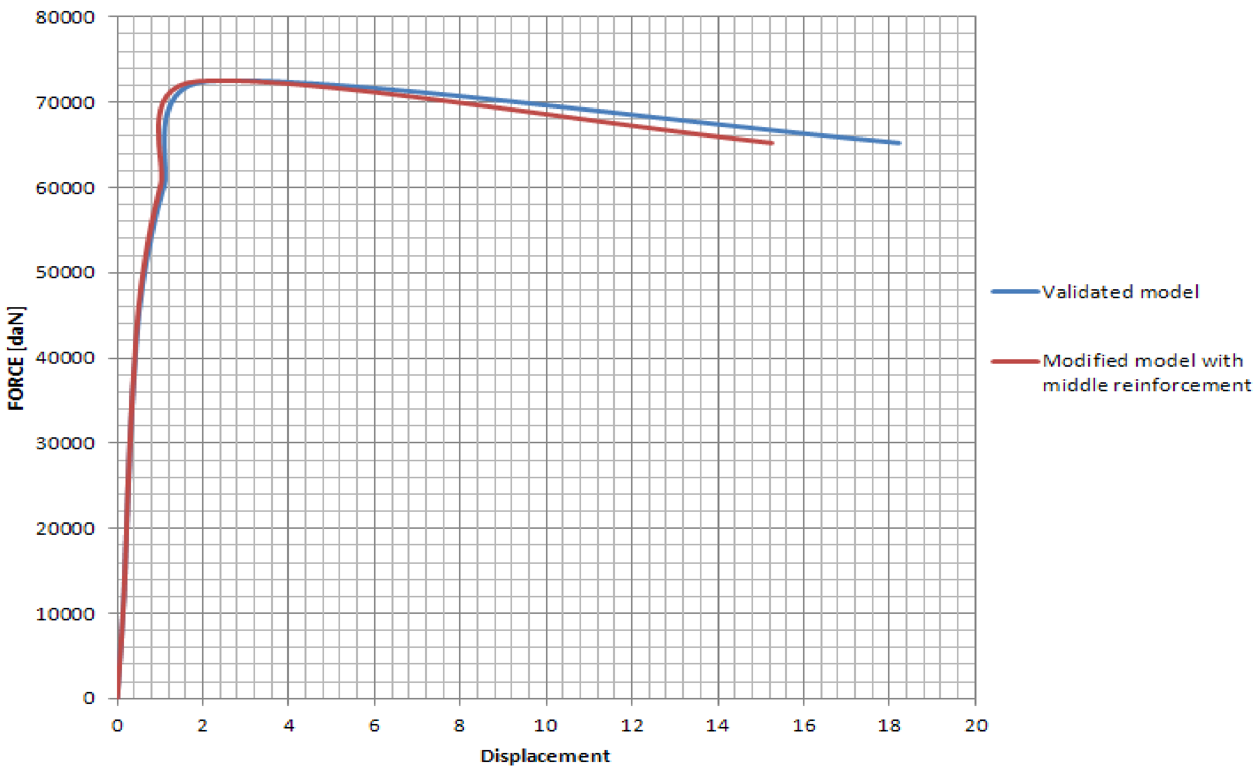



Figure 11.Comparison between the validated model and the modified model

## 4. Conclusions

Numerous models with different meshing tehniques and dimensions of the mesh,and concrete material curves were used until the validation of the model. In the available literature only a few papers make the subject of modelling precast large panels using finite elements so there is not much information regarding the behavior of precast large panels in the FEM field. At first the study was made using only solid elements to avoid any kind of mismatch between properties of the elements. In general a better behaviour could be captured using only solid elements because of the simulation of an overall effects such as friction,slip or debonding of the elements. The final model used for this study gave in general good results compared with the experimental results. After validating the finite element model,a new analysis was conducted using a new reinforcement at the middle of the central part to observe the overall behavior. As it could be seen it didn't influence the model,the strength of the concrete being much important,conclusion that was also made in the experimental study.

## 5. References

[1] Mirean Viorica „Contributii privind comportarea şi calculul diafragmelor din beton armat asamblate din panouri mari prefabricate" ,Timişoara,1998;

[2] Abaqus v.6.10., http://www.simulia.com;

[3] P101/78- „Instructiuni tehnice pentru proiectarea construcțiilor cu structura de rezistență din panouri mari.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Specific Behaviour of Lightweight Aggregate Concrete

Hans Udo KRASSER[1], Dan V. BOMPA[*2]

[1,2] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*Lightweight aggregate concrete is available for years but it has not been used for many structural applications due to its low compressive strength. Hardly, with prestress it can reach compressive strengths of 60 N/mm$^2$. It is well known that LWAC has a particular behaviour when compared to normal one. At failure point it crushes through the coarse part of the mixture making it an unpredictable material. The purpose of this paper is to identify the specific mechanical behaviour of lightweight aggregate concrete as presented in technical literature. A list of nine shear-walls with different constitutive laws are analysed using finite element method.*

## Rezumat

*Betonul cu agregate uşoare este disponibil pe piața construcțiilor de foarte mult timp, dar datorită rezistențelor la compresiune scăzute nu a fost folosit la multe aplicații structural. Cu greu, betonul uşor poate atinge rezistențe la compresiune de 60 N/mm$^2$, şi asta doar în prezența precomprimării. Lucrarea îşi propune să evidențieze o parte din caracteristicile specifice de comportare a betonului realizat din agregate uşoare. Lucrarea prezintă o analiză amănunțită prin metoda elementului finit asupra unei diafragme din beton uşor. Diafragma este analizată în nouă variante constitutive, şase cu agregate uşoare iar trei cu agregate normale.*

**Keywords:** Lightweight aggregate concrete, mechanical behaviour, finite element analysis

## 1. Introduction

Concrete is known to be the most widespread structural material due to its quality to shape up in various geometrical configurations. In some conditions, one might assume that normal weight concrete is inconvenient due to its density (2200-2400kg/m$^3$). Replacing partially or entirely the coarse part of normal weight aggregate concrete with lower weight aggregates produces lightweight aggregate concrete to a density between 1400-1850 kg/m$^3$ and can reach compressive resistance of 50N/mm$^2$. Lightweight aggregates used in structural lightweight concrete are fly ash, expanded clay or any other porous material.

Due to its lower density, LWAC reduces structural weight and dead load, that allows to use smaller section columns, smaller reinforcement ratio and foundations, with consequences on over-all structure cost. Structural lightweight concrete provides a more efficient strength-to-weight ratio in structural elements. Other important beneficial aspects are the thermal and durability characteristics. LWAC can be designed to reach high strength limits, similar to normal weight concrete.

---

[*] Corresponding author: Tel./ Fax.: +0264-401513
E-mail address: dan.bompa[at]bmt.utcluj.ro, bompadan[at]gmail.com.

## 2. Characteristic Behaviour of LWAC

Since the aggregates are porous and smaller in density, the shear force is directly proportional. The LWAC is weaker in tension and more susceptible to cracking. The cracking mainly produces through the aggregate and not adjacent reducing the aggregate interlock and thus the shear resistance, figure.1. In reinforced or prestressed concrete members it is beneficial to reduce the distance between the reinforcement bars to the minimum allowed to create superior shear transfer mechanisms due to the activation of tension stiffening phenomenon. During the last decades several experimental tests were carried out over beams and columns made of LWAC. It have been observed that the shear strength at tension diagonal cracking was influenced by the ratio between the shear span and the depth of the section and the shear strengths are approximative 15% lower than the predicted values.

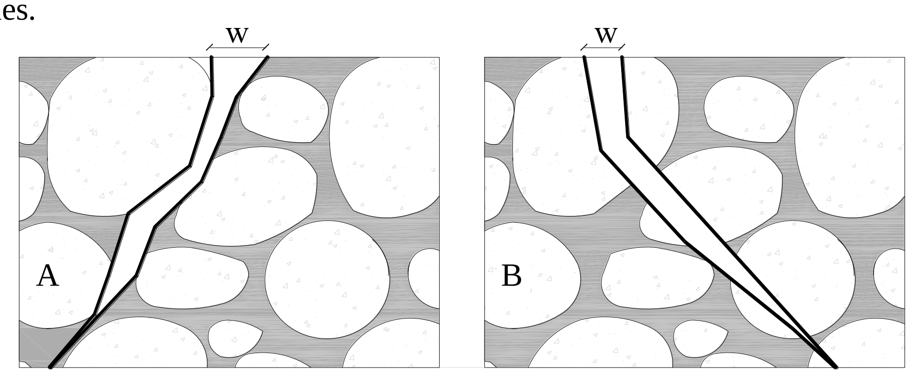



Figure 1. Shear failure - a. normal concrete b. lightweight aggregate concrete (through aggregate)

Another important observation was the difference in the behavior and strength between normal and lightweight concrete specimens related to the angle of the truss analogy compression strut. The authors (Hamadi and Regan, 1980) observed that the angle was less inclined for normal than for lightweight concrete. Moreover, for lightweight concrete specimens, stresses in transverse steel were higher that in normal weight specimens. It has also been observed that, besides the aggregate type influence, the behaviour of light concrete is similar with normal concrete in post-peak stage. Structural LWAC columns are allowed to be used in seismic regions being capable of developing significant inelastic deformation at limit states if design appropriate. Lightwieght columns behaviour have the tendency to be governed by shear failure and small deformations, in opposition to normal ones. That means that a reduction to the shear capacity must be applied. For example, ACI requires a reduction in the calculation of shear capacity with a ratio between 15% and 35% limiting high-strength concrete at 35.5 N/mm$^2$ in seismic design considering that the provided strength of the LWC is similar or superior comparable to the NWC. The ex-British Standard also inputs a limit of 40 N/mm$^2$ due to the reason that no shear boost in resistance occurs. Experimental tests carried out on high strength lightweight aggregate concrete exhibited higher deflection at the same load. The shear transfer mechanism showed a faster rate of degradation for LWAC at higher levels of ductility and the cracks, both from shear and flexure occurred at a earlier increment of load. In case of structural slabs, provisions for the use of structural lightweight aggregate concrete are included in several codes of practice, but limitations are applied as to the design values that may be used.

## 3. Numerical Analysis

Numerical analysis consists in a test upon the shearwall presented in figure 2, tested in six material constitutive configurations made of different types of lightweight aggregates: expanded clay (Liapor 4, Liapor 8), expanded shale (Berwilit) with an expected compressive strength of 15 and 25 N/mm$^2$ and three configurations with sand and gravel with an expected compressive strength of 15, 25 and 45 N/mm$^2$. The constitutive laws and ultimate strains for the aforementioned elements are to be found in table 1 and figure 3. The distance between horizontal bars is set to 15 cm and between the vertical bars to 20 cm. Stirrups have been position at the borders of the shearwall (fi10/20 at the top, fi15/10 cm on the vertical boundary). The reinforced bars are considered discrete in the homogenous solid concrete block. The concrete shear wall is meshed with tethraedrons of 100 mm leg C3D4.

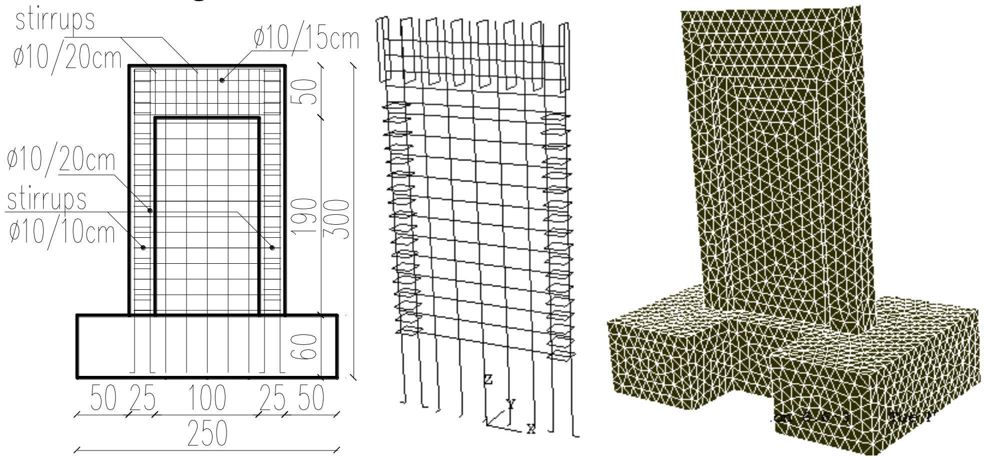



Fig 2. Tested shearwall, a. Geometrical configuration, b.FEM rebar, c. FEM Mesh

Both, reinforcement and concrete material characteristics are introduced as being elasto-plastic with 10 characteristic points. The base of the shear wall is considered fixed. Every specimen was loaded with a concentrated force of 700 kN acting on the top right corner, assuming a distribution similar with one produced by an earthquake. The results are presented in table 1 in matter of maximum stresses and maximum global displacements. It can be seen that the highest displacement occurred to the S1 specimen, close to S5. When compared to S7, made of sand and gravel, with an similar expected compressive strain of 15N/mm$^2$, S1 and S5 show a displacement three times larger. The comparison between the the specimens with expected strength of 25N/mm2 show a 25%-87% displacement above S8. The comparison between S4 and S9 shows that the LWAC made of Liapor 8 is 37% weaker due to its larger displacements.

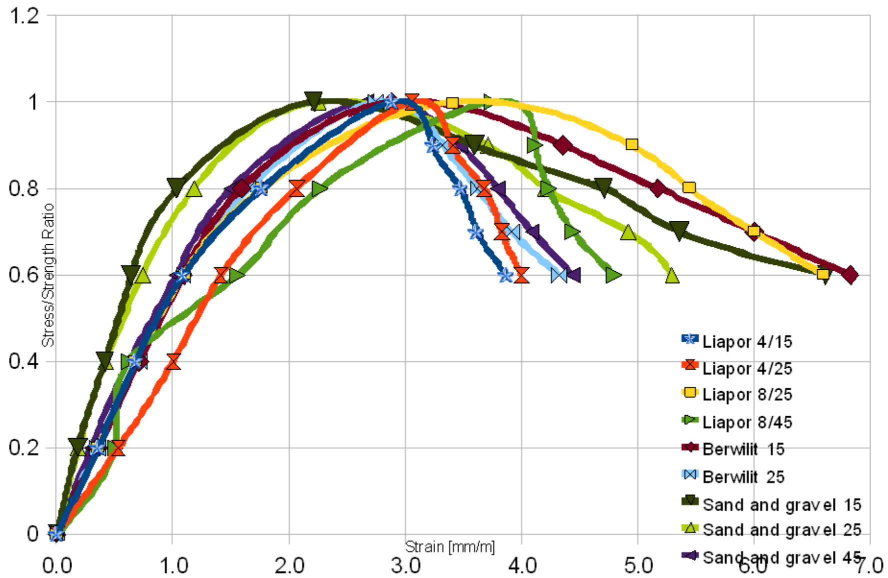



Fig. 3 Constitutive laws for LWAC

Table 1: Material characteristics and results

| *Specimen* | *Aggregate* | *Concrete density ($kg/m^3$)* | *28day strength ($N/mm^2$)* | *Modulus of deformation ($N/mm^2$)* | *Ultimate strain [mm/m]* | *Maximum Stress [$N/mm^2$]* | *Maximum displacement [mm]* |
|---|---|---|---|---|---|---|---|
| S1 | Liapor 4/15 | 1200 | 14 | 6221.51 | 2.71 | 3.377E+01 | 2.629E+00 |
| S2 | Liapor 4/25 | 1300 | 20 | 8384.75 | 3.04 | 2.004E+01 | 1.665E+00 |
| S3 | Liapor 8/25 | 1500 | 19 | 10129.17 | 3.41 | 1.384E+01 | 1.111E+00 |
| S4 | Liapor 8/45 | 1630 | 35 | 15573.11 | 3.64 | 1.147E+01 | 9.160E-01 |
| S5 | Berwilit 15 | 1350 | 11 | 6580.47 | 2.88 | 2.842E+01 | 2.484E+00 |
| S6 | Berwilit 25 | 1400 | 19 | 9133.32 | 2.74 | 1.854E+01 | 1.573E+00 |
| S7 | Sand and gravel 15 | 2100 | 10 | 12172.76 | 2.21 | 1.096E+01 | 8.385E-01 |
| S8 | Sand and gravel 25 | 2150 | 17 | 16441.53 | 2.25 | 1.086E+01 | 8.895E-01 |
| S9 | Sand and gravel 45 | 2240 | 35 | 25088 | 2.66 | 7.307E+00 | 5.751E-01 |

## 4. Conclusions

The paper presents an incursion upon the mechanical behaviour of lightweight aggregate concrete. A comparison between several constitutive configurations is made on nine different specimens of a shear wall subjected to a 700 kN horizontal load. The weakest element showed to be S1, made of Liapor 4 and the strongest S4, Liapor 8. It was shown that the displacement capacity and strength is depending on the density of the aggregate and also, the expected compressive strength. All specimens showed a higher displacement, and a weaker resistance to the horizontal load. When considering design specifications, it must be concluded that the reduction factors attached to the formulae used for normal concrete are perfectly justified. The numerical test showed a displacement for LWAC up to three times larger in comparison to NWAC. In some situations the aforementioned reduction factors from chapter two might be increased. Though, sustained research in this field must be submitted both on real specimens tested in laboratory and specimens tested via finite element method. The use of LWAC for structural elements might be used in good conditions when proper design is applied.

## 5. References


[1] H.D. Basche et al. - Analysis of shear capacity of lightweight concrete beams, *Engineering Fracture Mechanics 74 (2007) 179–193*

[2] J. L. Clarke – Shear strength of lightweight aggregate concrete, *Cement and Concrete Association, 1987*

[3] S.E. Hendrix - Shear Performance of High-Strength Lightweight Concrete Columns under Seismic Loads, *Master of Science Thesis, North Carolina State University*

[4] P.J. Robalino - Shear Performance of Reinforced Lightweight Concrete Square Columns in Seismic Regions, *Master of Science Thesis, North Carolina State University*

[5] N. Haque, H. Al-Khaiat - Strength and durability of lightweight concrete in hot marine exposure conditions, *Materials and Structures, vol 32. 1999*

[6] *** CEB/FIP Bulletin 121 – Lightweight aggregate concrete, 1977

[7] *** FIB Bulletin 4, August 1999:

[8] *** FIB Bulletin 8, May 2000:

[9] W. Raongjant - Finite element analysis on lightweight reinforced concrete shear walls with different web reinforcement**,** *The Sixth PSU Engineering Conference***,** *May 2008*

[10] Paraschiva Mâzgan, T. Oneț *:* Beton usor. *Editura Universității Transilvania din Brasov, 2005.*

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Consolidating Brick Masonry Structural Walls Using Ferrocement Coating

Gavrilă MUNTEAN [*1], Radu MUNTEAN[2], Traian ONET[3]

[1,2] Transilvania University of Brasov, 29 Eroilor Str., 500036, Brasov, Romania
[3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*This paper aims at giving a succinct presentation of some aspects regarding the advantages of the brick masonry walls, which makes their consolidation necessary and realistic in case of damage. Secondly, various solutions are presented for strengthening, with emphasis on those made of micro-concrete reinforced with thin wire mesh, also called "ferrocement", a solution verified by practice in the INCERC Cluj-Napoca laboratory.*

**Keywords:** Ferrocement coating, Structural walls, Brick masonry, Strengthening.

## 1. Introduction

The most widely used structural walls have been and will probably be those of brick masonry, as they present a number of advantages, among which we would like to mention:

- They are lasting: well built and appropriately preserved masonry can last for a few hundred years;
- They have a good thermic insulation: they have reduced thermic conductivity coefficients $\lambda_0$=0,80W/mK;
- Ensure good sound proofing;
- Ensure a good respiration of the rooms;
- Have a good fire resistance;

For different reasons, brick masonry undergoes deteriorations and degradations, among which cracks and fissures are the most frequent. Quite often, a crack appeared in the plaster hides a deep crack in the brick masonry.

Regarded from this perspective, the damage of the masonry can be divided into two categories:

- marked cracks, which indicate massive masonry dislocations;
- small but numerous cracks or fissures, which show a degradation of the masonry.

## 2. Causes for masonry deterioration

Generally, the causes which produce the deterioration or degradation of masonry are:

- the ageing of the material as time goes by (the bricks and mortar);
- the lack of maintenance of the building and the condensation phenomenon;

[*] Corresponding author: Tel./ Fax.: +40 268 410525

- the degradation of the foundation ground as a result of the infiltration of pluvial water, of the wastes from supply or sewerage installations, of the increase in the level of the water layer or the changes in the routes triggered by new buildings;
- the seismic action;
- other extraordinary actions such as explosions, fires etc.

## 3. The conception and the general consolidation principles

Generally, the consolidation conception of masonry buildings has to consider the following aspects:

- To eliminate the causes which produce material degradation;
- To avoid changes of the structural system;
- To improve the transmission of loads to foundations;
- To bind the adjacent vertical elements;
- To put together the vertical structural elements.

The consolidation of masonry structures can be accomplished through:

- rebuilding dislocated masonry;
- partial reinforcement with concrete;
- injecting and flatting cracks and fissures; mat
- stitching fissures using steel cramps;
- coating the walls with mortar, concrete or composite materials;
- bordering the gaps;
- binding corner areas;
- introducing ties and/ or metal fishplates ;
- placing horizontal and vertical elements made of reinforced concrete.

The consolidation concept of the structural system may require the combination of the afore-mentioned procedures, according to the causes that triggered the damage, the yielding mechanism and the condition of the building, in particular.

When accomplishing any rehabilitation work for brick masonry structures, a paramount stage is represented by the preparation of the masonry, which consists of the following:

- removing the existing plaster;
- deepening the empty spaces for 15-20mm;
- removing the non-adherent material by scrubbing using a wire brush until the pores of the masonry stone are open ;
- using compressed air to blow the cleaned surfaces with a view to removing the dust.

## 4. Coating consolidation

In what follows, we will refer to the solution of consolidating by means of coating the walls. This solution is indicated for old structures which are strongly deteriorated and which have a significantly diminished portant capacity of the structural walls.

In practice, there are different solutions for consolidating masonry walls by means of coating. Thus, there are coatings on one or both sides of the structural walls, using cement or concrete-based mortar, the reinforcement being accomplished by means of welded nets made of 4-6mm thick wire having meshes of 100-200mm, as well as using independent mesh-bound bars 6-8mm thick, polymeric grates or meshes made of thin wire, 1-2mm thick, with rectangular or hexagonal spaces between bars, 20-30mm thick, placed on one or several layers in the thickness of the coating layer.

Each of these solutions presents a series of advantages and disadvantages, as well as numerous peculiarities.

While research has been conducted and concrete solutions have been offered in the case of consolidating walls using thick welded mesh or using polymetric grates [8],[9],[10], little is known about the consolidation using micro-concrete reinforced with thin mesh, called ferrocement [1],[2],[3],[4],[5]. That is why in what follows we aim to present some aspects which should give emphasis to this method, based on the trials which were accomplished on experimental models at INCERC Cluj-Napoca.

The masonry walls used by INCERC Cluj Napoca for obtaining information with a view to improving the calculation methods for masonry structures under seismic actions [11] have been used in our research to check the efficiency of the consolidation by coating with ferrocement and to establish connections between the theoretical and the experimental calculation model.

The masonry walls have been built in two ways: from 240x115x63mm filled ceramic bricks and GVP 290x240x138mm ceramic bricks with vertical voids. The masonry mortar was M50. In both cases, an outline framing of the masonry has been accomplished with 24x10cm pillars and 24x20cm belts made of concrete class Bc10 (B150), according to the standard P2-85. The vertical reinforcement with pillars was 4ф8 PC52, the superior belt being also embedded in the foundation. The transversal reinforcement of the pillars was accomplished by means of black wire bars F5mm, having a transversal reinforcement percentage of 0.157%. The final dimensions of the masonry panels were: h=190cm (including the 20cm belt), l=180cm and the thickness of 24cm.

The wall models have been submitted to cyclical lateral trials in the presence of a constant vertical load which produced a 0.4Mpa axial effort at their foot (including its own weight).

As a result of the trials performed between 2001 and 2002, it was found that the panels made of masonry confined with pillars surrendered under the sliding shear force through on the route of inclined fissures and the shearing of the pillars.

In order to consolidate the respective walls, which were initially tried until they broke down, the following technology was used:

- the affected surfaces were polished with a chisel in order to remove the deteriorated material (exfoliated, broken, etc.);
- the surface was cleaned using a water jet and a wire brush to remove the small pieces of material and the dust;
- on the clean and rugged surface, nails were hammered in the mortar empty spaces. The distance between the nails was 20….25cm, and the remaining free length of 10….15mm. at the same time, plastic disks were fitted, playing as distance pieces;
- a layer of mixture was applied on the surface, made of cement milk;
- two layers of zinc-coated steel wire mesh were positioned, with a diameter of 1mm and the distance between the bars of 10x10mm. In order to fix the meshes, the previously positioned nails were used;
- the mortar was applied by an average 3,5 cm injection with concrete, the covering layer being at least of 0,5cm; the mortar was prepared according to the following recipe: cement Pa40 - 500kg/mc, sand 0-3mm 1700 kg/mc, water 250kg/mc. From the trials on the test pieces made of this mortar, the following values resulted regarding the resistance to compression: 7,5N/mm$^2$ after 7 days and 39N/mm$^2$ after 28 days. The values for the stretching resistance from bending to were 1,12N/mm$^2$ after 7 days and 7,75N/mm$^2$ after 28 days.
- the sides of the element were finished off through the application of a polishing plaster coat and of a white painting which makes fissures more visible.

The trial of the consolidated elements was accomplished in the same conditions as the unconsolidated ones, respectively by applying alternating lateral forces by means of hydraulic jack in the presence of a vertical force which created an effort of 0,4N/mm$^2$ at the foot of the masonry panel.

The two layers of concrete-injected mortar together with the broken masonry formed such a rigid unitary whole that, under lateral forces equal with those which made unconsolidated panels to break down, respectively 224KN, the panel rotated as a rigid solid in rapport with the contact point between the wall and the foundation, without fissures to be produced on the sides of the panel.

In order to mobilize the concrete-injected mortar, it was necessary to block the rotation by introducing an additional tie bar placed at the edge of the consolidated masonry panel.

In this new situation, the alternating forces were applied in an increasing pace, starting with the value of 40KN, in stages of 40kN. The breaking down took place when the value of the lateral force was around 230KN, at the VI cycle, when the mortar cover detached from the masonry.

Figures 1 and 2 show the diagrams F- Δ (force - movement) by load cycles for the unconsolidated masonry panel, respectively for the same panel, which was meanwhile coat consolidated.

Figures 3 and 4 show the fissures for the unconsolidated panel, respectively for the same consolidated panel during the breaking stage.

Figures 5 and 6 show the trial scheme and the way the element under trial was equipped with measuring apparata.

In fig.7 (photo), we presented the panel which underwent the trial using the above mentioned equipment.

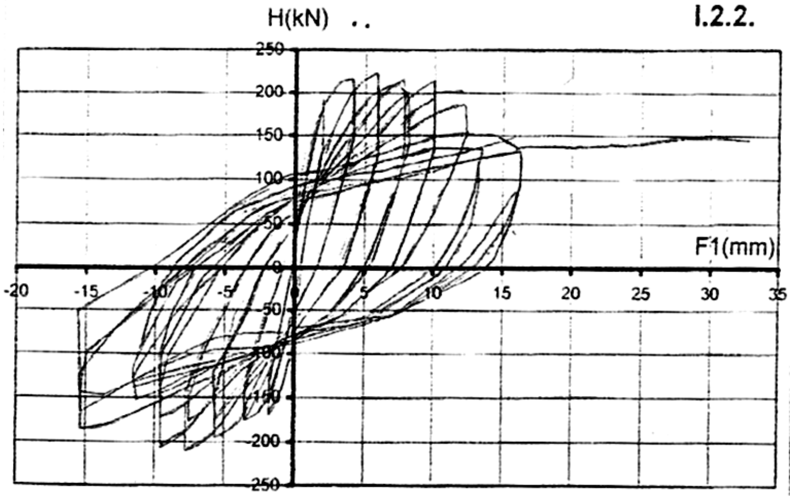



**Fig. 1**. Diagram F – Δ for the unconsolidated masonry element

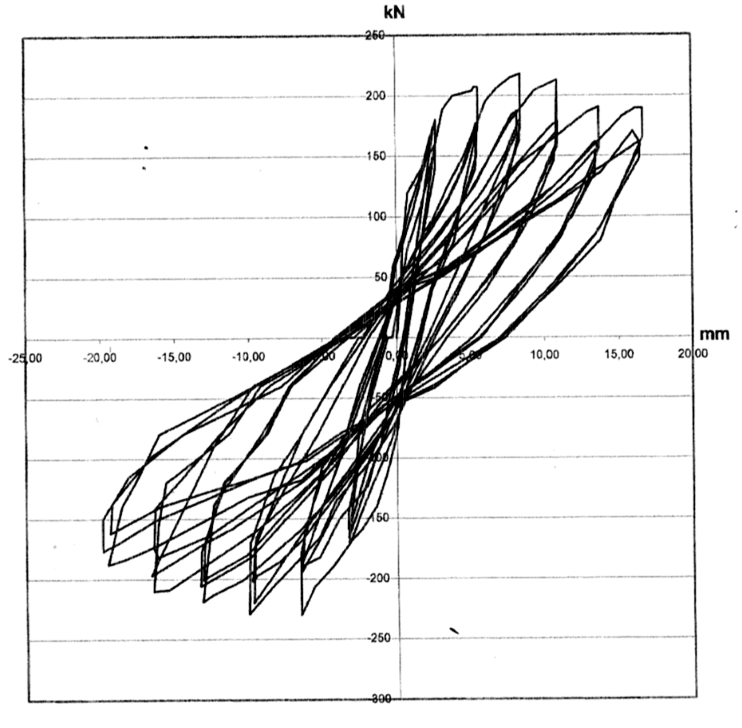



**Fig. 2**. Diagram F - Δ for the consolidated masonry element

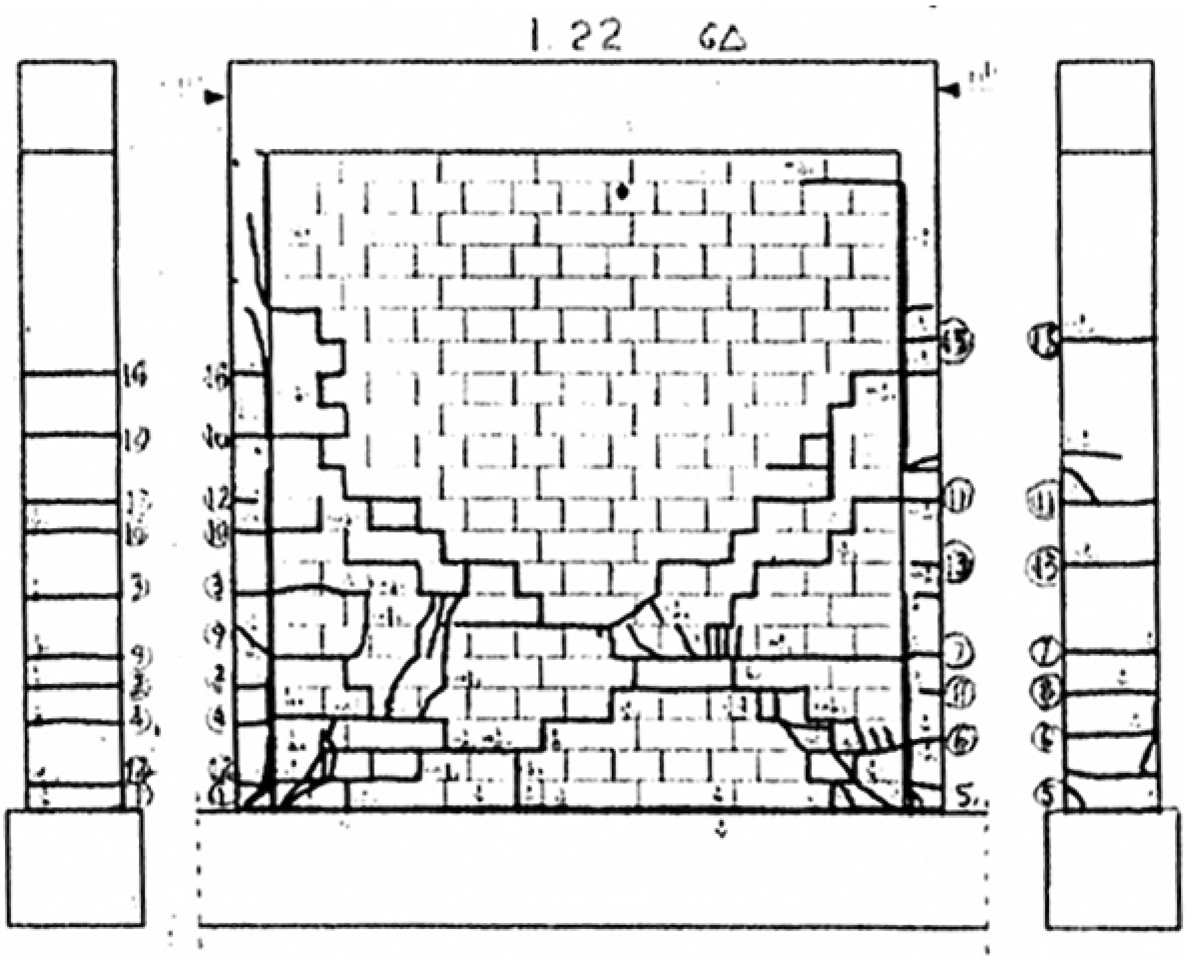


**Fig. 3**. The fissures for the unconsolidated masonry element

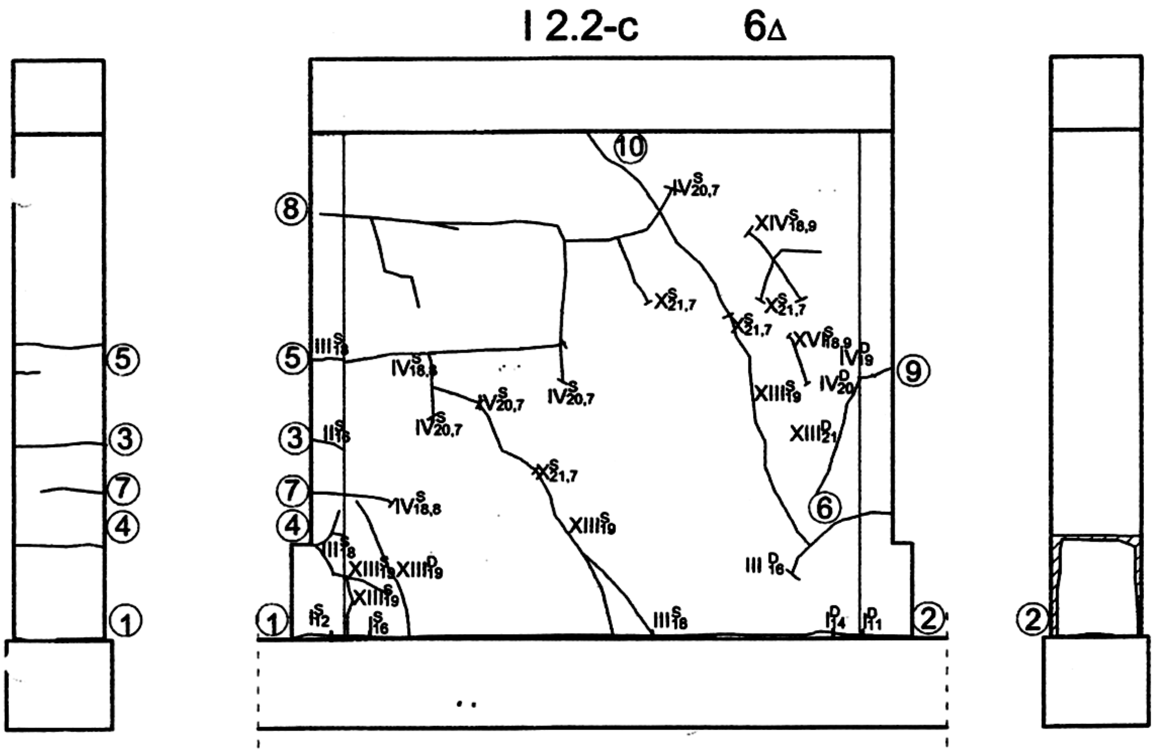



**Fig. 4**. The fissures for the consolidated masonry element

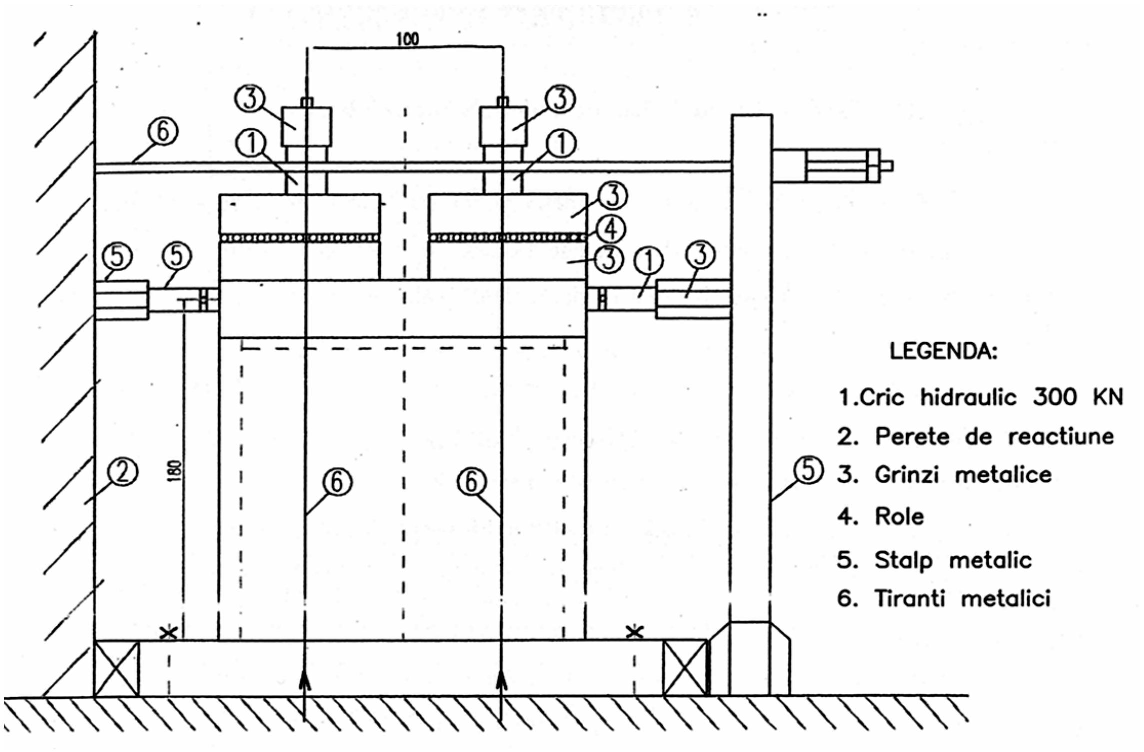



**Fig. 5**. Trial scheme

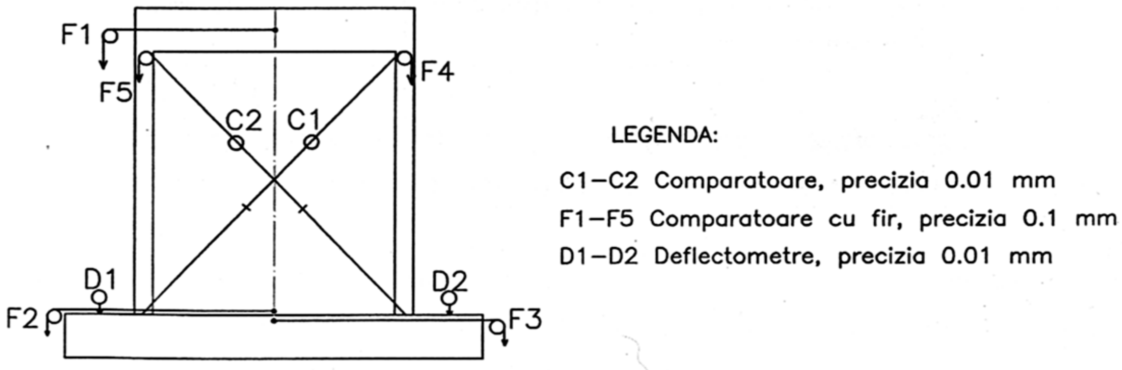



**Fig. 6**. Scheme of measuring equipment


**Fig. 7**. Equipping of the trial stall for the consolidated masonry element

## 5. Advantages of the solution

The quality of ferrocement to have a very good resistance to fissuring gives it a great advantage over the use of reinforced concrete. The increased resistance to fissuring, combined with the easiness of applying it, as well as its relatively small weight and the reduced cost, make ferrocement an ideal system for rehabilitating structures.

The main objective envisaged for the consolidation and rehabilitation of masonry structures is represented by the recovery of their portant capacity with as reasonable costs as possible. From the numerous reparation and rehabilitation programs for structures which have used ferrocement, H. Ahmed and L R. Austriaco [5] underline the following aspects:

- good behavior as regards fissuring;
- capacity to improve certain mechanical peculiarities of consolidated structures;
- the accomplished consolidations allow changes and subsequent reparations;
- the relatively reduced weight resulted from the consolidation systems does not require changes in the structure support system;
- facility to oppose to temperature changes;
- facility to allow good water proofing, without special treatments;
- easiness to purchase necessary materials;
- does not require special technological equipment;
- flexibility to subsequent changes;
- possibility not to alter the architectural concept of the structure and, implicitly, of the building as a result of the consolidation.

## 6. Conclusions

As a result of the afore mentioned trials, the following conclusions can be drawn.

- by applying coat consolidation on 2 sides, using cement mortar and thick wire mesh, the element regained the initial portant capacity (slightly enhanced);
- from the analysis of the diagrams F- Δ, it was found that there was an increase in the rigidity of the coat-consolidated element, despite the fact that it had practically been broken in the initial trial (when it was unconsolidated).
- from the analysis of the fissures, it was found that, while unconsolidated masonry panels exhibit great fissures, between 10 and 30mm, in the case of consolidated panels these fissures are much smaller, respectively between 0.05 and 0,5mm.

## 7. References

[1] A.C.I. – 549R – 93: State of the Art Report on Ferrocement

[2] A.C.I. – 549 –1R-93: Guide for the Design, Construction and Repair of Ferrocement

[3] B.K.Paul, R.P. Pama: Ferrocement – ITIC Publication, Bankok, Thailand 1978

[4] Oneț T., Vescan V., Szigeti L.: Realizări şi perspective privind utilizarea ferocimentului în construcții. Revista de construcții 2-3/1991

[5] H.I.Ahmed, L.R.Austriaco: State of the Art Report on Rehabilitation and Restrething of Structures Using Ferrocement – Journal of Ferrocement, vol.21, No.3/1991

[6] Mihai Budescu, ş.a.: Reabilitarea construcțiilor. Editura Vesper-2001

[7] Tologea S.: Probleme privind patologia şi terapeutica construcțiilor. Editura Tehnică-1975

[8] Ramiro A. Sofronie: Manual RichterGard. Total Publishing-Bucureşti 2004

[9] Berar T. .: Teză de doctorat-2004, Universitatea „Politehnica” Timişoara

[10] Secula S.: Teză de doctorat-2003, Universitatea „Politehnica” Timişoara

[11] INCERC Filiala Cluj-Napoca:Imbunatatirea metodelor de calcul al structurilor din zidărie la acțiuni seismice. Faza 2/2001. Incercare elemente.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Checking the Concordance between the Theoretical and the Experimental Model for Brick Masonry Walls Consolidated With Ferrocement Coating

Gavrilă MUNTEAN [*1], Radu MUNTEAN[2], Traian ONET[3]

[1,2] Transilvania University of Brasov, 29 Eroilor Str., 500036, Brasov, Romania
[3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*The paper demonstrates the concordance between the theoretical calculations and the experimental results obtained on real scale bricks masonry walls reinforced by ferrocement, in the INCERC Cluj-Napoca laboratory.*

**Keywords:** Ferrocement coating, Masonry walls, Strengthening, Experimental model.

## 1. Introduction

Although ferrocement as a material has been known for quite a long time, being used in a wide variety of buildings, from boats to sheeting lost casing and from tanks different liquids to architectural masterpieces, such as the Sydney Opera House, its use for the rehabilitation of buildings damaged from different causes has been insufficiently researched and applied.

## 2. Description of the consolidation solution by means of ferrocement coating

In what follows, reference will be made to the solution of coating consolidation using ferrocement. This solution is indicated for old structures which are strongly deteriorated and which have a significantly diminished portant capacity of the structural walls. As a result of the afore mentioned facts, the authors of this study aim at presenting some aspects regarding the calculation of the elements which were consolidated using ferrocement and then submitted to stresses on the trail stall from the platform INCERC Cluj-Napoca. The masonry walls (4 panels) which underwent trials between 2001 and 2002 at INCERC Cluj Napoca in order to obtain information for improving the calculation methods for masonry structures under seismic actions were used by the authors in 2005 to check the efficiency of the coating consolidation with ferrocement and to establish connections between the theoretical and the experimental calculation model.

The masonry walls have been built from 240x115x63mm filled ceramic bricks. The masonry mortar was M50. The outline framing of the masonry has been accomplished with 24x10cm pillars and 24x20cm belts made of concrete class Bc10 (B150), according to the standard P2-85. The vertical reinforcement with pillars was 4ф8 PC52, the superior belt being also embedded in the foundation. The transversal reinforcement of the pillars was accomplished by means of black wire bars F5mm, having a transversal reinforcement percentage of 0.157%. The final dimensions of the

[*] Corresponding author: Tel./ Fax.: +40 268 410525

masonry panels were: h=190cm (including the 20cm belt), l=180cm and the thickness of 24cm.

The wall models have been submitted to cyclical lateral trials in the presence of a constant vertical load which produced a 0,4 $N/mm^2$ axial stress at their foot (including its own weight).

As a result of the trials performed between 2001 and 2002, it was found that the panels made of masonry confined with pillars surrendered under the shear sliding force on the route of inclined fissures and the shearing of the pillars.

In order to consolidate the respective walls, which were initially tried until they broke down, the following technology was used:the affected surfaces were polished with a chisel in order to remove the deteriorated material (exfoliated, broken, etc.);

- the continuity of the reinforcement was restored using pillars with welded fishplates;
- the surface was cleaned using a water jet and a wire brush to remove the small pieces of material and the dust;
- on the clean and rugged surface, nails were hammered in the mortar empty spaces. The distance between the nails was 20….25cm, and the remaining free length of 10….15mm. at the same time, plastic disks were fitted, playing as distance pieces;
- a layer of mixture was applied on the surface, made of cement milk;
- two layers of zinc-coated steel wire mesh were positioned, with a diameter of 1mm and the distance between the bars of 10x10mm. In order to fix the meshes, the previously positioned nails were used;
- the mortar was applied by an average 3,5 cm injection with concrete, the covering layer being at least of 0,5cm; the mortar was prepared according to the following recipe: cement Pa40 - 500kg/mc, sand 0-3mm 1700 kg/mc, water 250kg/mc. From the trials on the test pieces made of this mortar, the following values resulted regarding the resistance to compression: 7,5$N/mm^2$ after 7 days and 39$N/mm^2$ after 28 days. The values for the stretching resistance from bending were 1,12$N/mm^2$ after 7 days and 7,75$N/mm^2$ after 28 days.

The sides of the element were finished off through the application of a polishing plaster coat and of a white painting which makes fissures more visible;

The trial of the consolidated elements was accomplished in the same conditions as the unconsolidated ones, respectively by applying alternating lateral forces by means of hydraulic jack in the presence of a vertical force which created an stress of 0,4$N/mm^2$ at the foot of the masonry panel.

The two layers of concrete-injected mortar together with the broken masonry formed such a rigid unitary whole that, under lateral forces equal with those which made unconsolidated panels to break down, respectively 224KN, the panel rotated as a rigid solid in rapport with the contact point between the wall and the foundation, without fissures to be produced on the sides of the panel (it tended to fall down).

In order to mobilize the concrete-injected mortar, it was necessary to block the rotation by introducing two additional steel tie bars of Ø = 24 mm each, with leak resistance of 331,8 $N/mm^2$; they were placed at the edge of the consolidated masonry panel.

In this new situation, the alternating forces were applied in an increasing pace, , in stages of 40kN. The breaking down took place when the value of the lateral force was around 460 KN, at the VI cycle, when the foundation broke down.

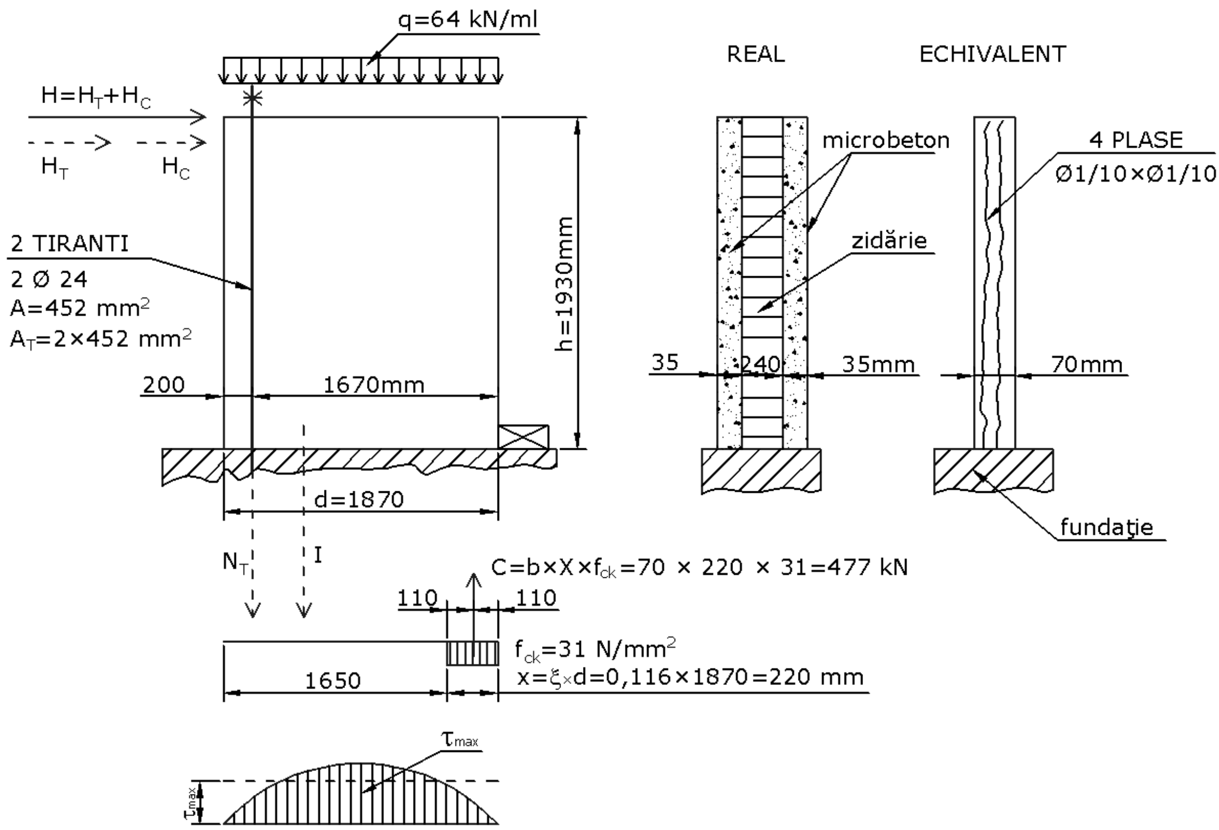
## 3. The calculus

The calculations were accomplished using the theoretical model which was put into practice, both for ultimate limit states (SLU) for determining normal stress from eccentric compression and tangent stress from shearing, and for limit states of normal exploitation (SLEN), respectively fissuring and deformations.

Taking into account the specific way of yielding, both for consolidated and unconsolidated elements, respectively through the shearing of the masonry and of the pillars and then, the appearance of diagonal fissures in the coating, in what follows we will consider the calculation of the stress in inclined sections resulted from the shear forces applied to the research element.

The calculations were accomplished according to two methods, the one present in the Code of ferrocement elements design drawn up by the Technical University of Cluj Napoca in 1999 and the stipulations EUROCODE EC2; in what follows, we will give them due consideration.

Calculation scheme




## 4. Calculation according to the model from „Code of ferrocement design” – drawn up by the Technical University of Cluj Napoca

Wire mesh is used for reinforcement, Ø = 1 mm thin, having a distance of 10 mm, with the following peculiarities:

- number of meshes: n = 4 meshes
- weight of the mesh: $G_a = 1{,}138$ kg/m$^2$
- area of the mesh: $A = 1{,}93 \times 1{,}87 = 3{,}61$ m$^2$
- steel density: $\rho_0 = 7850$ kg/m$^3$

The volumetric reinforcement percentage is found: $V_f$

$$V_f = \frac{V_p}{V_s}\text{, where:}$$

$$V_p = \frac{n \times G_a \times A}{\rho_0} = \frac{4 \times 1{,}138 \times 3{,}61}{7850} = 0{,}0021 \text{ m}^3 \text{ – volume of the meshes}$$

$$V_s = h \times A = 2 \times 0{,}035 \times 3{,}61 = 0{,}25 \text{ m}^3 \text{ – volume of micro-concrete}$$

$V_f = \dfrac{0{,}0021}{0{,}25} = 0{,}00837$; it results:

$V_{fx} = 0{,}5 \times V_f = 0{,}0042$ – on one direction.

The unitary stress capable of shearing at fissuring is determined with the formula:

$\tau_{cr} = f_{bt} + 450 \times V_{fx}$ [N/mm$^2$]

$f_{bt} = m_{bt} \times f^*_{ct}$

$f^*_{ct} = \dfrac{f_{tk}}{\gamma_{bt}}$ ; $m_{bt} = 1{,}0$ ; $\gamma_{bt} = 1{,}5$

$f_{tk} = 0{,}22 \times (f_{ck})^{\frac{2}{3}}$ [N/mm$^2$]

$f_{ck} = (0{,}87 - 0{,}002 \times f_{bk}) \times f_{bk}$ [N/mm$^2$]

$f_{bk} = 39$ N/mm$^2$ –experimentally obtained through submitting to trial the test pieces drawn from the micro-concrete used

$f_{ck} = (0{,}87 - 0{,}002 \times 39) \times 39 = 31$ [N/mm$^2$]

$f_{tk} = 0{,}22 \times (31)^{\frac{2}{3}} = 2{,}17$ [N/mm$^2$]

$f^*_{ct} = \dfrac{f_{tk}}{1{,}5} = \dfrac{2{,}17}{1{,}5} = 1{,}45$ [N/mm2]

$f_{bt} = m_{bt} \times f^*_{ct} = 1 \times 1{,}45 = 1{,}45$ [N/mm$^2$]

$\tau_{cr} = f_{bt} + 450 \times V_{fx} = 1{,}45 + 450 \times 0{,}0042 = 3{,}34$ [N/mm$^2$]

The (average) experimental unitary shearing stress

$\tau_{max} = \dfrac{3}{2} \times \dfrac{H_F}{b \times d} = \dfrac{3}{2} \times \dfrac{440 \times 10^3}{70 \times 1870} = 5{,}04$ [N/mm$^2$], where:

$H_F = 440$ kN – force under which the fissure appeared

$\tau_{med} = \dfrac{2}{3} \times \tau_{max} = \dfrac{2}{3} \times 5{,}04 = 3{,}36$ [N/mm$^2$]

From the afore mentioned data it results that the unitary tangent calculation stress $\tau_{cr} = 3{,}34$ N/mm$^2$ is approximately equal with the experimental unitary stress $\tau_{med} = 3{,}36$ N/mm$^2$, which shows a good concordance between the calculation formulae used in the Code of ferrocement design" and the experimentally obtained real scale findings.

## 5. Calculation according to the stipulations EUROCOD EC2

The shear force is determined, which could be taken over by the micro-concrete: $V_{Rdc}$

$$V_{Rdc} = [\frac{0{,}18}{\gamma_c} \times k \times (100 \times \rho_1 \times f_{ck})^{\frac{1}{3}} + 0{,}15 \times \sigma_{cp}] \times b_w \times d$$

where:

$\gamma_c = 1{,}5$ – safety coefficient from the table 2.3 from EUROCODE EC2

$$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{1870}} = 1{,}327 < 2$$

$$\rho_i = \frac{A_{si}}{b_w \times d} = \frac{1422{,}1}{70 \times 1870} = 0{,}011 < 0{,}02$$ – longitudinal reinforcement coefficient

$A_{si} = A_{plasa} + A_{tiranti} = 518{,}1 + 904 = 1422{,}1$ [mm$^2$]

$$A_{palsa} = \left(\frac{d-x}{s}\right) \times n \times \frac{\pi \times \phi^2}{4} = (10) \times 4 \times \frac{\pi \times 1^2}{4} = 518{,}1 \quad [\text{mm}^2]$$

$A_{tiranti} = 2 \times 452 = 904$ [mm$^2$]

n = 4 – number of meshes

s = 10 mm – size of the spaces between the bars

Ø = 1 mm – diameter of mesh wire

$b_w = 2 \times 35 = 70$ mm – total thickness of the concrete layer

$f_{ck} = 31$ N/mm$^2$ – characteristic resistance of concrete (experimentally found)

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{116000}{152600} = 0{,}76,$$ where:

$N_{Ed} = q \times d = 64 \times 1{,}87 = 116$ [kN], where:

q = 64 kN/m – vertical load

d = 1,87 m – width of the element

$A_c = 2 \times (1870 \times 35 + 310 \times 35) = 152.600$ [mm$^2$] – total compressed area (at the foot of the element)

It results:

$$V_{Rdc} = [\frac{0{,}18}{1{,}5} \times 1{,}327 \times (100 \times 0{,}011 \times 31)^{\frac{1}{3}} + 0{,}15 \times 0{,}76] \times 70 \times 1870 = 82{,}7 \quad [\text{kN}]$$

$$V_{min} = 0{,}035 \times k^{\frac{3}{2}} \times f_{ck}^{\frac{1}{2}} = 0{,}035 \times (1{,}327)^{\frac{3}{2}} \times (31)^{\frac{1}{2}} = 0{,}297 < 0{,}517$$

It results that $V_{Rdc}$ is well calculated.

It is found that $V_{Rdc}$ = 82,7 kN is less than the lateral force H = 460 kN under which the element was broken: $V_{Rdc} << H = 460$ [kN] –lateral force

We determine the maximum force shear force which can be taken over without the micro-concrete to be broken: $V_{Rdmax}$

$$V_{Rdmax} = \frac{\alpha_c \times b_w \times z \times \upsilon \times f_{cd}}{\cot g\theta + tg\theta} = \frac{1 \times 70 \times 1683 \times 0{,}5 \times 20{,}66}{1+1} = 609 \quad [\text{kN}]$$

where:

$\alpha_c = 1$ – coefficient for unpretensioned structures

$\upsilon = 0{,}6 \times (1 - \frac{f_{ck}}{250}) = 0{,}60 \times (1 - \frac{31}{250}) = 0{,}525$we adopt: $\upsilon = 0{,}5$

z = 0,9 × d = 0,9 × 1870 = 1683 – lever arm

$f_{cd} = \alpha_{cc} \times \frac{f_{ck}}{\gamma_c} = 1 \times \frac{31}{1,5} = 20,66$, where:

$\alpha_{cc} = 1$ – coefficient which takes into account lasting effects

$\theta = 45°$ - angle of inclination of compressed diagonals

$\gamma_c = 1,5$ – safety coefficient from table 2.3 from EUROCODE EC2

We determine the maximum shear force taken over by the transversal reinforcement (assimilated with reinforcement only with cradle stirrups): VRds

$$V_{Rds} = \frac{A_{sw}}{S} \times z \times f_{ywd} \times ctg\theta = \frac{3,14}{10} \times 1683 \times 270 \times 1 = 142684 \quad [N] = 142,7 \quad [kN]$$

where:

$$A_{sw} = 4 \times \frac{\pi \times \phi^2}{4} = 4 \times \frac{\pi \times 1^2}{4} = 3,14 \quad [mm^2]$$ – area of transversal wires

$$f_{ywd} = \frac{f_{yk}}{\gamma_s} = \frac{310}{1,15} = 270 \quad [N/mm^2]$$ – calculation of leak resistance for steel

$f_{yk} = 310\ N/mm^2$ – characteristic leak limit (from table 2.14 – Code ferrocement design)

Remarks:

- The value $V_{Rds}$ = 142,7 kN shows how much wire meshes can take from the lateral force H=460kN;
  The value $V_{Rds}$ = 142,7 kN is also confirmed by the experiment when the trial was performed without tie bars when the lateral force attained was H = 155 ÷ 159 kN;
- The difference $\Delta V_{Rds} = H - V_{Rds}$ = 460 – 142,7 = 317,3 kN is considered to be taken over by the two tie bars;
  $V_T = A_T \times f_{ywT} = 2 \times 452 \times 331 \approx 300.000\ N = 300\ kN$
  $A_T = 2 \times 452 = 904 \quad [mm^2]$
  $$f_{ywT} = \frac{f_{yT}}{\gamma_s} = \frac{381}{1,15} = 331 \quad [N/mm^2]$$
  $f_{yT} = 381\ N/mm^2$ – stress after which TER no longer recorded (entered the leak stage)
- I this hypothesis, the value of the shear force taken over by the entire reinforcement (meshes + tie bars) becomes:
  $V_{Rds}^T = V_{Rds} + V_T = 142,7 + 300 = 442,7\ kN \approx H = 460\ kN$
- The difference between the calculated value and the one experimentally obtained is:
  $\Delta V = 460 - 442,7 = 17,3\ kN$
- This difference can be ascribed to the punch effect created by the reinforcement with pillars and calculated with the formula:
  $$V_{dorn} = N \times 4,12 \times \phi_{bare}^{\frac{2}{3}} \times b_w \times \sqrt[3]{f_{ck}} = 4 \times 4,12 \times 8^{\frac{2}{3}} \times 70 \times \sqrt[3]{31} = 14,5 \quad [kN]$$

in which::

N = 4 – number of bars made of pillars

$Ø_{bare}$ = 8 mm – diameter of the bars made of pillars

The other dimensions were previously defined.

It results that that total lateral force that the consolidated assembly can take over is:

$$V_{total} = V_{Rds} + V_T + V_{dorn} = 142{,}7 + 300 + 14{,}5 = 457{,}2 \text{ kN} \approx H = 460 \text{ kN}$$

It is found that, by applying this method, we have the experimental confirmation of the theoretic model.

## 5. Advantages of the solution

The quality of ferrocement to have a very good resistance to fissuring gives it a great advantage over the use of reinforced concrete. The increased resistance to fissuring, combined with the easiness of applying it, as well as its relatively small weight and the reduced cost, make ferrocement an ideal system for rehabilitating structures.

The main objective envisaged for the consolidation and rehabilitation of masonry structures is represented by the recovery of their portant capacity with as reasonable costs as possible. From the numerous reparation and rehabilitation programs for structures which have used ferrocement, H. Ahmed and L R. Austriaco [5] underline the following aspects:

- good behaviour as regards fissuring;
- capacity to improve certain mechanical peculiarities of consolidated structures;
- the accomplished consolidations allow changes and subsequent reparations;
- the relatively reduced weight resulted from the consolidation systems does not require changes in the structure support system;
- facility to oppose to temperature changes;
- facility to allow good water proofing, without special treatments;
- easiness to purchase necessary materials;
- does not require special technological equipment;
- flexibility to subsequent changes;
- possibility not to alter the architectural concept of the structure and, implicitly, of the building as a result of the consolidation.

To this we should add the advantages linked to shock proofing, behavior as regards vibrations, fire, and last but not east, the advantage to be shaped in any possible form.

## 6. Conclusions

As a result of the trials on elements and of the calculations made on the theoretic model, we can conclude that, irrespective of the calculation method used, the values of the stress (or of the shear forces) are confirmed by the measurements accomplished when trials were performed on real scale models.

Together with other consolidation solutions using the coating of the element (ex. Reinforcement with polymeric meshes, micro-concrete reinforced with wire mesh 4-6mm thick with the distance between the bars of 100x100mm, etc), the solution proposed and checked by the authors of this study represents a viable solution, with a good theoretical and experimental support. Figures 1 shows the trial scheme of masonry panels. Figures 2 shows the way the panel under trial was equipped with measuring apparata. In fig.3 (photo) we presented the panel which underwent the trial using the above mentioned equipment. Figure 4 shows the fissures for the consolidated panel under trial, during the breaking stage. Figure 5 shows the diagram stress – deformation when tie bars are fitted which prevent the panel from falling down. In annex 1, we present the tensiometric data on the bars Ø = 24mm representing the tie bars meant to prevent the panel from falling, while in annex 2 we present the information gathered at the trail stall.

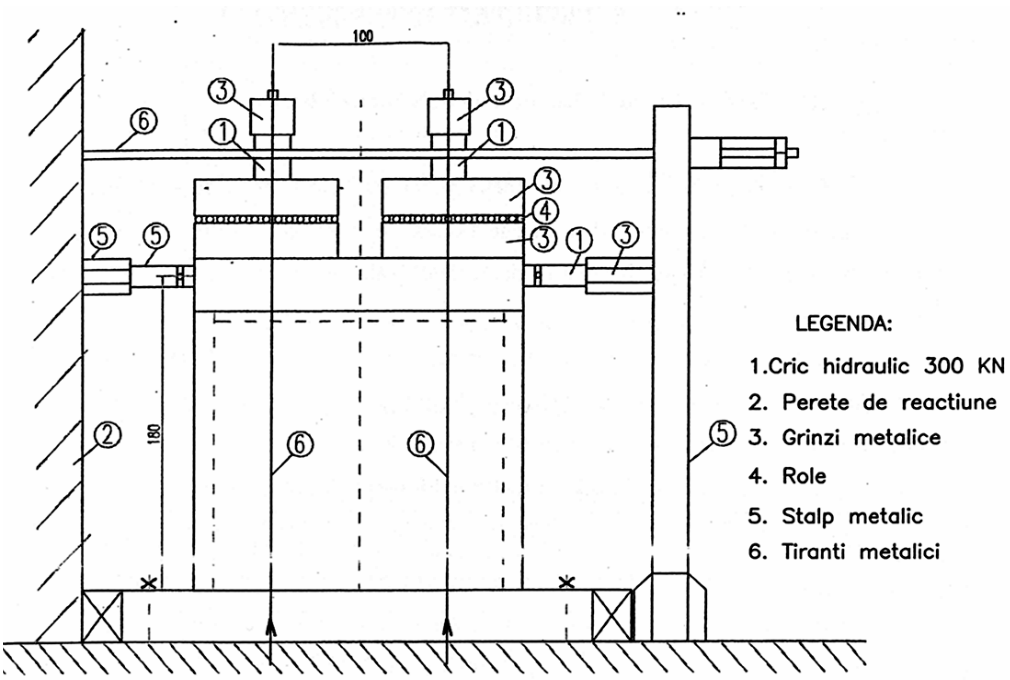



**Fig. 1**. Trail scheme

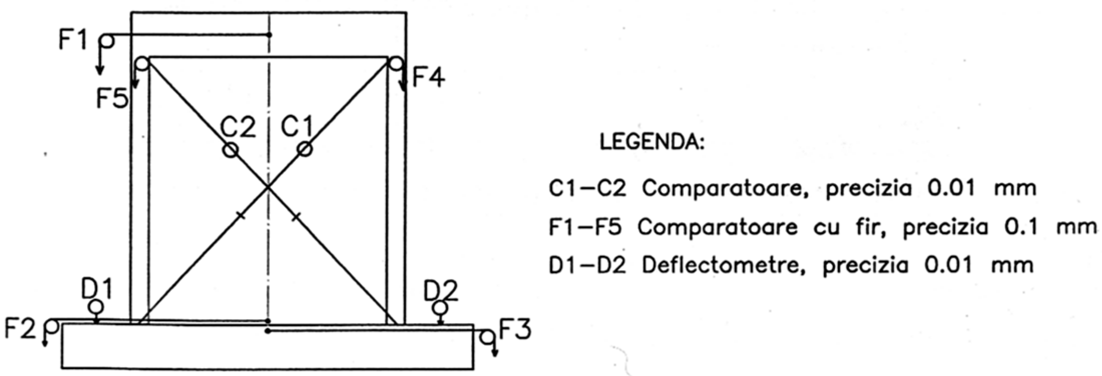



**Fig. 2**. Scheme of measuring equipment


Fig. 3. Equipping of the trial stall for the consolidated masonry element

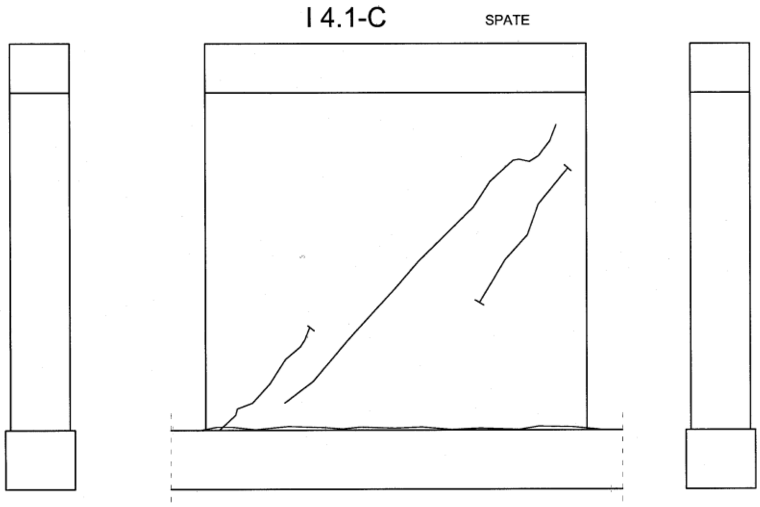



Fig. 4. The fissures for the consolidated element

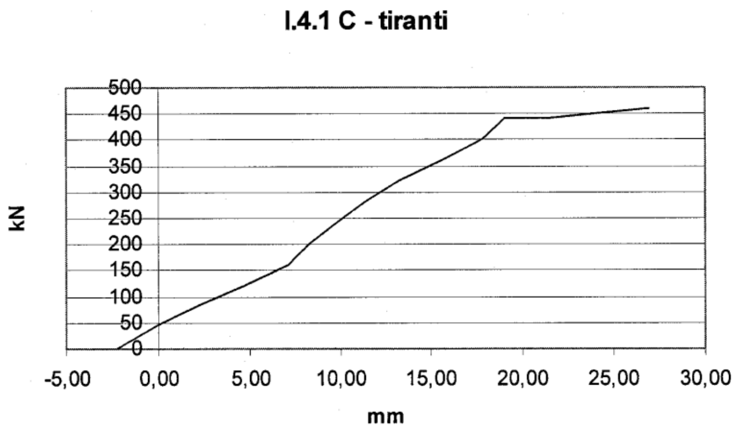



Fig. 5. Diagram force – movement for the consolidated element equipped with tie bars

## ANNEX 1

Etalonare TER pe bare Φ 24 mm

24.05.2005

A = 4,52 cm$^2$

| Forța daN | σ daN/cm$^2$ | Citiri | | Diferența |
|---|---|---|---|---|
| 0 | | 24.000 | 24000 | |
| 1000 | 221 | 60 | 80 | 70 |
| 2000 | 442 | 135 | 140 | 68 |
| 3000 | 664 | 210 | 220 | 78 |
| 4000 | 885 | 300 | 310 | 90 |
| 5000 | 1106 | 380 | 375 | 72 |
| 6000 | 1327 | 460 | 450 | 78 |
| 7000 | 1548 | 565 | 550 | 102 |
| 8000 | 1770 | 650 | 645 | 90 |
| 9000 | 1991 | 740 | 735 | 90 |
| 10000 | 2212 | 830 | 820 | 88 |
| 11000 | 2433 | 890 | 885 | 62 |
| 12000 | 2655 | 985 | 970 | 90 |
| 13000 | 2876 | | | |
| 14000 | 3097 | | | |
| 15000 | 3318 | TER nu mai înregistrează | | |
| 24000 | 5310 | Curgere | | |
| 32000 | 7080 | Rupere | | |

## ANNEX 2

Element I.4.1c Pag 1

| nΔ | Semi Cicl. | H kN | Δ/Δr | Observația |
|---|---|---|---|---|
| I | IS | 83<br>120 | | - Fis la bază α = 1 m |
| | ID | -58<br>-145 | | - fis bază α = 0,5 m.<br>- fis bază α.<br>- fis fundație înclinat - coboară până la bază ~ α=1,0 m. |
| IVD | | -154 | | - fis din fundație = 18 mm. - L=55.<br>- în partea din stânga - se ridică o coajă de beton → afectează D1 - tija metalică înclinată - de aceea cu |
| VS | | | | - fis mare în fundație - se exfoliază betonul de la partea sup. a fundației - nu pot conta pe D1 și D2.<br>- fac pe VD → prin tiranți |
| 27. V. | cu tiranți | | | |
| | | 400<br>440<br><br>440<br><br>460 | | - 2 fis mici stânga sus față<br>- 1 fis înclinată mare - tencuiala pusă - nu știu când a apărut.<br>- 1 mai scurtă sus<br>- fis. spate pe toată lungimea (prin diversificare<br>- ajungem pr... - se activează fundația |

## 7. References

[1] A.C.I. – 549R – 93: State of the Art Report on Ferrocement

[2] A.C.I. – 549 –1R-93: Guide for the Design, Construction and Repair of Ferrocement

[3] B.K.Paul, R.P. Pama: Ferrocement – ITIC Publication, Bankok, Thailand 1978

[4] Oneț T., Vescan V., Szigeti L.: Realizări şi perspective privind utilizarea ferocimentului în construcții. Revista de construcții 2-3/1991

[5] H.I.Ahmed, L.R.Austriaco: State of the Art Report on Rehabilitation and Restrething of Structures Using Ferrocement – Journal of Ferrocement, vol.21, No.3/1991

[6] Mihai Budescu, ş.a.: Reabilitarea construcțiilor. Editura Vesper-2001

[7] Tologea S.: Probleme privind patologia şi terapeutica construcțiilor. Editura Tehnică-1975

[8] Ramiro A. Sofronie: Manual RichterGard. Total Publishing-Bucureşti 2004

[9] Berar T. .: Teză de doctorat-2004, Universitatea „Politehnica" Timişoara

[10] Secula S.: Teză de doctorat-2003, Universitatea „Politehnica" Timişoara

[11] INCERC Filiala Cluj-Napoca:Imbunatâtirea metodelor de calcul al structurilor din zidărie la acțiuni seismice. Faza 2/2001. Incercare elemente.

[12] EUROCODE 2

[13] Kiss Z., Onet T., Muntean G., Indrumator pentru proiectarea betonului armat Ed. U.T. Pres, Cluj Napoca, 2004

[14] Onet T., Magurean C.,Vescan V., Cod de proiectare şi executie a elementelor din ferociment, Universitatea Tehnica Cluj Napoca, Contract MEN 33830/1999

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Performance of Modified Asphalt Mixtures Obtained Using Plastomers Added In Station

Mihai Iliescu[*1], Mariana Pop[2]

[1] *TUCN, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*
[2]*Regional Road Administration Company, Cluj-Napoca, Romania*

## Abstract

*Economic and technical reasons, we are currently witnessing a phenomenon of expansion on the market of additives used in the technology of asphalt mixtures. These products are used in order to improve performance and reduce the costs for asphalt pavement production and exploitation. In general, meeting all these goals by a single product is difficult, because a high quality requires higher costs. Latest research in this area have resulted in products whose quality is to increase performance at minimal cost. This paper presents a comparison of actual performance between an asphalt mixture with PmB modified bitumen and an asphalt mixture modified by adding plastomer type polymer during the manufacturing process, directly in the station. Mixture performance is improved in both cases, roughly the same level, with some specific differences for each type of material. The technology of modified mixtures proved to be beneficial both technical, technological and economic. Mixture characteristics are substantially improved. The risks of compromising the mixture due to technological accidents such as failure of storage temperature and bitumen storage times are excluded in this case. From an economic perspective, the raise of paving preparation and storage temperatures is waived and the modifier has a lower price per ton of mix. The conclusions highlight the criteria for adopting the optimal solution according to performance requirements.*

## Rezumat

*Din considerente economice şi tehnice, asistăm în prezent la un fenomen de extindere pe piaţă a unor produşi de adaos utilizaţi în tehnologia mixturilor asfaltice. Acesti produşi se utilizează cu scopul ameliorării performanţelor în exploatare şi a reducerii cheltuielilor de producere şi exploatare a imbrăcăminţilor asfaltice. Satisfacerea tuturor acestor deziderate de către un singur produs este dificil de realizat deoarece o calitate superioară presupune şi cheltuieli mai mari. Ultimele cercetări în acest domeniu au avut ca rezultat produse a căror calitate constă în cresterea performanţei cu cheltuieli minime. Acestă lucrare prezintă un studiu comparativ al performanţelor realizate între o mixtură asfaltică cu bitum modificat PmB şi o mixtură asfaltică modificată la staţie prin adăugarea polimerului de tip plastomer direct la staţie în cursul procesului de fabricaţie. Performanţele mixturii sunt imbunătăţite în ambele cazuri, aproximativ la acelaşi nivel, cu unele diferenţe specifice fiecarui tip de material.Tehnologia de modificare a mixturii s-a dovedit a fi avantajoasă atat tehnic, tehnologic cat şi economic. Caracteristicile mixturii sunt substanţial ameliorate. Riscurile de a compromite mixtura datorită unor accidente tehnologice cum ar fi nerespectarea temperaturilor şi a duratelor de stocare a bitumului sunt excluse în acest caz. Din punct de vedere economic se renunţă la cresterea temperaturilor de stocare preparare şi asternere, iar modificatorul are un pret mai redus raportat la tona de mixtură. Concluziile evidenţiaza criteriile de adoptare a soluţiei optime conform cu cerinţele privind performanţa.*

**Keywords:** asphaltic mixture / modified bitumen/ plastomer / elastomer

[*] Corresponding author: Tel./ Fax.: 0264 401249 / 0264 594967
E-mail address: mihai.iliescu@cfdp.utcluj.ro

## 1. Generalities. Performance in service of asphaltic pavements

Asphalt proved to be ideal for road construction, due to the performance targeting technical, technological, economic and ecological aspects: resistance to mechanical stress, aggression of atmospheric factors, comfort and safety of traffic, simple and fast preparation and execution technologies, short execution time without disturbing traffic, reducing noise, fuel consumption, harmful emissions or the degree of wear of tires in service.

In order to ensure these requirements to a high level of performance, various technologies were researched, in particular on products that are added in the mixture composition, to improve some of its characteristics.

Research and application of these technologies is based on advanced methods of performance investigation in the laboratory. These methods, as opposed to the empirical ones, allow a detailed study of fundamental properties of mixtures , which reflect the real way they respond to requests in service.

## 2. Aspects of using modifiers in asphalt mixture production

To reflect the various issues related to the use of thermoplastic polymer modifiers, a comparison was made, between the classical solution of modified bitumen and the newest solution , of direct modified mix.

The most commonly used asphalt mixtures modifiers are thermoplastic polymers. They are characterized by being able to reversibly change their state under the influence of temperature. In the domain of thermoplastic polymers there are two families that differ, mainly, by characteristics related to stiffness, elasticity, deformability: elastomers and plastomers.

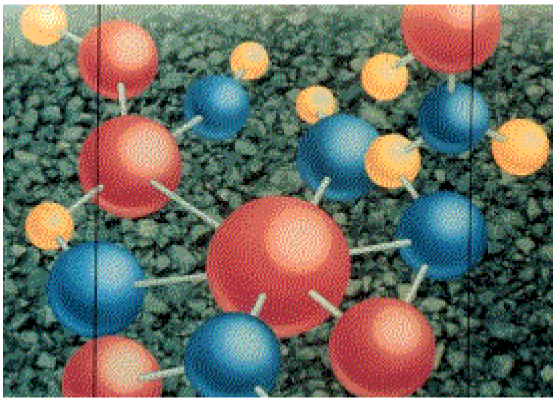


Figure 1. Representation of a polymer

Elastomers require a supply of mechanical energy in order to be mixed with the bitumen. For this reason, they are mixed with bitumen before the mixture is prepared.

Plastomers do not require additional power for mixing, thus they can be previously mixed with bitumen or they can be introduced directly into the mixer. The solution of introducing the modifier directly into the mixer can be adopted if there are only plastomers used. In case of plastomers with elastomers association, the bitumen will be previously modified . The modifying solution is specific to each case, according to technical requirements and economic possibilities.

Figure 2. Mixture with modified bitumen

The solution of previously modifying bitumen with SBS elastomer in order to obtain bitumen PmB, was far more widespread because of the potential analysis of the properties of bitumen before use.

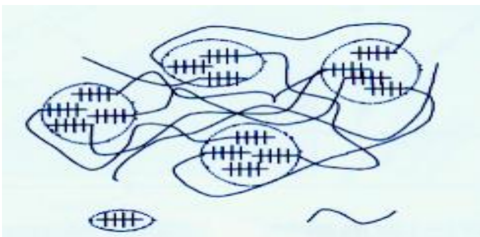
Modifying in the mixture solution is newer, it is much easier, complications associated with the transport and storage of modified bitumen and also with the high energy consumption for its production are eliminated. In this case, the modifier is in the form of granules (Figure 4) which are inserted directly into the mixture, without changing very much the manufacturing process. Temperatures are generally the same, it is one that requires more attention, that is the compaction temperature, which should be at least 130 °.

Following a comparative analysis of performance,advantages and disadvantages of these solutions in all stages of production and exploitation, it came out that the technology of modifying mix in the station is easier and more economically advantageous, offering the same level of performance with the use of modified bitumen solution.

## 3. Generalities regarding the thermoplastic polimers

Elastomers: SBS styrene-butadiene-styrene, styrene-isoprene-styrene, SIS, styrene-butadiene SB, have a heterogeneous structure consisting of clusters of polystyrene and polybutadiene chains. These polymers raise some issues related to:

- compatibility with bitumen,
- storage stability
- high viscosity, that can cause problems during manufacturing and application of the mixtures.



polistyrene polybutadiene chains

Figure 3. Schematic representation of the structure of an elastomer

Modified bitumens with elastomer are usually obtained by a strong mechanical mixing at a higher temperature than the flow temperature of the polymer. In some cases, dispersion chemical agents may be used, in order to optimize the operation, leading to more homogeneous mixtures.

Usually, the elastomer type binders are kept at high temperatures (T> 140 º C) with agitation.

Benefits of using elastomers are:

• considerable reduction of thermal susceptibility
• increased flexibility at low temperatures
• increased stiffness at high temperatures

Disadvantages in the use of elastomers are:

• increased viscosity at high temperatures
• limited stability at storage
• additional energy consumption for transport, storage and application

The most commonly used plastomers in road works are: EVA (ethylene and vinyl acetate), EMA (ethylene vinyl acrylate), EBA (ethylene and butyl acrylate). Their structure is composed of a hydrocarbon skeleton which provides rigidity and cohesion, including the crystalline fractions which regulate thermal susceptibility, on which is fixed the polar comonomer, allowing to control the compatibility of adhesivity and cristallinity. So, finally, the comonomer determines the basic properties of plastomers.

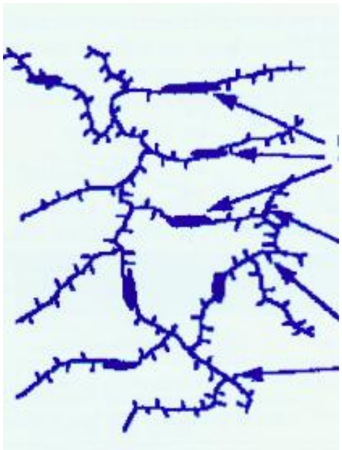



Figure 4. Schematic representation of a plastomer

Benefits of using plastomers are:

- decreased thermal susceptibility
- increased stiffness at high temperatures

Disadvantages in the use of plastomers are :

- fragility at low temperatures

## 4. Comparative study in the laboratory of mixtures with elastomer and plastomer

In order to eliminate the disadvantage related to the fragility at low temperatures of plastomers, the ideal solution is to achieve MASFm mixtures that have a higher dosage of bitumen and where fiber is associated with plastomers. For elastomers, the association with fiber in a MASFm mixture does not involve a substantially quality intake.

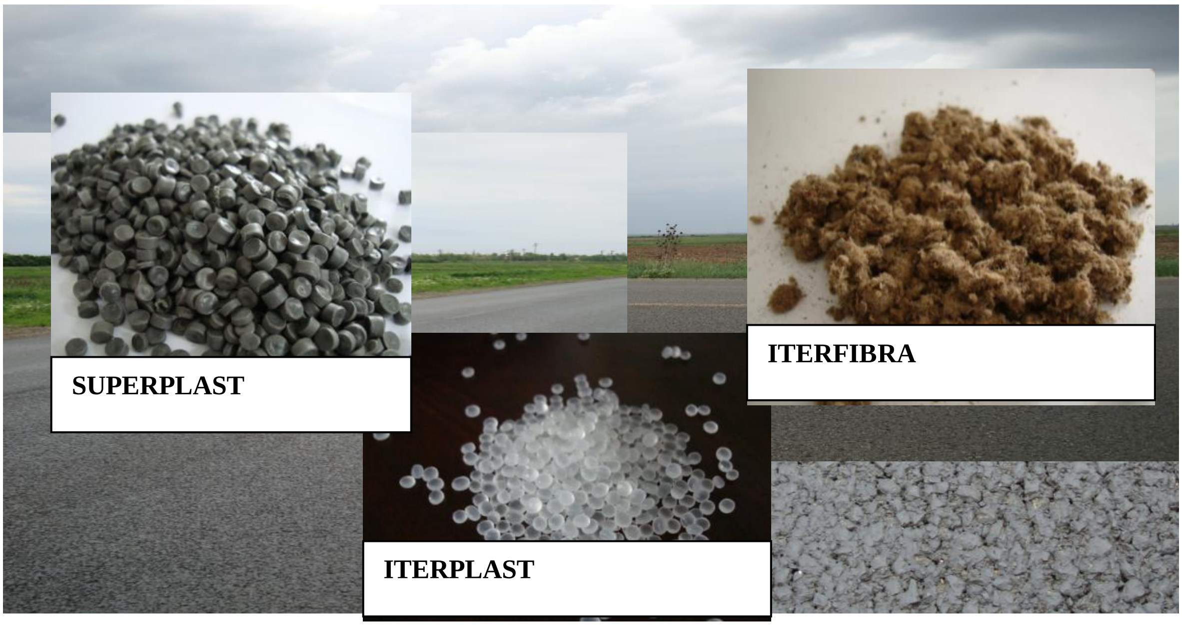



Figure 4. Products used at asphalt mixtures manufacturing

In laboratory were analyzed 5 (five) asphalt mixtures:

1. BA16 asphalt concrete with bitumen according to SR 174
2. Reference mix AR16 with granular frame characteristic for MASF, with bitumen without fibres
3. MASF16 asphaltic mixture stabilized with fibers ITERFIBRA - Iterchimica Italia
4. Reference mix AR16m with granular frame characteristic for MASF, with modified bitumen without fiber
5. MASF 16m asphalt mixture, with modified bitumen, stabilized with fibers

In case of AR16m and MASF 16m the modification was made in two ways:

- Previously modified bitumen, with elastomer SBS : PmB 45/80-65 Lotos Polonia
- Modified directly in the mixture, with plastomer: SUPERPLAST - Iterchimica Italia

The purpose of the analysis was to highlight the influence of the mixture of: mineral frame, fibres, elastomer, plastomers.

The test results are presented graphically and are set by fundamental characteristics: stiffness modulus, ornierage, dynamic flow and fatigue resistance.

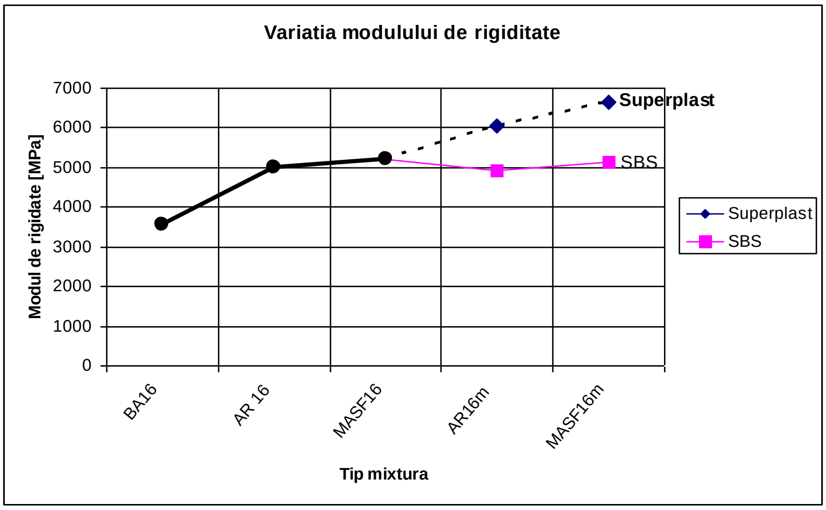



Figure 5. Stiffness modulus

For the stiffness modulus ( Figure 5) it came out:

- the contribution of the mineral frame at modulus increase
  - the stiffening effect more obvious in case of plastomers added directly in the mixture
  - the positive effect of collaboration between the fibres and plastomers

After the ornierage test ( Figure 6 ) came out :

- the effect of the mineral frame over the resistance to permanent strains increase
- introducing only the fibers has only a relatively small contribution in increasing the resistance to permanent strains
- the elastomer has a higher effect of increasing the ornierage resistance

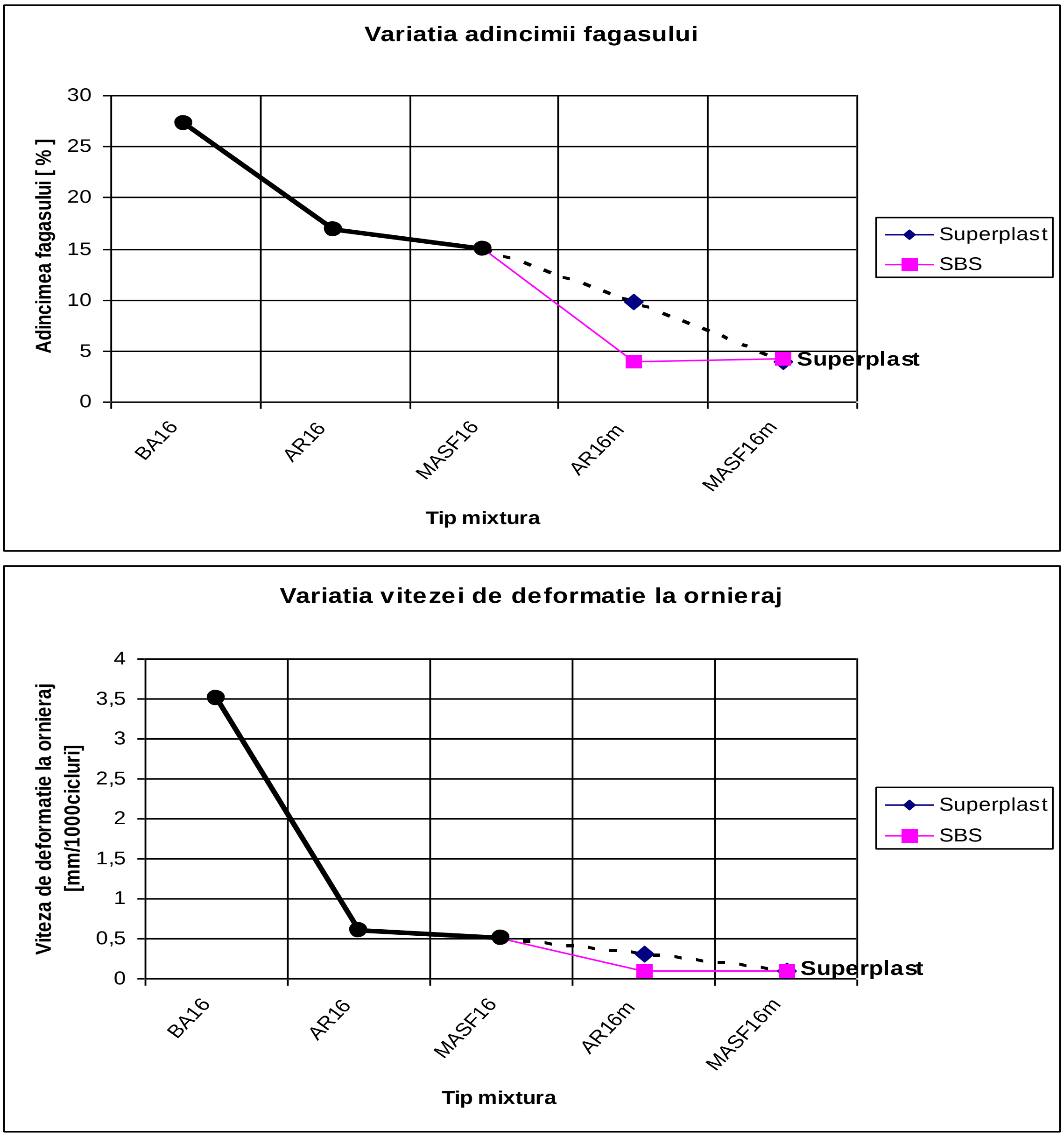
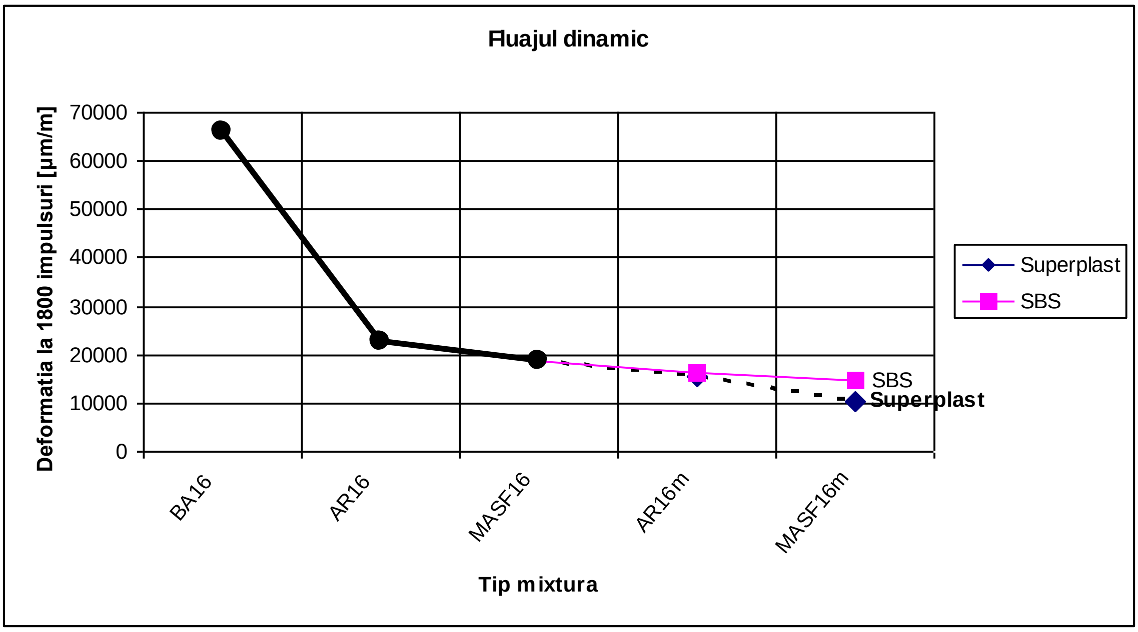



Figure 6. Ornierage test

- Plastomers in combination with fiber and elastomer in combination with fibers have approximately the same resistance at ornierage
- Using elastomers, the fiber supply does not improve the performance
- The strain rates are aproximately simillar for the mix with plastomer and elastomer.



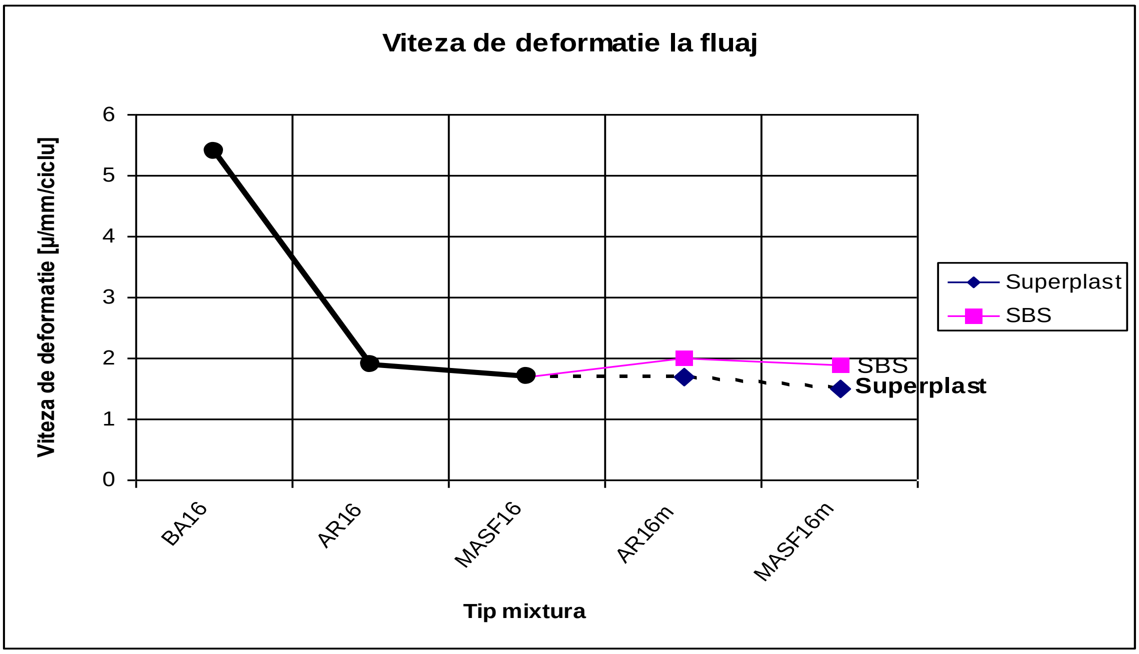




Figure 7. Dynamic flow

For the dynamic flow test ( Figure7):

- the behavior of mixtures with plastomers and elastomers are similar
- Superplast, combined with the fiber, ensure the highest resistance to dynamic flow
- The mineral frame has an important role in order to reduce dynamic flow

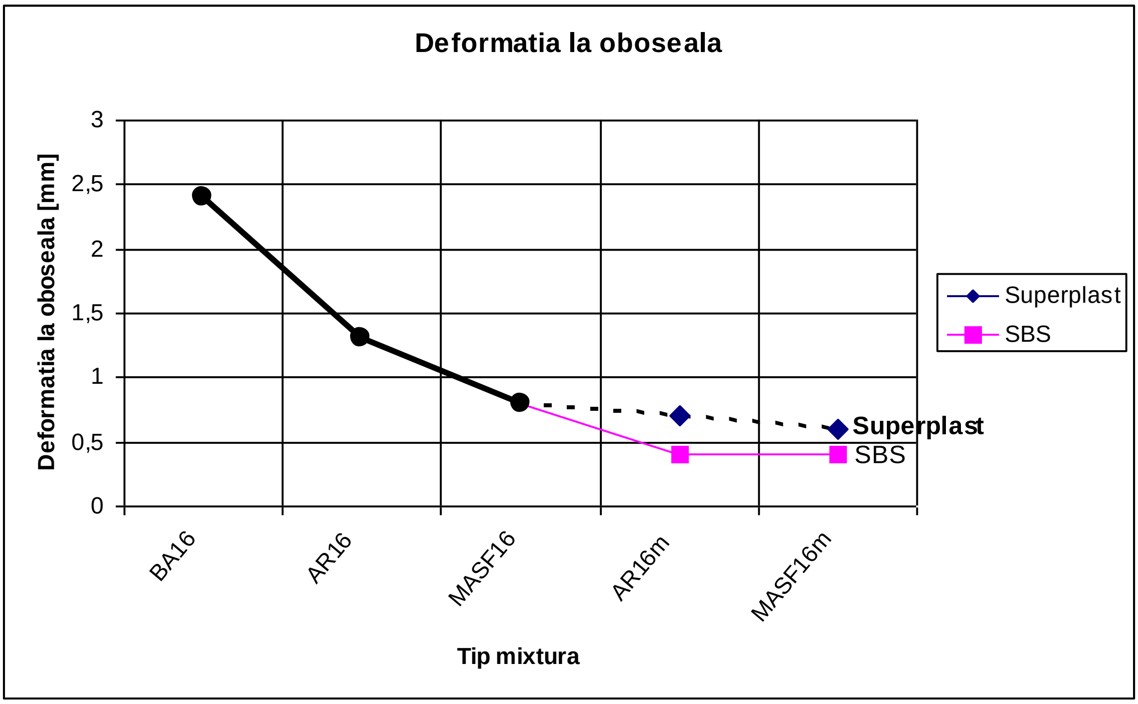



Figure 8. Fatigue resistance

Fatigue resistance ( Figure 8 ) :

- the mixture with SBS has a better fatigue resistance
- the mineral frame influence the resistance to repeated loads

In Table 1 are compared the results of the analyses made on mixture with elastomer SBS and plastomer Superplast, both in combination with fiber.

Comparative results are generally observed. MASF mixture with Superplast is stiffer.

Table 1. Comparison between asphaltic mixtures with polymers performance

| **CHARACTERISTIC** ***UM*** | **MASF16m (SBS)** | **MASF16m (SUPERPLAST)** | **LIMITS** |
|---|---|---|---|
| **Stiffness modulus at 15 ° C** ***Mpa*** | **5128** | **6654** | **Min. 4500** |
| **Fatigue strain at 15 ° C** ***mm*** | **0,4** | **0,6** | **Max. 1** |
| **Dynamic flow at 50 ° C Strain** ***µm/m*** | **24538** | **20457** | **Max. 30 000** |
| **Dynamic flow at 50 ° C Strain rate** ***µm/m/cycle*** | **1,9** | **1,5** | **Max. 3** |
| **Ornierage at 60 ° C Strain rate** ***mm/ 1000 cycles*** | **0,2** | **0,1** | **Max. 0,6** |
| **Ornierage at 60 ° C Ruts depth** ***%*** | **4,2** | **4,0** | **Max. 7** |

## 5. Conclusions

1. In order to obtain performance in asphalt mixtures, the mineral frame will be considered having a substantial influence on the fundamental characteristics.
2. If conditions are imposed on the resistance to extreme temperatures, is appropriate to use modified MASFm mixtures with fibres and plastomer or asphalt concrete mixtures made with bitumen modified with elastomer.
3. Using plastomers added directly in the mixture is a technical, technological and economic advantageous solution, which will contribute to a faster execution and a higher level of performance for the road works.

## 6. References

[1] Yves Brosseaud. Les liants modifiés et les additifs dans la fabrication des enrobés modifiés ou spéciaux. 2007

[2] Iterchimica 2008. Road and Airfield uses of SUPERPLAST Modified Hot mix Asphalt

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Estimation of Vehicle Road Emissions Factors Using Copert III Methodology

Cristian Tosa*[1]

[1] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 72-47 Observatorului Str., 400363, Cluj-Napoca, Romania*

## Abstract

*In this paper we used a computer model called Copert III to assess vehicle emissions from road transport. For the case study we used passenger cars, gasoline and diesel powered for which we obtained emission factors for carbon monoxide (CO), particulate matter (PM) and nitrogen oxides ($NO_X$), for different average speed profiles. We approximated the emission function as a second degree polynomial and through graph fitting operations we obtained a general equation for the emission factors. Having the speed dependant emission curve, we obtained an average speed for the studied vehicle categories for which the emission factors are minimum.*

## Rezumat

*În această lucrare am folosit un model de calcul, Copert III pentru estimarea emisiilor poluante rezultate din transportul rutier. În studiul de caz am obtinut emisiile poluante de monoxid de carbon (CO), particule fine (PM) si oxizi de azot ($NO_X$) pentru diferite viteze medii de deplasare la vehiculele de pasageri care folosesc combustibili fosili, precum benzina sau motorina. Aproximând graficul emisiilor poluante cu o functie polinomială de gradul al 2-lea, am obtinut o ecuatie generală a emisiilor, având ca si variabilă dependentă viteza de deplasare. În final am obtinut viteza medie pentru categoriile de vehicule studiate, pentru care emisiile poluante sunt minime.*

Keywords: air quality, COPERT III, emission factor, polynomial fitting, minimum emission speed, low emission zones

## 1. Introduction

Air quality in urban areas continues to be a cause of concern in consideration to environmental impacts and health. Environmental effects of road transport include air pollution, pollution of natural drainage systems, noise pollution and energy consumption [1]. Many cities and congested towns across the world are experiencing high levels of air pollution arising from emissions of different sources, road traffic being the predominant source in most urban areas. Therefore, controlling strategies need to be implemented to minimize the environmental impacts and increase the motorized transport sustainability [2].
When dealing with air quality problems, climate change and pollution mitigation measures, the estimation of road traffic emissions has become increasingly important for evaluation of environmental policies and proposed infrastructural developments [3]. The continued growth in vehicle use and the deterioration in driving conditions, such as congestion [4] amplified the

* Corresponding author: Tel./ Fax.: +40-726-383065
E-mail address: cristian.tosa@dpcf.utcluj.ro

contribution of the traffic emissions to the worsening of the air quality, especially in the urban environment. Many authorities run into frequent difficulties to meet the air quality standards and national emission ceilings, and, in consequence, there is a stringent need for reliable emission inventories in order to ensure that the road transportation system contribution is correctly assessed [5].
To represent the real conditions in an accurate way, the inventories should take into consideration local traffic measures and technological developments on fuel consumption and emissions. To be reliable and credible, emission calculations should be performed as a function of different parameters, like intelligent traffic systems, dynamic speed limits, ramp metering, etc. [4]. It is not a trivial task and therefore road traffic emissions and inventories are performed using several specialized emission models.

## 2. Methods for estimating vehicle emission factors

### *2.1 Emission model*

The vehicle emissions models are created to estimate emissions from different types of motor vehicles. The models are intended to help authorities develop emissions estimates in order to implement and make available the most effective control strategies and transportation planning. These models also help the decision factors of the local or national authorities to predict how different strategies would affect local emissions and, in consequence, decide the appropriate procedures in order to reduce the emissions over time. The different models developed make estimates of local air pollutants, greenhouse gas emissions, and toxic pollutants, depending on several input user data [6].

### *2.2 Air quality terminology*

In the following we will present some of the specific terminology used in vehicle emission research area. Air quality model is defined as air quality impact assessment tool. The air quality model is classified in 2 types, mathematical and physical model [7]. These are used to obtain the vehicle emission characteristics. In this article we will use a mathematical model which is defined as a set of mathematical equations, which explain physical and chemical atmospheric processes. The result of the calculations show the relations between the emission rates and air quality. The emission rates represent the mass of the air pollutants emitted from a source every unit of time. Emission factors are an approximation of the quantity of a particular pollutant that would be emitted by the typical or average vehicle, depending on vehicle classification [7].

### *2.3 Copert III methodology*

This paper uses a widely used methodology to assess vehicle emissions from road transport, namely the model designed and developed as a computer software called Copert. It is an MS Windows software program aiming at the calculation of air pollutant emissions from road transport. The technical development of COPERT was financed by the European Environment Agency (EEA), in the framework of the activities of the European Topic Centre on Air and Climate Change. Since 2007, the European Commission's Joint Research Centre has been coordinating the further scientific development of the model. In principle, COPERT has been developed for use from the National Experts to estimate emissions from road transport to be included in official annual national inventories. However, it was designed in such a way that it is available and free so that it can be used in any other research, scientific and academic applications.
In the following we will describe the features of the software and present examples of assessing vehicle emission by engine class and average speed. In the final part of the article we will use the

graphs obtained and try to find a general formula of basic emission factors, using polynomial fitting operations.
Copert III is a tool for the calculation of emissions from the road transport sector. The emissions calculated include regulated (CO, $NO_x$, VOC, PM) and unregulated pollutants ($N_2O$, $NH_3$, $SO_2$, etc.) and fuel consumption is also computed. The new features of Copert III include the effect of vehicle age (mileage degradation) on emissions, and options on gasoline and catalyst vehicles. Not all of those elements are necessary for the compilation of annual national inventories. However, they are of significant value when different scenarios need to be run for case studies and completion of an annual national emission data set from road transport [8].
The COPERT III methodology is assumed to reflect real world conditions, but it is not fully clear from its documentation to what extent fuel consumption estimates have been based on official test cycle results and to what extent they are based on measurement of real world cycles [9]. Yet, we used this methodology in order to obtain emission data, which represents the amount of pollutant depending on vehicle mileage, engine capacity and average running speed. The software takes in account the country specific monthly air temperature values, and some fuel characteristics when computing emission factors.

## 3. Case study

According to Copert III, vehicles can be classified into the following categories: passenger cars, light-duty vehicles, heavy-duty vehicles and busses, mopeds and motorcycles. In the following we will deal with the passenger cars, gasoline and diesel powered. The emission factors which we obtained for different engine categories are carbon monoxide (CO), particulate matter (PM) and nitrogen oxides ($NO_X$).
The gasoline and diesel powered vehicles were separated into engine capacity and emission class, as seen in Table 1.

Table 1: Passenger vehicles classification

| Fuel type | Engine capacity ($cm^3$) | Vehicle technology | COPERT III computed pollutants |
|---|---|---|---|
| Gasoline | < 1400 | Euro I, Euro II, Euro III, Euro IV | CO, $NO_X$ |
| | 1400 … 2000 | | |
| | > 2000 | | |
| Diesel | < 2000 | Euro I, Euro II, Euro III, Euro IV | PM, $NO_X$ |
| | > 2000 | | |

As the emission factors depend on the vehicle average speed, we used as input data different driving speeds. We need to mention that we used the urban option as the environment in which the vehicles release pollutants in the ambient air, mostly because of the reduced vehicle driving speed profiles and significant amounts of pollutant emitted. We exported the results into output data files, and we obtained the speed dependant pollutant emission factors. In order to process reliable results, we choose only some categories of vehicles which are representative for Romania's passenger vehicle fleet.
Figure 1 and Figure 2 show carbon monoxide and nitrogen oxide emission at gasoline vehicles with the engine capacity between 1400 and 2000 $cm^3$ for the above mentioned vehicle technologies.

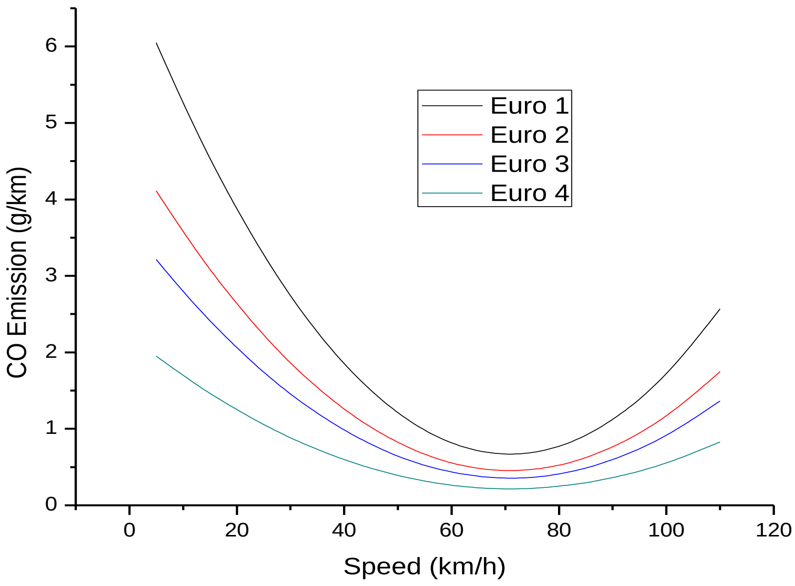



Fig. 1. Gasoline (1400... 2000 $cm^3$) carbon monoxide emission.

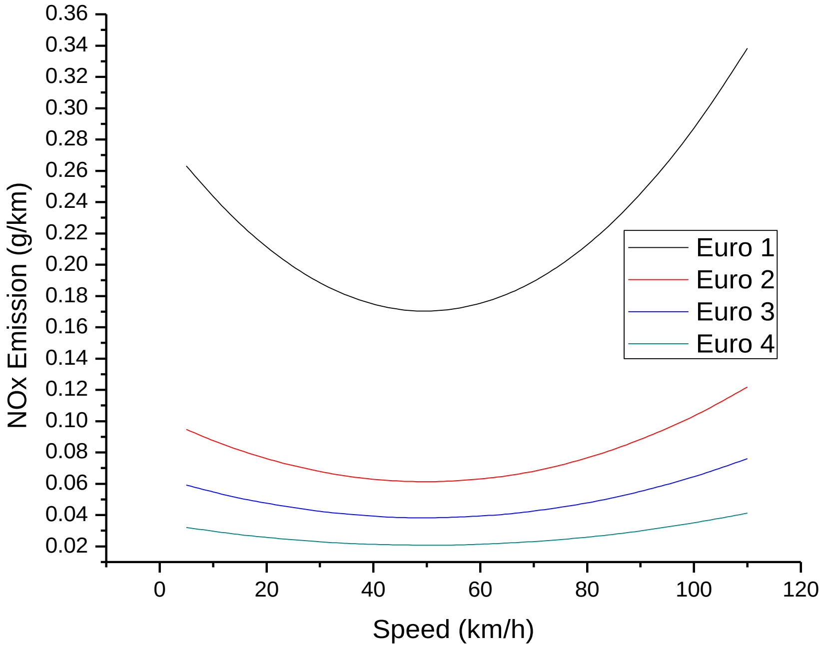



Fig. 2. Gasoline (1400... 2000 $cm^3$) nitrogen oxide emission.

Figure 3 and Figure 4 show the $NO_X$ and PM emission factors for diesel engines with engine capacity smaller than 2000 $cm^3$. The carbon monoxide emissions are not significantly important as in the case of gasoline engines.

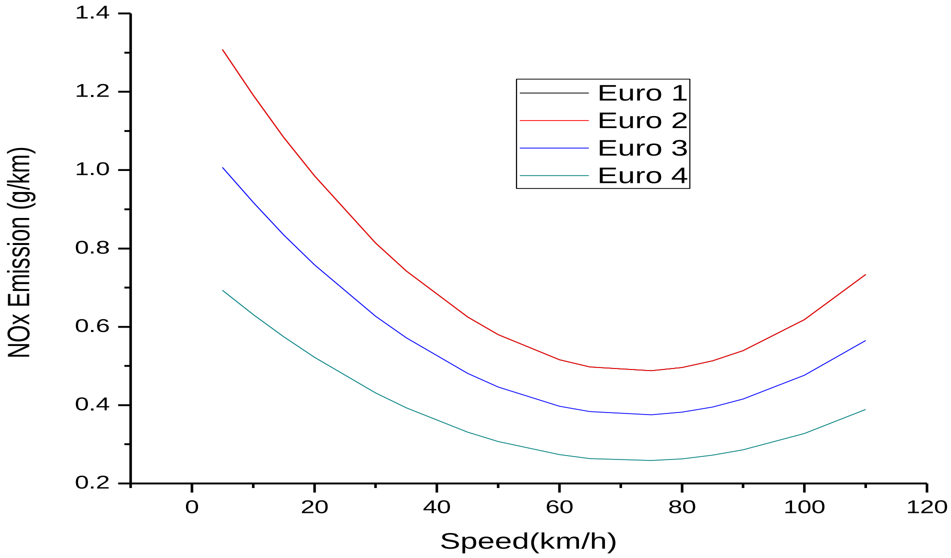



Fig. 3. Diesel ( <2000 cm$^3$) nitrogen oxide emission.

One can observe that in Figure 3 and Figure 4 the emission curves for the Euro I and Euro II categories slightly concur.

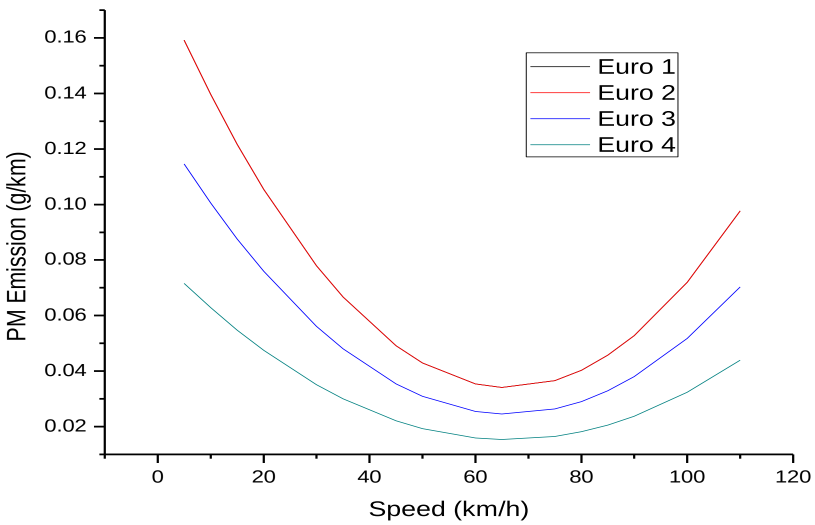



Fig. 4. Diesel ( <2000 cm$^3$) particulate matter emission.

Having these average speed dependant emission factors, in the following we will deduce a general formula, a parametric equation which uses a single dependant variable, speed.
We choose for the case study, carbon monoxide emissions of Euro II gasoline powered vehicles with engine capacity between 1400 and 2000 cm$^3$ and the nitrogen oxide emissions of the Euro III diesel powered vehicles, with the engine capacity below 2000 cm$^3$.
We approximated the emission function as a second degree polynomial. Thus, in the case of gasoline engines, after the emission graph fitting operation, we obtained the following general emission equation:

$$Y = A + B_1V + B_2V^2 \qquad (1)$$

where: Y is the emission factor, A is the intercept and $B_1$ and $B_2$ are the slopes and V is the vehicle speed. The numerical values and standard errors are presented in Table 2.

Table 2: Parameters of Equation 1. Gasoline engines, Euro II

| Emission | Coefficients | Values | Standard Error |
|---|---|---|---|
| CO | A | 3.71552 | 2.4065E-08 |
| | B1 | -0.10819 | 1.0208E-09 |
| | B2 | 8.30E-04 | 8.9450E-12 |

The emission equation in the case of diesel engines has the same degree and shape, the parameters being highlighted in Table 3.

Table 3: Parameters of Equation 1. Diesel engines, Euro III

| Emission | Coefficients | Values | Standard Error |
|---|---|---|---|
| $NO_X$ | A | 1.10379 | 1.9451E-08 |
| | B1 | -0.02002 | 8.2506E-10 |
| | B2 | 1.37E-04 | 7.2300E-12 |

From the plotted curves, one can see that the emission curves present a peak point. If we make an analysis on the fitted functions, we can provide the vehicle speeds for which the emission factors have the minimum values. The data is presented in Table 4.

Table 4: Minimum emission factors

| Engine technology | Fuel type | Emission | Engine capacity | Minimum emission speed (km/h) |
|---|---|---|---|---|
| Euro II | Gasoline | CO | 1400…2000 | 61.21 |
| Euro III | Diesel | $NO_x$ | < 2000 | 72.87 |

## 5. Conclusions

The purpose of this study is to provide local authorities information on urban traffic recommended speeds. Municipalities need to adapt to the rising urban traffic and to take measures against air pollution. In cities with predominantly congested traffic, strategies need to be implemented so that vehicle speed needs to be increased without significant influence on crowded pedestrian areas. Pedestrian flows need to be separated from vehicle flows, by building underground or over ground passages. Having a speed interval with low emission factors of different pollutants, taking into account vehicle engine technology and size, local authorities can implement the urban low emission zones. These are areas or roads where the most polluting vehicles are restricted from entering. This means that vehicles are banned, or in some cases charged, if they enter the low emission zone when their emissions are over a set level [10]. Thus, separation of motorized and non-motorized vehicles combined with an implementation strategy of the low emission zones will guide to urban sustainability and more healthy traffic participants.

## Acknowledgements
This paper was supported by the project "Doctoral studies in engineering sciences for developing the knowledge based society-SIDOC" contract no. POSDRU/88/1.5/S/60078, project co-funded from European Social Fund through Escorial Operational Program Human Resources 2007-2013.

## 6. References

[1] Brown, A.L., Affum, J.K. A GIS-based environmental modelling system for transport planners. *Computational Environmental Urban Systems*, **Vol. 26**, pp. 577-90. 2002.

[2] O'Donoghue R.T., Broderick B.M., Delaney K. Assessing the impacts of infrastructural road changes on air quality: A case study. *Transportation Research Part D*, **Vol. 12**, pp. 529-36. 2007.

[3] Smit R, Poelman M, Schrijver J. Validation of road vehicle and traffic emission models - A review and meta-analysis. *Atmospheric Environment*, V**ol. 44**, pp. 2943-53, 2010.

[4] Smit R, Poelman M, Schrijver J. Improved road traffic emission inventories by adding mean speed distributions. *Atmospheric Environment*, V**ol. 42**, no. 5, pp. 916-26, 2008.

[5] Noland, R.B., Quddus, M. Flow improvements and vehicle emissions: effects of trip generation and emission control technology. *Transportation Research-D*. **Vol. 11**, pp.1-14, 2006.

[6] Davis N., Lents J., Osses M., Nikkila N., Barth M. Development and Application of an International Vehicle Emissions Model, Technical Paper, *Transportation Research Board Annual Conference*, Washington, D.C., 2005.

[7] Thitisakdanon P. The dispersion of traffic air pollutants by using street canyon model in Airviro: Case study Ladprao Road. Master Thesis, Faculty of Graduate Studies, Mahidol University, 2005.

[8] Kouridis C., Ntziachristos L, Samaras Z. COPERT III Computer programme to calculate emissions from road transport. Technical Report No. 50, *European Environmental Agency*, 2000.

[9] Hrebícek J., Hejc M., Holoubek I. Current Trends in Environmental Modeling with Uncertainty. *3rd Biennial meeting of the Int. Env. Modeling and Software Society*, Vermont, U.S.A., 2006.

[10] Low emission zones in Europe, *Action Plan on Urban Mobility*. Website: http://www.lowemissionzones.eu.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
***Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng**

# Thermo-Energy Analysis of a Building for Housing with Insulation Placed Onto the Interior Surface of the Cover

Mayer Zsombor [*1], Mihaela Costin[2]

[1,2] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

The building is designed as a building with the thermal insulation placed on the interior surface of the opaque elements of the cover of each apartment of the building. The apartments of the building are thermal insulated both from the external environment and from the neighbouring apartments, from the staircase, girdles, common areas of the buildings etc.
Interior air quality and humidity control is done with heat recovery devices installed in rooms or by using heaters which are equipped with such devices.

## Rezumat

Constructia este conceputa ca o cladire cu izolatia termica amplasata pe suprafata interioara a elementelor opace ale anvelopei fiecarui apartament al cladirii. Apartamentele cladirii sunt izolate termic atat de mediul exterior cat si de celelalte apartamente invecinate, de casa scarii, ghene de gunoi, spatii comune ale cladirii, etc.
Controlul calitatii si umiditatii aerului interioar se face cu dispozitive cu recuperarea caldurii montate in incaperi sau prin utilizarea de instalatii de incalzire care sunt dotate cu asemenea dispozitive.

**Keywords:** Insulation, energy, extruded polystyrene, humidity, building.

## 1. Introduction

The building is designed as a building with the thermal insulation placed on the interior surface of the opaque elements of the cover of each apartment of the building. The apartments of the building are thermal insulated both from the external environment and from the neighbouring apartments, from the staircase, girdles, common areas of the buildings etc.

The particularity of the calculation of the energy need for this type of buildings:

- The need of energy is determined independently for each apartment of the building;
- The thermal inertia of the walls is not included in the calculation;
- The heating of the apartment may be done by systems installed in the flooring or by air heaters or by air-conditioning;

[*] Corresponding author: E-mail address: mihaela.costin@cif.utcluj.ro

- The apartment does not require emergency heating because after the heating installation is turned on, the air temperature inside increases in a relatively short time.

The extruded polystyrene used on the interior surfaces, having a high resistance factor for vapor diffusion, provides the protection of the walls against water vapor diffusion.
Interior air quality and humidity control is done with heat recovery devices installed in rooms or by using heaters which are equipped with such devices.

## 2. Thermaltechnical characteristics

The physical and thermaltechnical characteristics of the building materials and of the glazed surfaces were taken into consideration on the basis of the existing norms and technical documentation.
The calculation was made using the insulation type of the elements of the cover of the building:

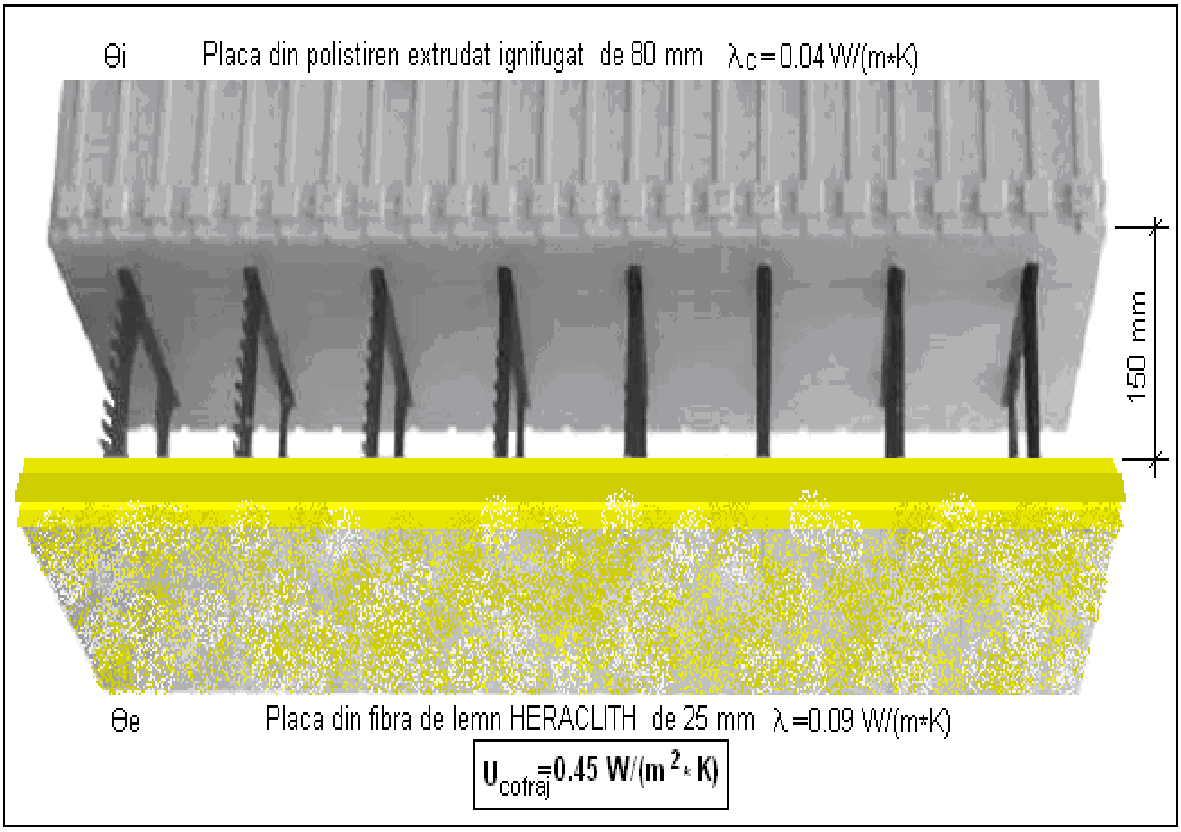
Opaque elements:

- Exterior walls: 8 cm of extruded polystyrene on the interior +2.5 cm Heraklit on the exterior;
- Interior walls on the apartment cover: 5 cm of double sided expanded polystyrene;
- Mansard roof: 20 cm of fire-proof mineral wool;
- Terrace roof: 15 cm of extruded polystyrene;
- Intermediary ceilings: 5 cm of double sided expanded polystyrene;
- Foundation of the building: 5 cm of extruded polystyrene.

Glazed elements:

- The case of the glazed surface from 6-room PVC profiles;
- The window made of 3 sheets of glass: 4+14+4+14+4 mm (40 mm);
- Filling gas: Krypton;
- Treatment: 2 treated glass sheets having the radiating capacity <=0.20;
- The thermal performance of the glazed surfaces (according to SR EN ISO 10077/1-2:2002:03) for:
  - Window transmittance Ug = 0.50 W/($m^2$*K) ;
  - Case transmittance Uf = 1.10 W/($m^2$*K) ;
  - Glazed surface transmittance $Uw_{med}$ =1.09 W/($m^2$*K) .

The ceilings are made of reinforced concrete cast in position. The interior resistance walls are made of membranes made of reinforced concrete cast in position. Before pouring in the concrete, the thermal insulation sheets are placed on the sides of the concrete forming.
The exterior resistance walls are made of reinforced concrete cast in position poured in a thermal-insulating concrete forming specially designed for this.




Picture1. Proposition for the prefabricated modulated element for the insulation and concrete forming of the exterior walls

Wall finishes:

- A classic plaster coating of 1.5-2.0 cm on the exterior surface of the walls;
- A thin coat of plaster specific for thermal-systems on the interior surface.

In the APPENDIX you will find as a selection of examples, the energy performance certificate of the building placed in the first climatic area, as well as the numeric results obtained by apartments and by the whole building for the first climatic area, as well for the real building and for the reference building.

## 2. Contour conditions

For the temperatures used to calculate the interior air, the values from STAS 1907 were taken into consideration, and the values from the Normative C 107/1-3:2005 were taken into consideration for the contour conditions and calculation values for the external air. The building is considered to be placed in turn in all four climatic regions of Romania.

*<u>1st climatic region</u>* - Mangalia:

- $\theta e = -12.0\ ^{o}C$;
- $\theta_a = 11.4\ ^{o}C$;
- $N_{12}^{20} = 2\,880$ K .days
- $D_{12} = 187$ days

For the solar radiation, the following $I_{Gj}$ values are adopted:

- S orientation $I_{G\,S} = 97.8\ W/m^2$
- SE or SW orientation $I_{G\,SE} = I_{G\,SV} = 79.8\ W/m^2$
- E or W orientation $I_{G\,E} = I_{G\,V} = 48.8\ W/m^2$

- NE or NW orientation $I_{G\,NE} = I_{G\,NV} = 25.7$ W/m$^2$
- N orientation $I_{G\,N} = 20.2$ W/m$^2$
- Horizontal surfaces $I_{G\,O} = 83.2$ W/m$^2$

*2nd climatic region* – Bucharest:
- $\theta e = -15.0$ °C;
- $\theta_a = 10.6$ °C;
- $N_{12}^{20} = 3170$ K .days
- $D_{12} = 190$ days

For the solar radiation, the following $I_{Gj}$ values are adopted:
- S orientation $I_{G\,S} = 92.5$ W/m$^2$
- SE or SW orientation $I_{G\,SE} = I_{G\,SV} = 76.0$ W/m$^2$
- E or W orientation $I_{G\,E} = I_{G\,V} = 47.4$ W/m$^2$
- NE or NW orientation $I_{G\,NE} = I_{G\,NV} = 25.7$ W/m$^2$
- N orientation $I_{G\,N} = 20.3$ W/m$^2$
- Horizontal surfaces $I_{G\,O} = 82.0$ W/m$^2$

*3rd climatic region* – Cluj – Napoca:
- $\theta e = -18.0$ °C;
- $\theta_a = 8.3$ °C;
- $N_{12}^{20} = 3\,730$ K .days
- $D_{12} = 218$ days

For the solar radiation, the following $I_{Gj}$ values are adopted:
- S orientation $I_{G\,S} = 88.2$ W/m$^2$
- SE or SW orientation $I_{G\,SE} = I_{G\,SV} = 74.2$ W/m$^2$
- E or W orientation $I_{G\,E} = I_{G\,V} = 48.5$ W/m$^2$
- NE or NW orientation $I_{G\,NE} = I_{G\,NV} = 27.7$ W/m$^2$
- N orientation $I_{G\,N} = 21.5$ W/m$^2$
- Horizontal surfaces $I_{G\,O} = 88.4$ W/m$^2$

*4th climatic region* – Târgu Secuiesc:
- $\theta e = -21.0$ °C;
- $\theta_a = 6.8$ °C;
- $N_{12}^{20} = 4\,370$ K .days
- $D_{12} = 237$ days

For the solar radiation, the following $I_{Gj}$ values are adopted:
- S orientation $I_{G\,S} = 94.9$ W/m$^2$
- SE or SW orientation $I_{G\,SE} = I_{G\,SV} = 79.9$ W/m$^2$
- E or W orientation $I_{G\,E} = I_{G\,V} = 52.5$ W/m$^2$
- NE or NW orientation $I_{G\,NE} = I_{G\,NV} = 30.6$ W/m$^2$
- N orientation $I_{G\,N} = 24.4$ W/m$^2$
- Horizontal surfaces $I_{G\,O} = 96.8$ W/m$^2$

The heat need was determined based on the temperature spatial field according to the stipulations of the Methodologies: Mc001, Mc002, Mc002:2006 and of SR EN ISO 13790:2005 “Calculation of the energy need for heating”.

## 3. The calculation and thermalenergetic analysis method of the building

Establishing the heat losses for each room and apartment was done based on the temperature spatial field and was done using the calculation program "NECESARC" derived from the calculation program "CIMPSPAT". In order to use the program, the whole building, together with the land on which it was built, included between the horizontal and vertical outline, was divided using section plans thus forming an calculation orthogonal matrix of the temperature spatial field with variable steps on the 3 cartesian axis. The calculation network is generated automatically by the calculation program (axis X: 182 axes, axis Y: 205 axes, axis Z: 176 axes, thus resulting a number of 6,566,560 of calculation nodes);

In order to write and solve the system of 6,566,560 linear algebraic equations, the numerical method used is the exact energetic equilibrium in each node of the meshing spatial matrix of the whole. The calculation program follows the stipulations of the Normative EN ISO 10211/1-95 on establishing the meshing and estimation matrix of the flux equilibrium in the nodes of the calculation matrix.

## 5. Conclusions

The energetic performance of the building *Block of flats* is superior to the *Reference Building* for all the four locations in the climatic regions of Romania.

The energetic classification of the building:

- For the climatic regions I – III: - B for the real building and C for the reference building;
- For the climatic region IV: - C for the real building and D for the reference building;

The energetic performance of the building is superior to the buildings under design and execution which are frequently energetically classified with C and only in certain cases with B.

The energetic performance of the building is much superior to the buildings built between 1950-1990 whose energetic consumptions are between 450-650 kWh/($m^2$*year), meaning E – F energetic classification, and for some buildings even G.

The energy needed to heat the building is under 100.0 kWh/($m^2$*year) if located in the $1^{st}$ and $2^{nd}$ climatic regions.

For the building located in the $1^{st}$ climatic region, the energy need to heat the building exceeds the limit to energetically classify with letter A 8.3 kWh/($m^2$*year), and for heating the apartments only with 5.3 kWh/($m^2$*year).

## APPENDIX

**CERTIFICATION OF THE BUILDING ENERGY PERFORMANCE**

Below you can find the energy performance certificate of the building located in the $1^{st}$ climatic region.

| Cod poștal localitate | Nr. înregistrare la Consiliul Local | Data înregistrării |
|---|---|---|
| | | |

**Certificat de performanță energetică**

| **Performanța energetică a clădirii** | Notare energetică: | **95,5** |
|---|---|---|
| Sistemul de certificare: *Metodologia de calcul al Performanței Energetice a Clădirilor elaborată în aplicarea Legii 372/2005* | Clădirea certificată | Clădirea de referință |
| Eficiență energetică ridicată | | |
| A | | |
| B | B | |
| C | | C |
| D | | |
| E | | |
| F | | |
| G | | |
| Eficiență energetică scăzută | | |
| Consum anual specific de energie [kWh/m²an] | 153,5 | 223,5 |
| Indice de emisii echivalent $CO_2$ [$kg_{CO_2}$/m²an] | 29,2 | 42,5 |

| Consum anual specific de energie [kWh/m²an] pentru: | | Clasă energetică: Clădirea certificată | Clasă energetică: Clădirea de referință |
|---|---|---|---|
| Încălzire: | 78,3 | B | B |
| Apă caldă de consum: | 48,3 | C | D |
| Climatizare: | - | - | - |
| Ventilare mecanică: | - | - | - |
| Iluminat artificial: | 26,9 | A | C |

Consum anual specific de energie din surse regenerabile [kWh/m²an]: 0

| Date privind clădirea certificată: | |
|---|---|
| Adresa clădirii: Mangalia | Aria utilă : 886,90 m² |
| Categoria clădirii: Locuinte | Aria construită desfășurată : 1289,29 m² |
| Regim de înălțime: Ds+P+2E+Mansarda | Volumul interior al clădirii : 2629,57 m³ |
| Anul construirii: 2009 | |
| Scopul elaborării certificatului energetic : *TEZA DE DOCTORAT* | |

Programul de calcul utilizat: „CIMPSPAT","AEC","NECESAR", „CERTENERG",versiunea: 2008

Date privind identificarea auditorului energetic pentru clădiri:

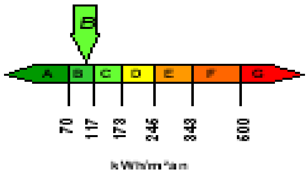
| Specialitatea ( C,I,CI ) | Numele și prenumele | Seria și Nr. certificat de atestare | Nr. și data înregistrării certificatului în registrul auditorului | Semnătura și ștampila auditorului |
|---|---|---|---|---|
| | | | | |

*Clasificarea energetică a clădirii este făcută funcție de consumul total de energie al clădirii, estimat prin analiză termică și energetică a construcției și instalațiilor aferente.*
*Notarea energetică a clădirii ține seama de penalizările datorate utilizării neraționale a energiei.*
*Perioada de valabilitate a prezentului Certificat Energetic este de 10 ani de la data eliberării acestuia*

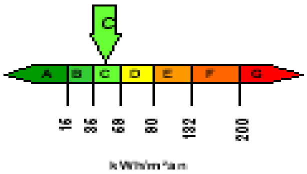
DATE PRIVIND EVALUAREA PERFORMANȚEI ENERGETICE A CLĂDIRII

- **Grile de clasificare energetică a clădirii funcție de consumul de căldură anual specific:**

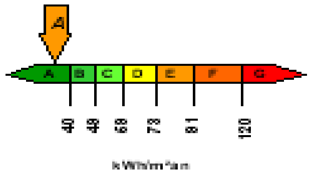
**ÎNCĂLZIRE:**



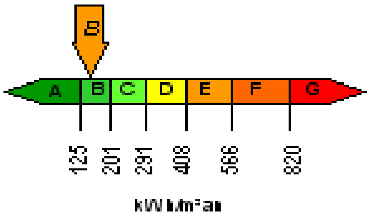

**APĂ CALDĂ DE CONSUM:**



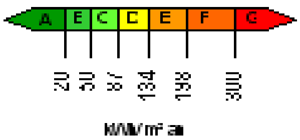

**ILUMINAT:**



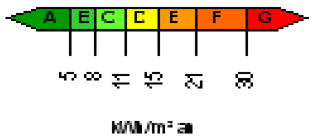

**TOTAL:ÎNCĂLZIRE, APĂ CALDĂ DE CONSUM, ILUMINAT**




**CLIMATIZARE:**




**VENTILARE MECANICĂ:**




- **Performanța energetică a clădirii de referință:**

| Consum anual specific de energie [kWh/m²an] | | | Notare energetică |
|---|---|---|---|
| pentru: | | | |
| Încălzire: | | 111,5 | **90,1** |
| Apă caldă de consum: | | 62,0 | |
| Climatizare: | | - | |
| Ventilare mecanică: | | - | |
| Iluminat: | | 50,0 | |

- **Penalizări acordate clădirii certificate și motivarea acestora:**

$P_0$ = 1,1 – după cum urmează :

- Subsol uscat, cu posibilitate de acces la instalația — $p_1$ = 1,00
- Ușile de intrare in clădire sunt prevăzute cu sistem automat de închidere — $p_2$ = 1,00
- Ferestre / uși în stare bună, etanșe — $p_3$ = 1,00
- Armăturile de reglaj ale corpurilor statice sunt funcționale — $p_4$ = 1,00
- Instalația de încălzire a fost spălată / curățată cu mai puțin de trei ani în urmă — $p_5$ = 1,00
- Coloanele de încălz. sunt prevăzute cu armături de separare și golire a acestora — $p_6$ = 1,00
- Tencuială exterioară in stare buna — $p_8$ = 1,00
- Pereții exteriori in stare buna și nu prezintă pete de condens — $p_9$ = 1,00
- Clădire fără sistem de ventilare organizată — $p_{12}$ = 1,10

- **Recomandări pentru îmbunătățirea performanței energetice a clădirii:**
  - Soluții recomandate pentru anvelopa clădirii : Reproiectarea detaliilor .
  - Soluții recomandate pentru instalațiile aferente clădirii: Nu este cazul.

*Clasificarea energetică a clădirii este făcută funcție de consumul total de energie al clădirii, estimat prin analiză termică și energetică a construcției și instalațiilor aferente.*
*Notarea energetică a clădirii ține seama de penalizările datorate utilizării neraționale a energiei.*
*Perioada de valabilitate a prezentului Certificat Energetic este de 10 ani de la data eliberării acestuia*

## Numerical results obtained per apartments and per whole building
## REAL BUILDING (Energy consumptions – 1st climatic region)

BLOC DE LOCUINTE, Localit. Mangalia ( Z.Clim. I ), Ds +P +2E +MANSARDA ( CLADIREA REALA )

| Incaperea Nivelul Cladirii | Pierderi prin cond., conv., rad. si Ventilatie (n=0.3/h conf. SR EN ISO 13790) | | | | | | | | | | | Pierderi totale | | | Aporturi de caldura | | | | Necesarul de caldura | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Pereti | | Supr. Vitrate | | Plafon | | Pardoseala | | Total Pierderi prin elem. anvelop.clad. | | Ventil. improsp aerului | Pierderi prin elem. anvelopei clad. si improspat.aerului | | | Interne | Radiatie | Aporturi Tot | | Total Anual Qnec | | Necesar caldura specific anual |
| | kWh | Qp/Qt % | kWh | Qv/Qt % | kWh | Qpl/Qt % | kWh | Qpa/Qt % | Qt kWh | % | kWh | kWh | Anv % | Ven % | kWh | kWh | kWh | % | kWh | % | kWh/m²an |
| Sc.1 Parter Apart. 1 | 1533,5 | 53,3 | 499,3 | 17,3 | --- | --- | 846,8 | 29,4 | 2879,6 | 3,3 | 381,8 | 3261,4 | 88,3 | 11,7 | 555,6 | 95,8 | 651,4 | 25,0 | 2610,0 | 4,0 | 98,3 |
| Sc.1 Parter Apart. 2 | 1303,1 | 22,0 | 2912,6 | 49,3 | --- | --- | 1695,6 | 28,7 | 5911,4 | 6,8 | 749,1 | 6660,5 | 88,8 | 11,2 | 1090,3 | 750,0 | 1840,4 | 38,2 | 4820,1 | 7,3 | 92,5 |
| Sc.1 Parter Apart. 3 | 1402,7 | 23,8 | 2850,2 | 48,4 | --- | --- | 1631,0 | 27,7 | 5883,9 | 6,8 | 708,9 | 6592,7 | 89,2 | 10,8 | 1031,8 | 1159,5 | 2191,3 | 49,8 | 4401,5 | 6,7 | 89,3 |
| Sc.1 Etaj. 1 Apart. 4 | 1361,6 | 31,9 | 2908,2 | 68,1 | --- | --- | --- | --- | 4269,8 | 5,0 | 749,1 | 5018,9 | 85,1 | 14,9 | 1090,3 | 1271,0 | 2361,4 | 88,9 | 2657,5 | 4,0 | 51,0 |
| Sc.1 Etaj. 1 Apart. 5 | 1273,4 | 30,8 | 2855,9 | 69,2 | --- | --- | --- | --- | 4129,2 | 4,8 | 708,9 | 4838,1 | 85,3 | 14,7 | 1031,8 | 1159,5 | 2191,3 | 82,8 | 2646,8 | 4,0 | 53,7 |
| Sc.1 Etaj. 2 Apart. 6 | 1130,3 | 23,9 | 2909,0 | 61,6 | 680,4 | 14,4 | --- | --- | 4719,7 | 5,5 | 749,1 | 5468,8 | 86,3 | 13,7 | 1090,3 | 1271,0 | 2361,4 | 76,0 | 3107,4 | 4,7 | 59,6 |
| Sc.1 Etaj. 2 Apart. 7 | 1014,2 | 22,1 | 2856,8 | 62,3 | 714,4 | 15,6 | --- | --- | 4585,4 | 5,3 | 708,9 | 5294,3 | 86,6 | 13,4 | 1031,8 | 1159,5 | 2191,3 | 70,6 | 3103,0 | 4,7 | 62,9 |
| Sc.1 Mansar. Apart. 8 | 1520,7 | 30,8 | 2330,6 | 47,1 | 1094,6 | 22,1 | --- | --- | 4945,9 | 5,7 | 744,8 | 5690,7 | 86,9 | 13,1 | 978,4 | 1140,0 | 2118,4 | 59,3 | 3572,3 | 5,4 | 76,4 |
| Sc.1 Mansar. Apart. 9 | 1183,2 | 29,0 | 1776,0 | 43,5 | 1120,8 | 27,5 | --- | --- | 4080,0 | 4,7 | 666,5 | 4746,5 | 86,0 | 14,0 | 1023,4 | 925,9 | 1949,3 | 69,7 | 2797,2 | 4,2 | 57,2 |
| Sc.2 Parter Apart.10 | 1435,3 | 25,8 | 2657,7 | 47,7 | --- | --- | 1474,2 | 26,5 | 5567,2 | 6,5 | 644,1 | 6211,3 | 89,6 | 10,4 | 937,6 | 280,3 | 1217,8 | 24,4 | 4993,5 | 7,6 | 111,5 |
| Sc.2 Parter Apart.11 | 1442,8 | 25,3 | 2566,0 | 44,9 | --- | --- | 1700,8 | 29,8 | 5709,7 | 6,6 | 749,1 | 6458,8 | 88,4 | 11,6 | 1090,3 | 1091,1 | 2181,5 | 51,0 | 4277,3 | 6,5 | 82,1 |
| Sc.2 Etaj. 1 Apart.12 | 1510,7 | 37,1 | 2560,3 | 62,9 | --- | --- | --- | --- | 4071,0 | 4,7 | 749,1 | 4820,2 | 84,5 | 15,5 | 1090,3 | 1088,2 | 2178,6 | 82,5 | 2641,6 | 4,0 | 50,7 |
| Sc.2 Etaj. 1 Apart.13 | 1315,5 | 33,0 | 2665,5 | 67,0 | --- | --- | --- | --- | 3981,0 | 4,6 | 644,1 | 4625,2 | 86,1 | 13,9 | 937,6 | 278,7 | 1216,2 | 35,7 | 3409,0 | 5,2 | 76,1 |
| Sc.2 Etaj. 1 Apart.14 | 1948,9 | 49,0 | 2024,9 | 51,0 | --- | --- | --- | --- | 3973,8 | 4,6 | 611,1 | 4584,8 | 86,7 | 13,3 | 889,4 | 378,9 | 1268,4 | 38,2 | 3316,5 | 5,0 | 78,0 |
| Sc.2 Etaj. 2 Apart.15 | 1263,4 | 29,4 | 2561,4 | 59,7 | 468,6 | 10,9 | --- | --- | 4293,4 | 5,0 | 749,1 | 5042,5 | 85,1 | 14,9 | 1090,3 | 1088,2 | 2178,6 | 76,1 | 2863,9 | 4,3 | 55,0 |
| Sc.2 Etaj. 2 Apart.16 | 1041,1 | 26,1 | 2665,6 | 66,9 | 279,8 | 7,0 | --- | --- | 3986,5 | 4,6 | 644,1 | 4630,7 | 86,1 | 13,9 | 937,6 | 278,7 | 1216,2 | 35,6 | 3414,4 | 5,2 | 76,2 |
| Sc.2 Etaj. 2 Apart.17 | 1236,0 | 33,8 | 2023,6 | 55,4 | 395,4 | 10,8 | --- | --- | 3655,0 | 4,2 | 611,1 | 4266,1 | 85,7 | 14,3 | 889,4 | 378,9 | 1268,4 | 42,3 | 2997,7 | 4,5 | 70,5 |
| Sc.2 Mansar. Apart.18 | 920,0 | 24,9 | 1845,1 | 49,9 | 932,8 | 25,2 | --- | --- | 3697,9 | 4,3 | 550,7 | 4248,6 | 87,0 | 13,0 | 845,5 | 870,4 | 1715,9 | 67,7 | 2532,7 | 3,8 | 62,7 |
| Sc.2 Mansar. Apart.19 | 1336,3 | 32,4 | 1763,2 | 42,7 | 1028,0 | 24,9 | --- | --- | 4127,4 | 4,8 | 771,2 | 4898,6 | 84,3 | 15,7 | 929,2 | 700,7 | 1629,9 | 49,9 | 3268,7 | 5,0 | 73,6 |
| Parter Windfang | --- | --- | 335,0 | 76,2 | --- | --- | 104,5 | 23,8 | 439,4 | 0,5 | 110,4 | 549,8 | 79,9 | 20,1 | --- | 121,5 | 121,5 | 28,4 | 428,3 | 0,6 | 55,8 |
| Casa Scarii | --- | --- | 1104,6 | 81,5 | 251,2 | 18,5 | --- | --- | 1355,8 | 1,6 | 2122,3 | 3478,1 | 39,0 | 61,0 | --- | 1343,7 | 1343,7 | 63,0 | 2134,4 | 3,2 | 17,4 |
| Total Apartamente | 25172,7 | 29,8 | 45231,8 | 53,5 | 6714,8 | 8,0 | 7348,4 | 8,7 | 84467,8 | 97,9 | 12890,7 | 97358,6 | 86,8 | 13,2 | 18561,0 | 15366,4 | 33927,4 | 53,5 | 63431,1 | 96,1 | 71,5 |
| TOTAL CLADIRE | 25172,7 | 29,2 | 46671,4 | 54,1 | 6966,0 | 8,1 | 7452,9 | 8,6 | 86263,0 | 100,0 | 15123,5 | 101386,5 | 85,1 | 14,9 | 18561,0 | 16831,6 | 35392,6 | 53,6 | 65993,9 | 100,0 | 74,4 |

**CALCULE EFECTUATE IN VARIANTA DE IZOLARE :**

| | |
|---|---|
| Pereti exteriori | : cofraj 2,5 Heraklit+8 cm Polist.extrudat |
| Acoperisul mansarda | : 20 cm Vata Minerala rez.la FOC |
| Acoperisul terasa | : 15 cm Polistiren extrudat |
| Plansee intermediare | : 5 cm Polistiren expandat |
| Rama supraf. vitrate | : Prof.din PVC-6 Cam. cu 3 Foi de Sticla |
| Vitraj 4+14+4+14+4 mm: | Gazul:Kripton+2 Stic. cu emisiv.<=0.20 |

**CLASIFICAREA SI NOTAREA ENERGETICA A CLADIRII**

| | | |
|---|---|---|
| Randamentul mediu anual al sistemului de incalzire | Rand.med.anual = 95.0 % | 95.8 |
| Consumul specific de en. anual ptr. încalzirea icaperilor | qinc = 75.3 [kWh/m²an] | |
| Consumul specific de en. anual ptr. încalzirea cladirii | qincl = 78.3 [kWh/m²an] | 95.5 |
| Consumul specific de en. anual ptr. prepar. apei calde de cons. | qacm = 48.3 [kWh/m²an] | |
| Consumul specific de en. anual ptr. iluminarea spatiilor | qil = 26.9 [kWh/m²an] | |
| Cons.specif.total de en.anual ptr. încalz., apa c.c., ilum. apart. | qT = 150.5 [kWh/m²an] | B |
| Cons.specif.total de en.anual ptr. încalz., apa c.c., ilum.cladire | qTc = 153.5 [kWh/m²an] | B |

## THE REFERENCE BUILDING (Energy consumptions – 1st climatic region)

BLOC DE LOCUINTE, Localit. Mangalia ( Z.Clim. I ), Ds +P +2E +MANSARDA ( CLADIREA DE REFERINTA )

| Incaperea Nivelul Cladirii | Pierderi prin cond., conv., rad. si Ventilatie (n=0.5/h cf. Metod. I Cap.I. p. 9.7) | | | | | | | | | | | Pierderi totale | | | Aporturi de caldura | | | | Necesarul de caldura | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Pereti | | Supr. Vitrate | | Plafon | | Pardoseala | | Total Pierderi prin elem. anvelop.clad. | | Ventil. improsp aerului | Pierderi prin elem. anvelopei clad. si improspat.aerului | | | Interne | Radiatie | Aporturi Tot | | Total Anual Qnec | | Necesar caldura specific anual |
| | kWh | Qp/Qt % | kWh | Qv/Qt % | kWh | Qpl/Qt % | kWh | Qpa/Qt % | Qt kWh | % | kWh | kWh | Anv % | Ven % | kWh | kWh | kWh | % | kWh | % | kWh/m²·an |
| Sc.1 Parter Apart. 1 | 1882,2 | 57,9 | 540,0 | 16,6 | --- | --- | 826,8 | 25,4 | 3249,0 | 3,3 | 636,3 | 3885,2 | 83,6 | 16,4 | 555,6 | 95,8 | 651,4 | 20,1 | 3233,8 | 3,6 | 121,8 |
| Sc.1 Parter Apart. 2 | 1944,6 | 28,3 | 3213,4 | 46,8 | --- | --- | 1706,7 | 24,9 | 6864,7 | 6,9 | 1248,5 | 8113,2 | 84,6 | 15,4 | 1090,3 | 750,0 | 1840,4 | 29,3 | 6272,8 | 7,0 | 120,4 |
| Sc.1 Parter Apart. 3 | 1901,1 | 29,1 | 3007,8 | 46,0 | --- | --- | 1634,9 | 25,0 | 6543,7 | 6,6 | 1181,4 | 7725,2 | 84,7 | 15,3 | 1031,8 | 1159,5 | 2191,3 | 39,6 | 5533,9 | 6,2 | 112,2 |
| Sc.1 Etaj. 1 Apart. 4 | 2011,6 | 38,5 | 3208,9 | 61,5 | --- | --- | --- | --- | 5220,5 | 5,3 | 1248,5 | 6469,0 | 80,7 | 19,3 | 1090,3 | 1271,0 | 2361,4 | 57,5 | 4107,7 | 4,6 | 78,8 |
| Sc.1 Etaj. 1 Apart. 5 | 1776,6 | 37,1 | 3013,5 | 62,9 | --- | --- | --- | --- | 4790,2 | 4,8 | 1181,4 | 5971,6 | 80,2 | 19,8 | 1031,8 | 1159,5 | 2191,3 | 58,0 | 3780,3 | 4,2 | 76,7 |
| Sc.1 Etaj. 2 Apart. 6 | 1725,3 | 30,5 | 3209,8 | 56,7 | 727,2 | 12,8 | --- | --- | 5662,4 | 5,7 | 1248,5 | 6910,9 | 81,9 | 18,1 | 1090,3 | 1271,0 | 2361,4 | 51,9 | 4549,5 | 5,1 | 87,3 |
| Sc.1 Etaj. 2 Apart. 7 | 1487,7 | 28,3 | 3015,0 | 57,3 | 757,9 | 14,4 | --- | --- | 5260,6 | 5,3 | 1181,4 | 6442,0 | 81,7 | 18,3 | 1031,8 | 1159,5 | 2191,3 | 51,5 | 4250,8 | 4,8 | 86,2 |
| Sc.1 Mansar. Apart. 8 | 2161,9 | 38,7 | 2280,1 | 40,8 | 1145,0 | 20,5 | --- | --- | 5587,0 | 5,6 | 1241,3 | 6828,3 | 81,8 | 18,2 | 978,4 | 1140,0 | 2118,4 | 45,0 | 4709,9 | 5,3 | 100,7 |
| Sc.1 Mansar. Apart. 9 | 1638,3 | 36,0 | 1726,0 | 38,0 | 1181,6 | 26,0 | --- | --- | 4546,0 | 4,6 | 1110,9 | 5656,9 | 80,4 | 19,6 | 1023,4 | 925,9 | 1949,3 | 52,6 | 3707,6 | 4,2 | 75,8 |
| Sc.2 Parter Apart.10 | 1882,3 | 30,8 | 2756,0 | 45,1 | --- | --- | 1472,3 | 24,1 | 6110,6 | 6,2 | 1073,6 | 7184,2 | 85,1 | 14,9 | 937,6 | 280,3 | 1217,8 | 20,4 | 5966,4 | 6,7 | 133,2 |
| Sc.2 Parter Apart.11 | 2153,0 | 32,0 | 2866,2 | 42,6 | --- | --- | 1712,2 | 25,4 | 6731,4 | 6,8 | 1248,5 | 7979,9 | 84,4 | 15,6 | 1090,3 | 1091,1 | 2181,5 | 37,6 | 5798,5 | 6,5 | 111,3 |
| Sc.2 Etaj. 1 Apart.12 | 2230,6 | 43,8 | 2859,3 | 56,2 | --- | --- | --- | --- | 5089,9 | 5,1 | 1248,5 | 6338,5 | 80,3 | 19,7 | 1090,3 | 1088,2 | 2178,6 | 52,4 | 4159,9 | 4,7 | 79,8 |
| Sc.2 Etaj. 1 Apart.13 | 1773,7 | 39,1 | 2764,5 | 60,9 | --- | --- | --- | --- | 4538,2 | 4,6 | 1073,6 | 5611,8 | 80,9 | 19,1 | 937,6 | 278,7 | 1216,2 | 27,7 | 4395,6 | 4,9 | 98,1 |
| Sc.2 Etaj. 1 Apart.14 | 2268,4 | 51,4 | 2148,5 | 48,6 | --- | --- | --- | --- | 4416,9 | 4,5 | 1018,5 | 5435,3 | 81,3 | 18,7 | 889,4 | 378,9 | 1268,4 | 30,4 | 4166,9 | 4,7 | 98,0 |
| Sc.2 Etaj. 2 Apart.15 | 1927,6 | 36,3 | 2860,5 | 53,9 | 522,3 | 9,8 | --- | --- | 5310,3 | 5,3 | 1248,5 | 6558,8 | 81,0 | 19,0 | 1090,3 | 1088,2 | 2178,6 | 49,7 | 4380,2 | 4,9 | 84,1 |
| Sc.2 Etaj. 2 Apart.16 | 1480,8 | 32,5 | 2764,6 | 60,7 | 307,6 | 6,8 | --- | --- | 4553,0 | 4,6 | 1073,6 | 5626,5 | 80,9 | 19,1 | 937,6 | 278,7 | 1216,2 | 27,6 | 4410,3 | 5,0 | 98,4 |
| Sc.2 Etaj. 2 Apart.17 | 1527,8 | 37,2 | 2148,5 | 52,3 | 434,0 | 10,6 | --- | --- | 4110,3 | 4,1 | 1018,5 | 5128,7 | 80,1 | 19,9 | 889,4 | 378,9 | 1268,4 | 32,9 | 3860,4 | 4,3 | 90,8 |
| Sc.2 Mansar. Apart.18 | 1337,0 | 32,7 | 1772,8 | 43,3 | 983,2 | 24,0 | --- | --- | 4093,0 | 4,1 | 917,8 | 5010,8 | 81,7 | 18,3 | 845,5 | 870,4 | 1715,9 | 52,1 | 3294,9 | 3,7 | 81,6 |
| Sc.2 Mansar. Apart.19 | 1946,5 | 40,7 | 1769,7 | 37,0 | 1071,6 | 22,4 | --- | --- | 4787,8 | 4,8 | 1285,3 | 6073,1 | 78,8 | 21,2 | 929,2 | 700,7 | 1629,9 | 36,7 | 4443,2 | 5,0 | 100,1 |
| Parter Windfang | --- | --- | 348,5 | 75,6 | --- | --- | 112,3 | 24,4 | 460,9 | 0,5 | 184,0 | 644,9 | 71,5 | 28,5 | --- | 121,5 | 121,5 | 23,2 | 523,4 | 0,6 | 68,2 |
| Casa Scarii | --- | --- | 1029,9 | 80,5 | 249,2 | 19,5 | --- | --- | 1279,1 | 1,3 | 3537,1 | 4816,2 | 26,6 | 73,4 | --- | 1343,7 | 1343,7 | 38,7 | 3472,6 | 3,9 | 28,3 |
| Total Apartamente | 35056,9 | 36,0 | 47925,2 | 49,2 | 7130,4 | 7,3 | 7352,9 | 7,5 | 97465,4 | 98,3 | 21484,6 | 118950,0 | 81,9 | 18,1 | 18561,0 | 15366,4 | 33927,4 | 39,9 | 85022,5 | 95,5 | 95,9 |
| TOTAL CLADIRE | 35056,9 | 35,3 | 49303,6 | 49,7 | 7379,6 | 7,4 | 7465,2 | 7,5 | 99205,3 | 100,0 | 25205,8 | 124411,1 | 79,7 | 20,3 | 18561,0 | 16831,6 | 35392,6 | 39,8 | 89018,5 | 100,0 | 100,4 |

VARIANTA DE IZOLARE NORMATA:

| | | |
|---|---|---|
| Pereti exteriori | : 1,50 | [m²/kW] |
| Acoperisul mansarda | : 3,50 | [m²/kW] |
| Acoperisul terasa | : 3,50 | [m²/kW] |
| Planseul peste subsol | : 1,65 | [m²/kW] |
| Suprafete vitrate | : 0,55 | [m²/kW] |

CLASIFICAREA SI NOTAREA ENERGETICA A CLADIRII

| | | |
|---|---|---|
| Randamentul mediu anual al sistemului de incalzire | Rand.med.anual = 90.0 % | 90.6 |
| Consumul specific de en. anual ptr. încalzirea icaperilor | qinc = 106.5 [kWh/m²an] | |
| Consumul specific de en. anual ptr. încalzirea cladirii | qincl = 111.5 [kWh/m²an] | 90.1 |
| Consumul specific de en. anual ptr. prepar. apei calde de cons. | qacm = 62.0 [kWh/m²an] | |
| Consumul specific de en. anual ptr. iluminarea spatiilor | qil = 50.0 [kWh/m²an] | |
| Cons.specif.total de en.anual ptr. încalz., apa c.c., ilum. apart. | qT = 218.5 [kWh/m²an] | C |
| Cons.specif.total de en.anual ptr. încalz., apa c.c., ilum.cladire | qTc = 223.5 [kWh/m²an] | C |

## 6. References


[1] Moga, I. - Programele de calcul CIMPSPAT si NECESARC

[2] Fletcher, R. - Practical methods of optimization, Ed. John Wiley and Sons, New York, 1972

[3] Ancau, M., Liviu, N. - Tehnici numerice de optimizare in proiectarea asistata de calculator. Ed.Tehnica, Bucuresti, 1996

[4] Ghid privind optimizarea nivelului de protectie termica la cladirile de locuit GP 058-2000

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Considerations on Building Degradations due to Excessive Humidity

Mihaela Costin[*1], Mayer Zsombor[2]

[1,2] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*Numerous buildings in our country show degradations due to humidity causing esthetic, higrothermic, biological and also structural dysfunctionalities. The most affected ones are the old inhabited buildings, but also some historical buildings, architectural monuments.*
*The most important cause for such degradations is the ascending capillary humidity due to lack of horizontal and/or vertical hydro-insulation, but also the use of highly porous building materials, lack of maintenance for rainwater collecting and drainage systems. That is why it is very important to find the most adequate solutions scientifically demonstrated for draining the humidity of the buildings affected by this calamity.*

## Rezumat

*Numeroase construcții din țara noastră prezintă degradări cauzate de umiditate, implicând disfuncționalități de natură estetică, higrotermică, biologică dar şi de natură structurală. Cele mai afectate sunt clădirile de locuit vechi dar şi unele clădiri istorice, monumente de arhitectură.*
*Cea mai importantă cauză a acestor degradări este umiditătea capilar- ascedentă datorată lipsei de hidroizolații orizontale şi/ sau verticale, dar şi utilizarea materialelor de construcții foarte poroase, lipsa întreținerii sistemelor de colectare şi drenare a apelor pluviale. Iată de ce se impune cu stringență găsirea celor mai adecvate soluții fundamentale ştiințific pentru asanarea umidității clădirilor afectate de acest flagel.*

**Keywords:** Degradations, humidity, buildings, hydro-insulations

## 1. Introduction

During the last decades, so-called solutions and procedures were offered to keep under control the humidity and the damages caused by it. There are "procedures" which worsened the situation once applied, causing even bigger deteriorations. Out of these "procedures" we mention latex-based painting, bitumen painting, as well synthetic resin plastering which are leading to deficiencies in the water vapor diffusion capacity in case of an increased humidity of the base material followed by the appearance of blisters and exfoliations. Another example is the lime plaster which although has a very high capacity of water vapor diffusion, it cannot take over the salts transported from the soil and the result is damping and salt efflorescence on the surfaces.

Another inadequate solution is the cement-based plaster with the purpose of "sealing in" the humidity, a utopic procedure because this humidity is going up through capillars until evaporation is possible again.

[*] Corresponding author: E-mail address: mihaela.costin@cif.utcluj.ro

The consequences of these solutions can be seen on buildings whose foundation base was newly plastered and where, after a while, there are problems with damping and salt efflorescence. Even waterproof corrugated sheets are part of these inadequate procedures. Their goal was to reduce the actual conditions of humidity in the wall: the undulations of the air channels should have led to a dry wall. But due to additional waterproofing protection coatings, which prevented the disposal of water and drying of masonry, the result was a significant increase of damping in the upper areas and an upward movement of the damping level.

The procedures mentioned above describe the measures by which the humidity was sealed in masonry, but there are also wrong procedures through which one wants to remove the underground moisture. One of these ventilation methods would be the clay ventilation tubes which are placed at an angle in the masonry, having the role of reducing the degree of damping and ensuring quick drying.

## 2. The consequences of an inappropriate hydro-insulation

The hydro-insulation systems (design and execution) chosen wrong, non-performant hydro-insulating materials, failure in observing the technology etc. are causing major degradations and damages in construction. Some of these mistakes are pictured below. Inappropriate hydro-insulations are causing deterioration of the electric wiring [3], deterioration of the false ceiling, deterioration of the plaster and paining, as well reducing the thermal insulation capacity of building (picture1).

Picture1. Deterioration of painting [3]

Salts and oxides are causing stains on the exterior plates and may cause even the fall of the plates (picture 2) thus allowing the rain water to infiltrate directly [3].

Picture 2. Degradation of exterior plates [3]

Other consequences of rainwater leakage and inappropriate rainwater drainage is the presence of parasitic vegetation as well the degradation of warm flooring, furniture etc. [2], [3] (Picture 3).

Picture 3. Degradations due to rainwater and salts [3]

Another effect of inappropriate or lack of hydro-insulation is the dampness and mold. The mold, as any other fungus, grows and spreads where conditions are favorable, that is increased humidity and reduced air ventilation (picture 5). In case of side dampness, the cause of the damage is found alongside the wall, that is why the dampness and the salts are found mainly above the flooring and less towards the ceiling (picture 4). [4]

Picture 4. Degradations due to dampness

Picture 5. Degradations due to mold

## 3. Causes and solutions to eliminate the effects of humidity in construction (leakages, dampness, condensation or mold)

In order to eliminate the unpleasant effects of the humidity (dampness, condensation, mold), some basic rules need to be followed [5]:

- Finding the causes of such problems and not only treating the effects;
- Elimination of water leakage and relatively high humidity in the rooms.

The solutions for fixing such dysfunctionalities need to be scientifically fundamented, validated by practice and done by specialists with expertise in this field.

Some of the most frequent degradations of the construction elements in the buildings affected by overhumidity, possible causes and fixing methods are schematically presented in the table below.

[6]

| PROBLEM | POSSIBLE CAUSE | POSSIBLE SOLUTIONS | Observations |
|---|---|---|---|
| **Stains on the ceiling** | - water leaks through the roof<br>- condensation due to areas which are not thermal insulated<br>- thermal bridges | - fixing the roof, the leaks<br>- roof ventilation<br>- thermal insulation of the thermal bridges | |
| **Mold on the walls (during the cold season)** | - increased interior humidity | - find and remove the humidity source<br>- vapor barrier in cold areas<br>- thermal insulation of the thermal bridges<br>- use of inorganic interior finishes | - mold is a dangerous fungus for the health of humans and is omnipresent in our living environment |
| **Water in the basement** | - mechanical deficiencies in walls / flooring<br>- failure in / lack of hydro-insulation<br>- water leaks from pipes<br>- condensation due to lack of ventilation / poor hydro-insulation<br>- higher level of phreatic bed | - repairs to the sheeting, gutters and eaves or drainage<br>- repair the pipes<br>- repair the hydro-insulation<br>- draining the water to the exterior of the building | |
| **Efflorescence on the masonry / concrete** | - humidity moves through the building materials | - reduce / eliminate the water leaks or humidity source | - the worse mistake in such situations is removing the old plastering and applying a new one<br>- the dissolved salts which entered the wall structure do not disappear, they will keep on surfacing on the walls continuing their destructive activity |
| **Exterior painting peeling** | - humidity transportation to the exterior | - reduce the interior humidity level<br>- install a vapor barrier<br>- eliminate the humidity under the exterior finish by a good ventilation | |

## 5. Conclusions

The degradation of building elements of old buildings, some of them historical, due to overhumidity combined with various physical and chemical actions comes more and more to the attention of the specialists. Due to failure in observing the execution technologies, bad design, chosen of materials as well some inappropriate rehabilitation methods for these buildings, supplementary degradations occur. In order to fix the problems caused by overhumidity in construction, we need to find first the cause of damping; the cause needs to be removed and not covered.

## 6. References

[1] Frank Frössel, "Drying the masonry and basement sanitation ",

[2] Dorin Victor Popa "Contributions to the theory and practice of foundation and wall sanitation for historical and social-cultural buildings", PhD thesis, Technical University of Cluj Napoca, 2006

[3] http://www.hidroizolatii.afacereamea.ro/radcon7-performante-deosebite.php

[4] http://proaditiv.ro/produse.php?cat=5&prod=24

[5] http://hidroizolatii.afacereamea.ro/solutii-condens-igrasie-mucegai.php

[6] http://hidroizolatii.afacereamea.ro/hidroizolatii_autodepanare.php

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Numerical Solving of the Thermal Singularity of the Ground-Building Connection in Permanent Thermal Regime

Ousseynou Diao[*1], Ioan Moga[2], Ligia M. Moga[*3]

[1] *École Supérieur Polytechnique, Université Cheikh Anta DIOP de DAKAR-SÉNÉGAL, 5085 dakar-fann, Dakar, Senegal*
[2,3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*A 2D numerical model for calculating the heat transfer through the ground-building connection under permanent thermal regime is proposed in the following paper. The method takes into account geometric and thermal parameters that have a great influence on the treated structure. The method was applied to a semi-underground building and an on-ground building. The results show that semi-underground building loses more heat than the on-ground building.*

## Rezumat

*In lucrarea de față se prezintă un model de calcul bi-dimensional (2D) pentru calculul transferului de căldură prin legătura creată intre clădire- teren în regim termic permanent. Metoda prezentată ia în considerare parametrii geometrici şi termici care au o influență importantă asupra clădirii considerate. Metoda a fost aplicată la o clădire semi-îngropată şi la o clădire dispusă la cota terenului sistematizat. Rezultatele obținute demonstrează faptul că o clădire semi-îngropată pierde o cantitate mai mare de căldură dispusă direct pe teren.*

**Keywords:** thermal transfer, numerical model, semi-underground building, on-ground building, linear thermal transfer, MatLab.

## 1. Introduction

To solve the problem of thermal comfort in buildings, it is necessary to control the flow through the building envelope. The work in this paper focuses on the heat transfer at thermal singularities. The latter is an area where the assumption of unidimensionality is not applicable, as would be the ground-building connection. The paper proposes a numerical model for the calculation of the heat flow through the ground-building connection. The numerical model was applied at a semi-underground building and an on-ground building.

## 2. The Ground-Building Connection

Although many buildings are in direct contact with the ground, few studies have been devoted to developing the heat transfer phenomenon through the ground-building connection. This is

[*] Corresponding author: Tel./ Fax.: 0264401529
E-mail address: ligia.moga@cif.utcluj.ro

especially true in dynamic regime and it is surprising that the authors of the software sometimes very sophisticated and also designed to simulate the thermal behavior of buildings, seem to ignore this problem or reject by considering that the treated spaces are all highly ventilated crawl space.
Thermal problems that arise from the ground-building connections are very complex because of the 3D character of the heat transfer at this level, the thermal inertia of the ground which is very high and immensurable with other elements of the building envelope and soil heterogeneity, the presence of moisture that makes it very difficult to select the appropriate thermophysical characteristics [1].
The thermal behavior of buildings in contact with the ground is a topical issue that interests many researchers. Indeed, the air conditioning or heating loads control requires a thorough understanding of this behavior. Numerical simulation of thermal behavior of these configurations has known for several decades considerable progress in the wake of rapid development of computing resources. One of the most effective method is the 2D transfer functions with substructuring. Indeed, this method based on Seem's algorithm (1980) has been successfully applied to semi-buried buildings. Validation of this procedure was confirmed by comparing their results with those obtained by the alternating direction implicit method [2].
Despite the complexity of the problem, there are researchers who are interested in making an experimental study as Houghten, Taimuty, Gutberlet and Brown [3]; Boileau and Latta who used the method of circular arcs [4] Shipp and Broderick compare this method with a digital one [5].

## 3. Modeling

In the following figure when using the thermal equilibrium, the temperature of the central node T (i, j) is given by the equation (1) where $\Lambda_{q\text{-}g}$ is the thermal permeability between points q and g.

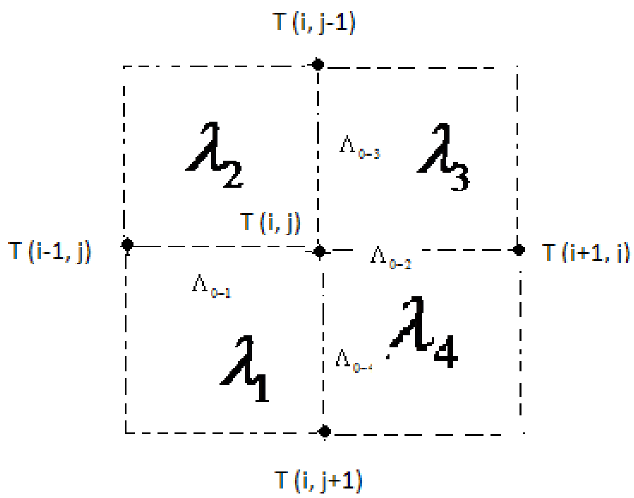



Figure 1. Discretization of a normal node

$$T(i,j)=\frac{\Lambda_{0-1}}{\sum_{1}^{k}\Lambda_{0-k}}T(i-1,j)+\frac{\Lambda_{0-2}}{\sum_{1}^{k}\Lambda_{0-k}}T(i+1,j)+\frac{\Lambda_{0-3}}{\sum_{1}^{k}\Lambda_{0-k}}T(i,j-1)+\frac{\Lambda_{0-4}}{\sum_{1}^{k}\Lambda_{0-k}}T(i,j+1) \tag{1}$$

## 4. Connection between an On-Ground Building and the Ground

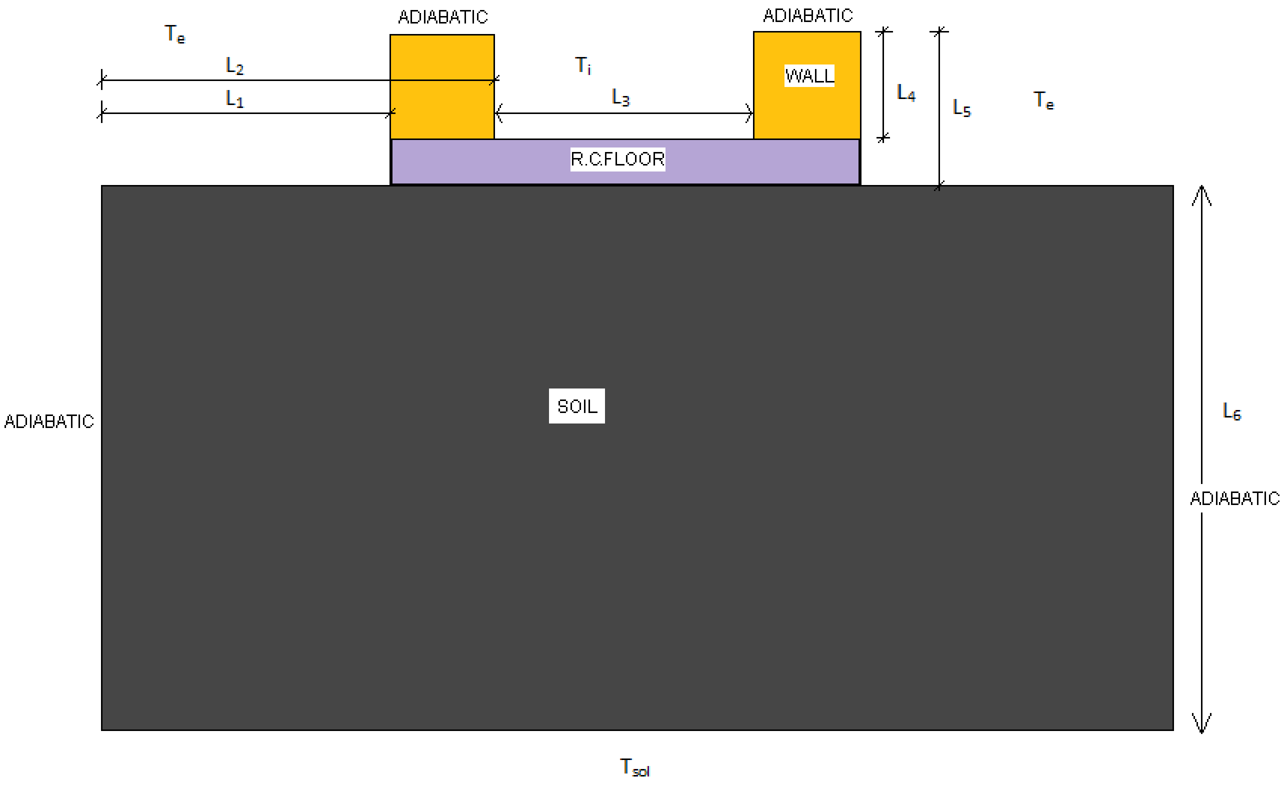



Figure 2. Geometrical model of the on-ground building

The connection between the building and the ground was simulated by taking into consideration the following data:

Table 1**:** Data for the on-ground building

| | |
|---|---|
| $L_1$ | 1,8 m |
| $L_2$ | 2,2 m |
| $L_3$ | 3 m |
| $L_4$ | 0.4 m |
| $L_5$ | 0.8 m |
| $L_6$ | 6.6 m |
| Exterior temperature $T_e$ | 30 °C |
| Interior temperature $T_i$ | 20 °C |
| Soil temperature $T_{sol}$ | 10 °C |
| Medium thermal conductivity of the soil | 2 W/(m·K) |
| Medium thermal conductivity of the floor | 2 W/(m·K) |
| Medium thermal conductivity of the wall | 0.3 W/(m·K) |

The thermal behavior of the reinforced-concrete floor disposed on the ground is very different from that of the building walls.

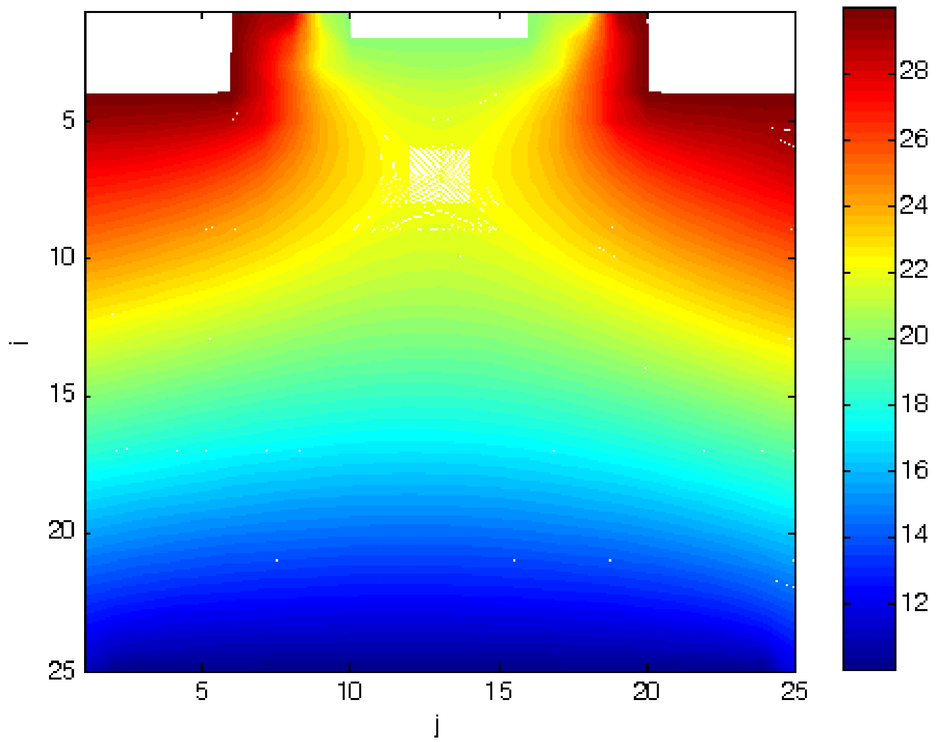



Figure 3. Isothermal lines- Thermal simulation of the connection between the on-ground building and the soil

## 5. Connection Between an Semi-Underground Building and the Ground

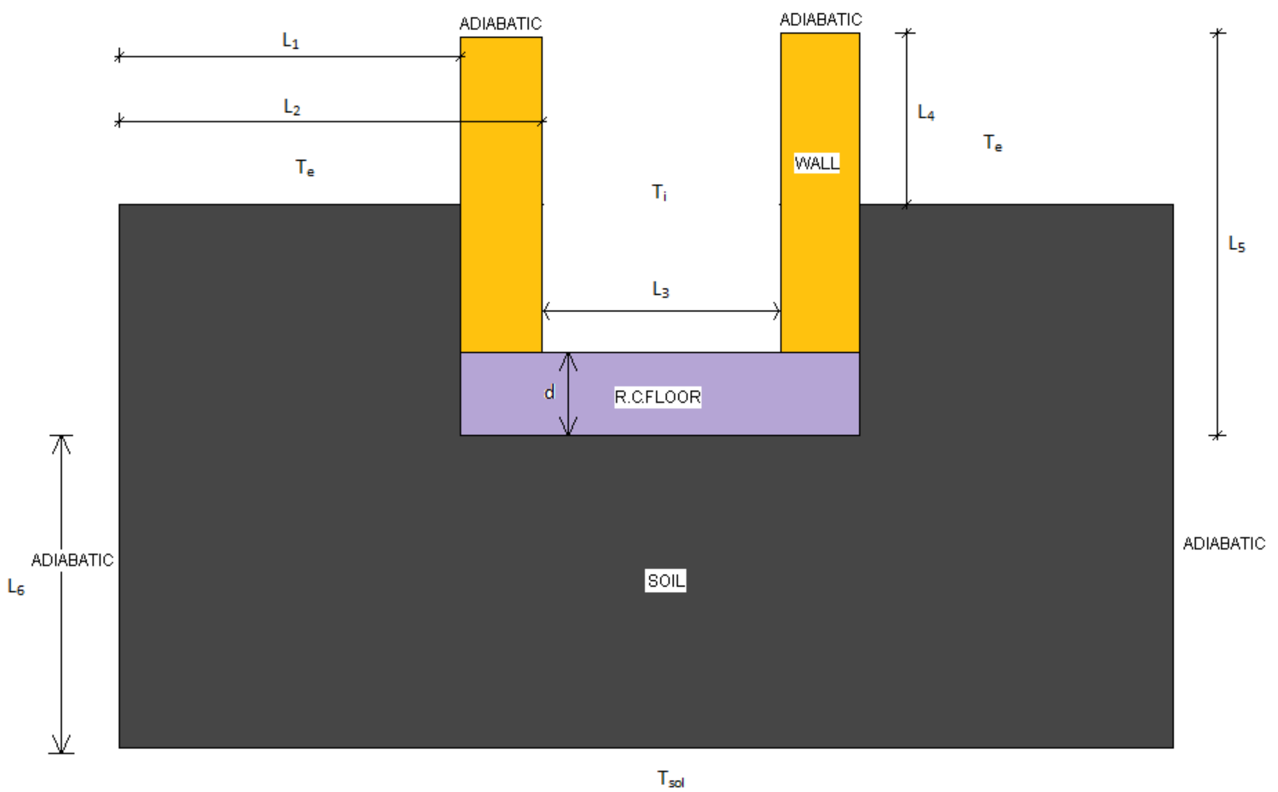



Figure 4. Geometrical model of the semi-underground building

The connection between the semi-underground building and the ground was simulated by taking into consideration the following data:

Table 2**:** Data for the semi-underground building

| | |
|---|---|
| $L_1$ | 1,8 m |
| $L_2$ | 2,2 m |
| $L_3$ | 3 m |
| $L_4$ | 0.4 m |
| $L_5$ | 1 m |
| $L_6$ | 6 m |
| Exterior temperature $T_e$ | 30 °C |
| Interior temperature $T_i$ | 20 °C |
| Soil temperature $T_{sol}$ | 10 °C |
| d | 0.4 m |
| Medium thermal conductivity of the soil | 2 W/(m·K) |
| Medium thermal conductivity of the floor | 2 W/(m·K) |
| Medium thermal conductivity of the wall | 0.3 W/(m·K) |

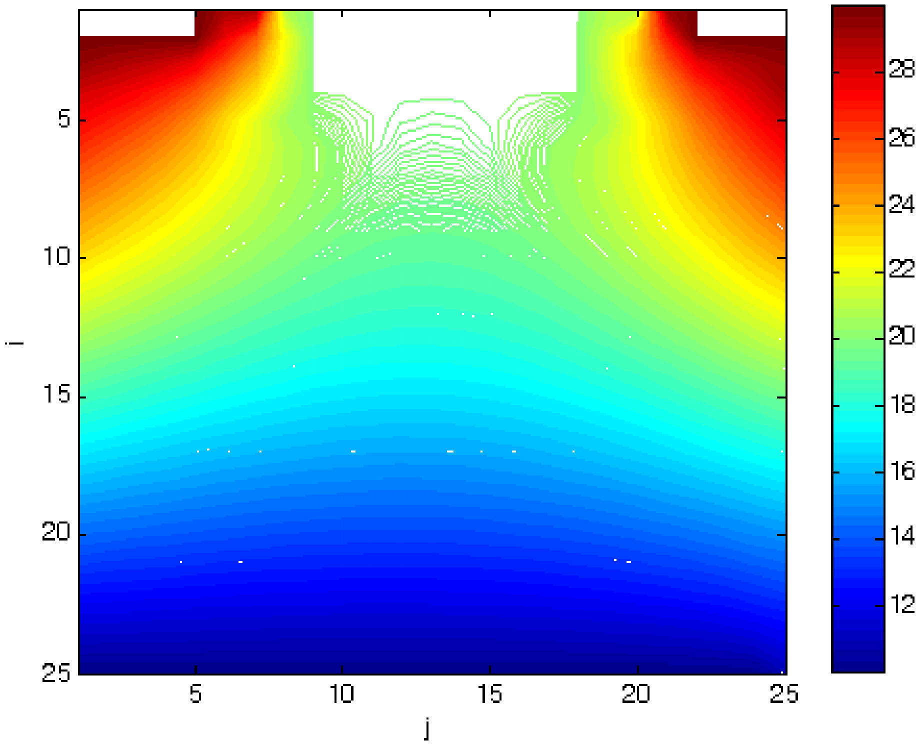



Figure 5. Isothermal lines- Thermal simulation of the connection between the semi-underground building and the soil

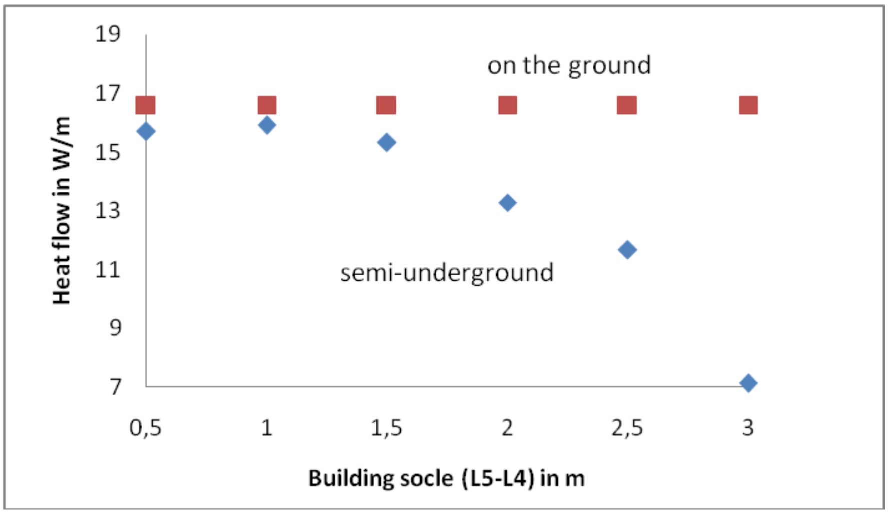



Figure 6. Comparison of linear heat flow between the two systems

We note that the semi-underground building reduces heat loss through the ground-building connection.

## 6. Conclusions

The obtained results show that the linear thermal power (strength) of a semi-underground building is lower than that of an on-ground building because the semi-buried building is closer to the cold source ($T_{sol}$). For the semi-underground building there is not much difference on the linear thermal power up to 1m of the building socle. In climatic conditions such as Senegal's, a semi-underground building reduces the energy bills. This study shows that specialists in the field must not neglect the thermal losses through these singularities in the evaluation of the energy balance, mostly when they are interested in the temperature of the interior surface of the walls.

## 6. References


[1] Achard (G)-Berou(R)-Yezou (R). *Etude des transferts thermiques entre un bâtiment et le sol environnant*, INSA Lyon ATP/PIRSEM.

[2] Amjad S., Abdelbaki A. and Zrikem Z. Transfer functions method and sub-structuration technique for two-dimensional heat conduction problems in high thermal mass systems. Application to ground coupling problems. *Energy and Building Journal*, **N° 35**, pp. 593-604, 2003.

[3] Houghten F.C, Taimuty S.K, Gutberlet C. and Brown C.J. Heat loss through basement walls and floors. *Heating, Piping and Air-Conditiooning*, **Vol.14**, pp.69-74,

[4] Boileau G. G.and Latta J.K. Calculation of basement heat loss. *Technical paper **No.292** of the Div. of Building Research,* NRCC 10477, National Research Council Canada, Ottawa K1A OR6,19 pp, 1968.

[5] Shipp H.P. and Broderick T.B. Comparison of annual heating loads for various basement wall insulation strategies using transient and steady-state models. *ASTM conference.* Owens-Corning Fiberglas Corporation, Technical Center, Granville, Ohio, 1980.

[6] Moga I., *Contributions to hygrothermal optimization of buildings in Cluj-Napoca area*, PhD Thesis, Institutul Politehnic Iaşi, 1987.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# The Fire Spread Outside of a Building

Ruxandra DARMON*1

*Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*The article is a review of the research about the fire spread outside of a building, the shape of the venting plume which expand out of a window, the mechanisms of fire spread upward on a building and to an adjacent building and some of the existing fire design regulations to prevent the fire spread. As conclusions, are given some directions for future research in this domain.*

## Rezumat

*Articolul este o trecere în revistă a studiilor şi cercetărilor despre răspândirea flăcărilor în exteriorul clădirilor, forma flăcărilor ce ies prin deschiderile de fereastă, mecanismele de răspândire a incendiilor pe fațade sau la clădiri învecinate, precum şi câteva dintre prevederile normativelor de proiectare pentru prevenirea extinderii incendiilor. În final, sunt date câteva direcții pentru cercetările viitoare în acest domeniu.*

**Keywords:** exterior flame, flame spread, window openings.

## 1. Introduction

There are existing various studies and experimental data about the compartment fires. Were carried out extensive analysis about the stages of a fire inside an enclosure, the fire spread mechanisms inside a building, the fire hazard evaluation methods, the ways of escape for users, assessment of damages cause by a compartment fire etc. The research is quite limited about the fire spread outside of the burning room.

The first studies about exterior venting fire plume, were performed by Yokoy [1] in 1960 to assess the risks associated with the fire spread from window openings in buildings. He described for the first time the venting plume trajectory depending on the temperatures and flame velocity distribution along the plume axis. By setting the critical temperature when standard glass fails at approximate 500 °C, Yokoi found a way to estimate the necessary length of the spandrel between window openings, to prevent the ignition of an upper level from the flames emerging from lower levels.

Webster [2] determined the high of the flames emerging from a window opening, performing fire tests on natural scale considering cubic rooms with openings on one side. The results, correlated with a non-dimensional analysis, considering the flame temperature around 1000 °F (538 °C), were in accord with the research results obtained by Yokoi.

To determine the flame size during fires inside the unprotected steel structures, Seigel [3] consideratedthe plume as a horizontal jet emerging from window openings The correlations stated

* Corresponding author: Tel./ Fax.: 0264 594967
E-mail address: ruxandra.darmon@cif.utcluj.ro

by Seigel for flame length can be used when fuel load and burning rates of materials are known. He also assumed a flame tip temperature of 538 °C, because flame temperatures below this value do not pose a significant risk to exposed steel structures.

Thomas and Law [4] determined the maximum flame high over the window openings considering the effect of supplying air to the room, which enhances the burning rate and the effect of the window geometry on the flame trajectory, measuring directly the radiative heat transfer. Law drafted a Design Guide [5] which details the effect of the venting plume on the steel stuctures exterior. The issue of the guide was succeeded by the elaboration of a Handbook written out by Law and O'Brian [6], which explained and simplified the information previously presented by Law.

Oleskiewicz [7] performed researced about the radiative heat flow and total heat flow emerging from window openings, modifying Law's [5] original plume shape and changing the conservative assumption of the flame with a constant thickness with a conical flame.

Natural scale tests conducted aimed to develop mechanisms of fire spread outside of buildings, the building fire response, the fire behaviour materials and also, the fire effect on facade cladding systems.

## 2. Mechanisms of fire spread outside of the building

A fire which reached the fully developed stage inside a room, can spread outside of the building to the upper floors of to the ajacent buildings through several fire spread mechanisms.

A first fire spread mechanism is leap-frogging, the progressive upward flame spread, when the plume emerging from the window openings of a lower lever, alights the combustible materials within a level above, through direct contact, if the window failed or it is open, or through radiative heat transfer.

The fire can spread outside of the building or to adjacent buildings when the combustible cladding systems used for exterior walls are ignited .

The fire spread to adjacent buildings occurs indirectly through radiative heat transfer, when the distances between buildings are too short, or by the direct ignition of combustible materials used for roofs or facades due to the burning particles carried by air currents.

To prevent the fire spread to the neighbourhood, the romanian norm for fire safety, P118-99, recommends to be provided some minimum safety distances between the buildings, in accord with their fire resistance degree.

Table 1. Safety distances between the buildings.

| Fire resistance degree | Minimum safety distances [m] to buildings having fire resistance degree | | |
|---|---|---|---|
| | I - II | III | IV - V |
| I - II | 6 | 8 | 10 |
| III | 8 | 10 | 12 |
| IV - V | 10 | 12 | 15 |

## 3. Factors which influence the flame spread outside of the building

The factors which influence directly the flame shape and the fire severity outside of a building are related with the conditions from the room where the fire started, but also with the environmental conditions like *wind vellocity and direction*. Inside an enclosure, several factors can lead to the flashover occurence and this may result the flame spread outside the openings, or, conversely the fire extinguishment, if the thermal load is insufficient.

In case that flashover occurs, the venting plume will be affected by factors like system of smoke control or automatic venting system which may influence the fire evolution in this stage.

After emerging out of an opening the fire plume spreads on the building facade. *The cladding* *system*, the kind of materials used for lining the frontage, plays an important role in fire spread. In this stage, another determinant factor can be the *window geometry*. Experimental tests showed that the heat transfer by convection is higher for narrower windows and lower when the window opening is squared or it has the width longer that the high. Larger glass sheets have a higher breaking load limit than smaller sheets. Kerski-Rahkonen [8] demonstrated in his studies that the glass breaking is caused by the induced thermal efforts, due to the differences of temperature within the glass sheet, from center towards the shaded border. The moment when the glass fails corresponds with the moment when is reached the breaking effort in the shaded part of the glass. Also, the window joinery can accelerate the glass failure, if it is made by combustible materials, like PVC, which melts before the glass.

The geometry of facade can influence the flame spread. Oleszkiewicz [9]
has shown in his experimental results how the horizontal or vertical projections on a facade can affect the exterior fire plume trajectory. For example, horizontal projections, such as balconies or aprons deflect the fire plume and diminish damage caused by fire on a wall above the window opening where the flames are emerging outwards, while vertical projections, channel the fire plume upward, thus increasing the intensity of fire exposure to the wall above.

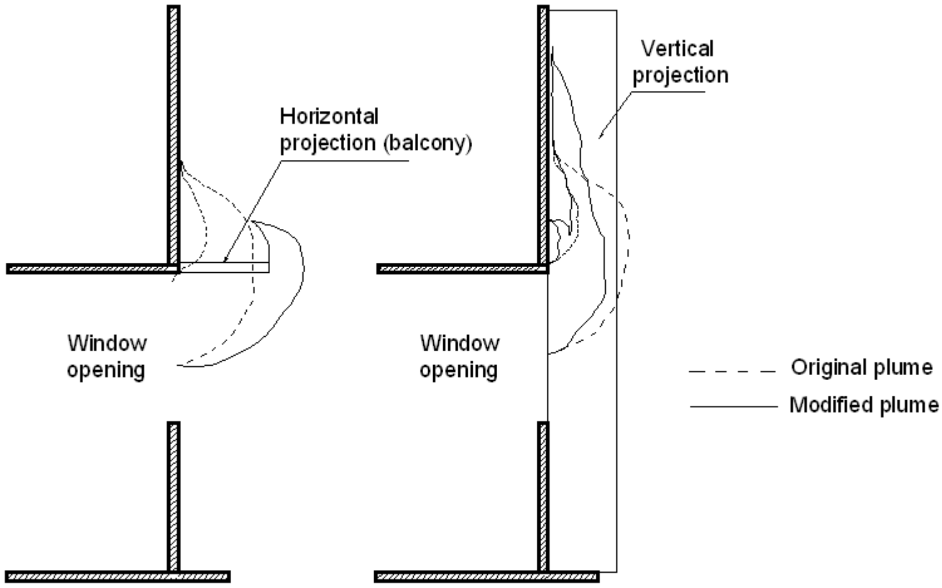



Fig. 1. The influence of horizontal and vertical projections towards venting flames emerging the window openings [9] .

To prevent the upward flame spread on the building façade, the fire design norms from different countries recommend certain minimum size for spandrels between floors and for horizontal projections, respective the minimum necessary width for a balcony to be effective in fire spread prevention. In figure 2 it is reproduced a drawing with the minimum accepted dimensions from The Building Code of Australia. In New Zealand the spandrel between two successive window openings has to be at least 150 cm high.

The romanian fire safety norm P 118-99 prescribes that the boundary walls in storied high and very high buildings to be designed and built up so that to restrict the fire spread from one floor to another, being provided with separatios having minimum 120 cm, fire-proof minimum 30 minutes between glazing. In addition, the casement has to be from non-combustible materials.

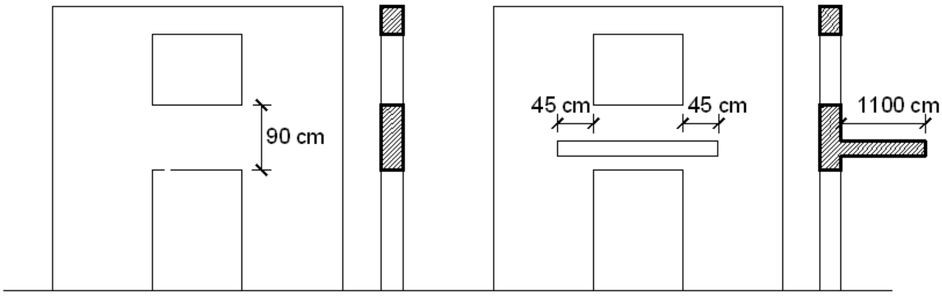



Fig. 2. Deemed-to-satisfy solution from The Building Code of Australia.

## 4. Conclusions

The lack of research about the mechanisms of fire spread outside of a burning room, upward building facade or to the adjacent buildings is a problem in România, in actual conditions when a lot of buildings were insulated with expanded polystyrene. The danger to flame out and to spread a burning fire to the neighbourhood threatens dozens of storied buildings.

The legislation and fire design norms are outdated and are in a process of harmonization with european regulations. There are needed fire safety specialists and designers to provide a new system of technical requirements performance based and in accord with the actual state in România and with european norms.

## 5. References

[1] S. Yokoy, "Study on the Prevention of Fire-Spread Caused by a Hot Upward Current", Report of the Building Research Institute, Japan, Report no. 34 , 1960

[2] C.T. Webster, M.M. Raftery, "The Burning of Fires in Rooms: Part II Test with Cribs and High Ventilation on Various Scales" Joint Fire Research Organisation, Borehamwood, 1959

[3] L.G.Seigel, "The Projection of Flames from Burning Buildings", Fire Technology, Vol.5, No.1, pp.43-51, 1969

[4] P.H.Thomas, M. Law, "The Projection of Flames from Buildings on Fire", Fire Prevention Science and Technology, No. 10, pp.19-26, 1972

[5] M.Law, "Fire Safety of External Building Elements – The Design Approach", American Iron and Steel Engineering Journal, Second Quarter, pp. 59-74, 1978

[6] M.Law, T.O'Brian, "Fire Safety of Bare External Structural Steel", Construction Steel Research and Development Organisation, London, 1989

[7] I. Oleszkiewicz, "Heat Transfer from a Window Plume to a Building Façade", HTD- Vol.123, Colledted papers in Heat Transfer, Book No. H00526, pp. 75-86, 1989.

[8] O. Kerski-Rahkonen, „Breaking of Window Glass Close to Fire, II: Circular Panes", Fire and Materials, Vol.15, pp. 11-16, 1991

[9] I.Oleszkiewicz, "Fire exposure to Exterior Walls and Flame Spread on Combustible Cladding", Fire Technology, pp 357-375, 1990.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Issues Regarding Stadiums Resistance Structures

N. Socaciu [*1], B. Petrina[2], Al. Cătărig[3], T.Petrina[4], R. Hulea[5], R. Zoicaş[6]

[1,2,3,4,5,6]*Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

Stadiums are impressive buildings through their structural shapes and the technical solutions adopted, real works of art, in most cases perceived as representative buildings for the nation`s development and civilization level at that time. This study describes the structural concept of Cluj-Napoca City Stadium. The paper consists of two parts, the first containing an architectural description, then the second, describing the resistance structure, loadings and structure`s particular elements. The adopted foundation solution is given by rigid isolated foundations under the columns and continuous under diaphragms, with the mention that the superstructure is in monolithic and precast reinforced concrete frames and the roof is a 3D steel structure.

## Rezumat

Stadioanele, construcţii impresionante prin formele structurale şi soluţiile tehnice adoptate, adevărate lucrări de artă, pot fi considerate, în cele mai multe cazuri, edificii reprezentative pentru nivelul de dezvoltare şi civilizaţie al naţiunilor la momentul respectiv. În prezenta lucrare este descris conceptul structural al stadionului municipal din Cluj-Napoca. Lucrarea este structurată în două părţi, prima conţinând o descriere arhitecturală, iar în cea de a doua fiind descrisă structura de rezistenţă, încărcările şi elementele particulare a structurii. Soluţia de fundare adoptată este fundaţii izolate rigide sub stâlpi şi continue sub diafragme, suprastructura fiind în cadre din beton armat monolit şi prefabricat, iar acoperişul este o structură metalică spaţială.

**Keywords:** Stadiums; Precast reinforced concrete frames; 3D steel structures.

## 1. Stadium structures examples

### *1.1. Allianz Arena*

Stadium`s building (Fig.1), with a capacity of 69901 seats, was completed in 2005. It is an impressive modern structure, with a partially-closed roof. The stadium has 11000 parking lots. Allianz Arena stadium structure consists of eight modules (Fig.2), four for core (middle) and 4 for corner, this due to its bowl shape. It is a mixed structure. The roof and the façade are made of steel, standing on a structure of reinforced concrete frames. The resistance elements in each unit are: reinforced concrete columns, insulated foundations connected by beams, floors and reinforced concrete beams, precast elements. The amount of material used was: 120.000 $m^3$ of concrete and 22.000 *tons* of steel. The structure is sustained by 350 columns, 10.000 *kN* on each. Structure`s steadiness (stability) is provided by eight elevators and staircases. The units in the middle are

[*] Corresponding author: E-mail address: nicolae.socaciu@arch.utcluj.ro

positioned in 10 reinforced concrete frames and a staircase with the main purpose of stabilizing the structure and to retrieve the torsion efforts. The reinforced concrete frames are designed to take the horizontal and vertical forces. The frame structure takes over the supporters' dynamic action as well.

Fig.1. Allianz arena Source:
http://images.businessweek.com/ss/06/06/worldcup_arena/image/intro.jpg

.

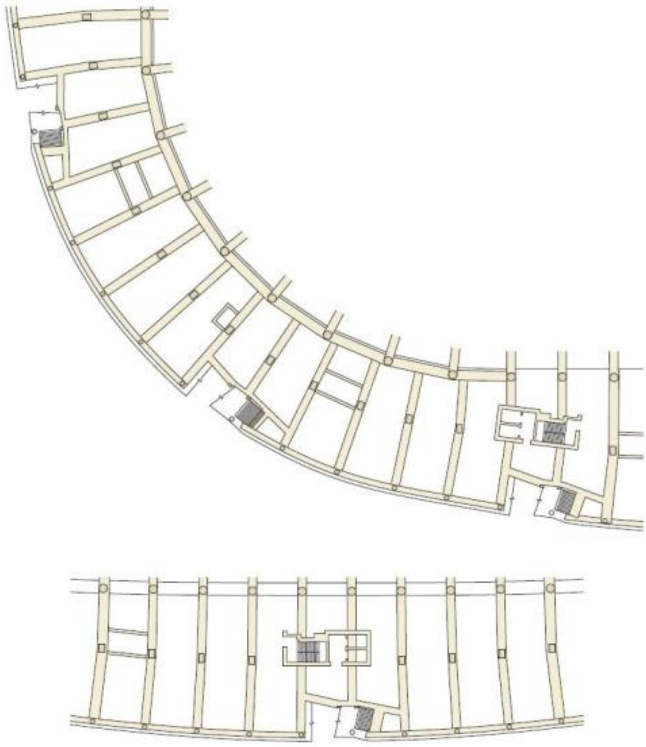


Fig.2. The eight corner-and-core modules
Source: The Allianz Arena: A new football stadium for Munich, Germany

Corner units are much more complicated and more important in the body structure due to the radial shape. The corner area is organized in a grid with 16 divisions with a length of 120 *m*. The corner section principle is not different from the middle sections one, but the issue of the loadings in the structure transfer and the people in the tribunes is far more complex

The columns are made of reinforced concrete B45, and B55 where there are higher voltages. On the outside of the building there are prefabricated circular columns connected through floors with steel mounting plates on the top and bottom sides. Because of the console roof, in these columns results tensile efforts as well. The columns follow the exterior façade, they change direction on every level, which creates additional horizontal efforts inside the frames.

Figure 3 symbolizes the elements that ensure the entire frame stability.

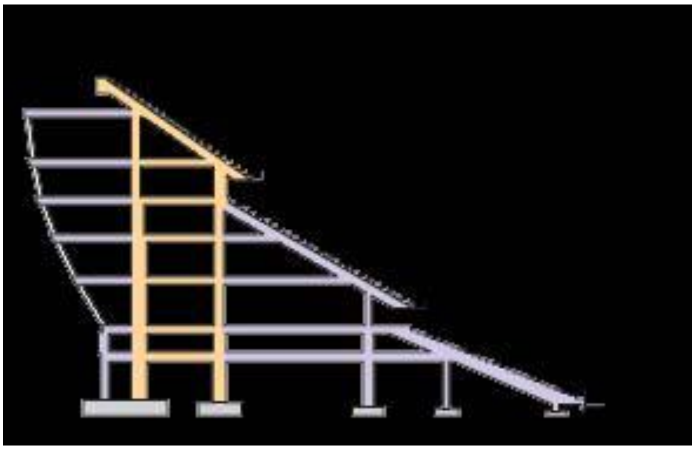


Fig.3 The elements that ensure stability
Source: The Allianz Arena: A new football stadium for Munich, Germany

The roof is a console-shaped steel structure (Fig.4).The usage of fixed links on the columns it is necessary for easing (helping) the transfer of the tensions from the console to the foundations. The columns that are supporting the roof take the wind loadings, these can be either of pressure or suction and of their own weight.

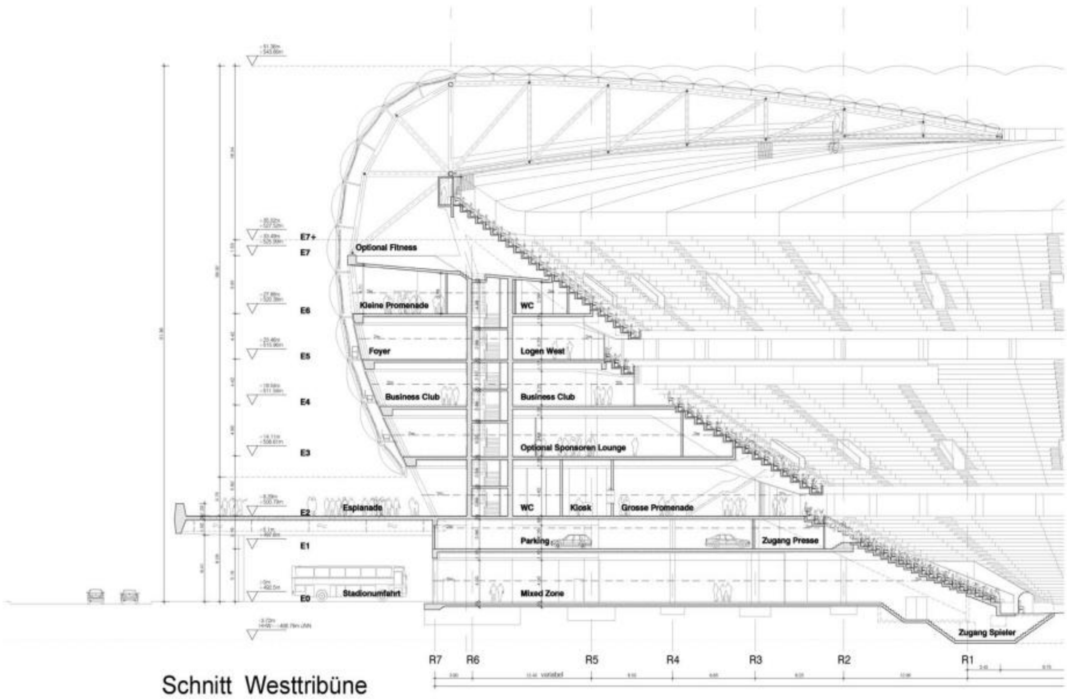



Fig.4. Cross section
Source: The Allianz Arena: A new football stadium for Munich, Germany

The most interesting part of the stadium is the façade, covered with ETFE (a polymer of tetrafluoroethylene and ethylene), a special type of polymer, with a thickness of 0.2 *mm* and insignificant weight, mounted on steel. This is a waterproof, snow and fire resistant material. Mass production was not possible as the ETFE pieces were different sizes. For their enlightenment have been used 5.344 lighting sources. The pillow-shaped membranes are filled with low pressure air, which provides good isolation. This type of cover retains 90% of solar energy, reducing energy use by 30%.

### *1.2. Braga Stadium*

The stadium was completed in 2003, in Braga town, Portugal (Fig.5) for the 2004 European Footbal Championship, with a capacity of 30.154 seats.

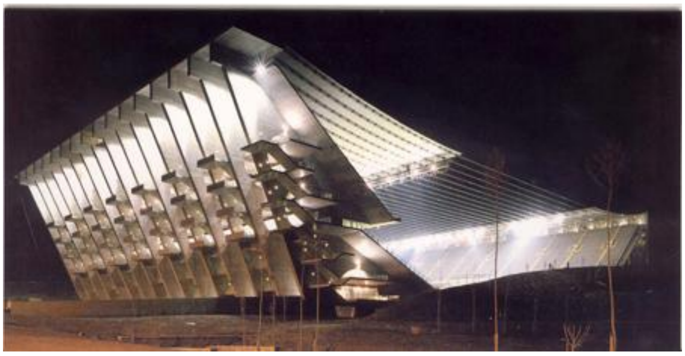


Fig.5. Braga Stadium
Source: http://www.svibs.com/images/braga.jpg

The stadium was located at the foot of Mount Castro, on a granite hill. The stadium has two tribunes, one is integrated into a rock, and the other one is independent.

The biggest problems that the designers had to face were the roof and rock excavation. The roof had to be integrated into the landscape. The solution came from a suspended roof used in the Portuguese Pavilion in EXPO 98, where they used the rock as steel cable anchor. The original idea was to build a complete roof, for a better stability. Thus it was not given enough light to the football field, problem solved just by covering the tribunes.

The most impressive structural element of the stadium is the roof. Made of cables spaced 3.75 *m* apart, they sustain two precast concrete roof tiles covering the tribunes. The opening between them is 202 *m*, and the fact that between the two roofs cables were loose (free) is a challenge. Anchoring cables was done inside the two beams on the top of the tribunes.

The Eastern side has a height of 50 *m*, stiffened by the floors and walls have a thickness of 1 *m*. Designing the Western tribune was more difficult because of the structure and foundations work and the vertical rock wall where the structure was anchored.

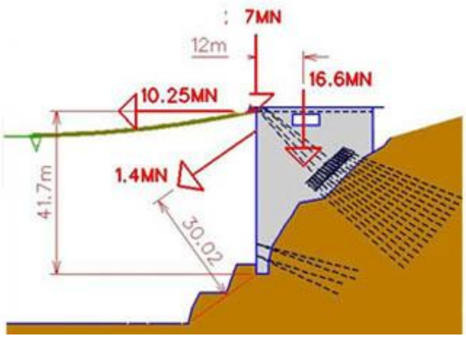



Fig.6. The balance of forces on the Western tribune
Source: New Braga Municipal Stadium, Braga Rui Furtado, Eng., Carlos Quinaz, Eng., Renato Bastos, Eng.,AFAssociados, SA,V. N. Gaia, Portugal

Under the stadium were built two levels comprising the locker rooms, parking lots and other units, necessary for the homologation. Slope excavation and stabilization was difficult because were excavated approximately 1.7 million $m^3$ of rock. The roof is made of steel with a single curve. The cables are anchored in reinforced concrete beams, the beams being stabilized by their own weight. The behavior in wind was tested into the wind tunnel with the help of the rigid and elastic model. To drain the water from the roof cables are successively left foot from end to end, allowing water to drip from the ends.

To determine the wind effects, the medium speed and turbulences due to land topology, were performed wind tunnel tests on a 1:5000 scale model. Determination of wind pressure considered from multiple directions was obtained in wind tunnel on a 1:400 scale rigid model. The roof was monitored using 200 pressure sensors positioned both on the top and bottom. The values were automatically recorded for 36 directions at a 10 degrees interval. The calculation of the structure to dynamic wind loading with values obtained from tests was done using two methods. The first was a deterministic dynamic analysis *time history*, by step by step integration of dynamic equation obtained from the linear elastic analysis of structure plates using the finite element method with the stiffness matrix which was assumed to be constant. The stiffness matrix was obtained using the axial effort from the cables from the permanent charge. By this analysis the dynamic response of the structure to the wind dynamic action can be calculated. The maximum displacement resulted is 47 *cm*. The second method is a probabilistic dynamic analysis using orthogonal decomposition method. The method is based on the principle that a variable pressure field can be simplified by designing on a space generated by the covariance matrix`s own vectors of the initial field. This method has two major advantages. First of all the new fields of pressure are mutually uncorrelated. Secondly, the energy content of the entire domain is well represented by several components in the transformed area, allowing representation of the pressure field through less pressure ways (means). This analysis allowed the estimation of the quasi-stationary and resonance components from the structural response in terms of stresses and strains. These values were used in designing more roof structural elements. Mathematical designing based on the wind tunnel test results with the first physical patterns guarantee the possibility on non occurrence of instabilities caused by wind on the roof. Although it was agreed that such conduct would be extremely unlikely it was decided to develop physical models that were to be tested in a wind tunnel. The aerodynamic stability of the initial solution was proved by tests on a 1:200 scale model. The lack of aero elastic instability in the final solution was proved by tests on a 1:70 scale model. Both models proved that the aero elastic stability of the roof and the maximum measured deflection was almost equal to that calculated.

## 2. The City Stadium of Cluj –Napoca

### *2.1. The presentation of The City Stadium of Cluj-Napoca*

The stadium is situated in Cluj-Napoca in the Central Park, on the south side of the Somes River. The new stadium is under construction now, it has international standards, will have an approximately 30000 capacity, will follow all the standards imposed by FIFA and UEFA and also will follow the codes that rules athletics tracks of "A" category (Fig.7).

The levels on height will be: two underground levels, ground level and two stories in the north and south parts and with five stories in the main parts. The maximum height is 36.60 *m* and at the cornice is 33.60 *m*.

Figure 7. City Stadium of Cluj-Napoca

The transversal section of stadium, exhibiting both, the infrastructure and superstructure (reinforced concrete up the level +24.88 and steel up the level +36.73) is presented in figure 8.

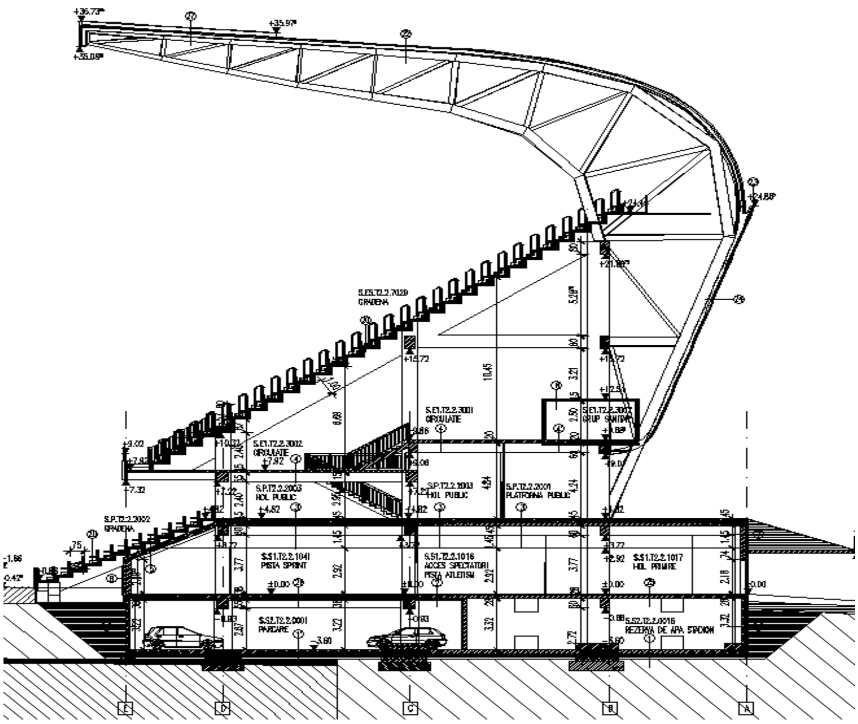


Figure 8. Transversal section of the T2

### *2.2. The structural solution*

* <u>*The infrastructure*</u>

The foundation solution chosen for all parts is isolated foundation under the columns and continuous foundation under the reinforced concrete walls.

The isolated foundations are made of a plain concrete part plus an upper part made of reinforced concrete. The continuous foundations are also with a part made of plain concrete plus an upper part made of reinforced concrete. The isolated foundations go to a depth of -5.15 *m* to -6.55 *m* (measured from 0.00 level) and at -8.05 *m* in the area where the underground depth is bigger. These foundations are connected by reinforced concrete beams on both directions. The foundations will be placed for all parts T1 and T2 in the ground layer called diorite sand with a conventional base pressure of $P_{conv}$ = 750 *kPa* and for the parts P1 and P2 in the layer made of sand and grabble with a conventional base pressure of $P_{conv}$ = 450 *kPa*.

According to the geotechnical study the ground water is moving like a blade in the alluvial gravel layer with sand and binder. The general flow direction is towards Somesul Mic river bed, the ground waters draining themselves towards Somesul Mic river bed. In normal rainfall conditions the geotechnical drillings carried out on site show that the aquifer horizon has relatively small thicknesses, measured in *dm*. During periods of high precipitation ground water can occur around the rate of -2.00 *m* from the current ground level.

In order to detect land stratifications and setting foundation conditions 24 geotechnical drillings were made and drillings and data were taken from previously drawn geotechnical studies on site. Here are the layers:

- Earth filling with rocks, gravel and bricks.
- Powder sand/brown dust clay soft or consistent plastic.
- Tamping medium brown sand.
- Gravel with sand and binder.
- Gravel with interspaces filled with brown consistent plastic clay.
- Gravel with sand and binder- stuffed or average tamping.
- Hard gray shale (basecoat badenian age) - semi rocky compact rock.

The tribunes` resistance structure is made in frames with reinforced concrete columns and beams. The plates are made of monolithic reinforced concrete with and without precast under-panels ($h_p$ = 20 *cm*) and are calculated to form rigid washers in their plan, for taking away the horizontal loadings. The tiles download on the frame beams (monolithic and prefabricated). The tribunes will be made of precast reinforced concrete and will download on the frame beams too.

On the basements contour reinforced concrete elevations will be made, equipped with belts both at the top and bottom. The elevations will be vertically waterproofed with thermo welded membranes protected on the outside with Tefond type membranes. In connection areas special connection profiles will be provided. Given the high rate of groundwater level, the ground plate (board) will be 25 *cm* thick to withstand water pressure. The board will be linked with the isolated foundations and with the continuous foundations belts. First concrete will be poured for leveling over which is made the thermo welded membranes waterproofing. To the connections will be installed special connection profiles (sections) and plastic stoppers. The waterproofing will be protected by a concrete screed and over it will be made the plate (board) reinforcement.

In areas of columns and diaphragms, on the foundation blocks will be ran rigid waterproofs and membranes will be connected over them.

The vertical waterproofing on top of the elevations will be connected with the horizontal one resulting a tight tank at the basements.

* <u>The superstructure (overall structure)</u>

The bulkheads will be made of varying sizes masonry, depending on the marked (enclosed) areas. Stairs will be designed as reinforced concrete ramps, resting on level floors and beams. The terraces, ramps and stairs from the ground level will be made of reinforced concrete on Proctor 95% compacted ballast filling.

The resistance structure of the roof is made of a plane cantilever truss that covers the seats. The steel structure for the roof will be made of parts pre-assembled or assembled on site on the ground or directly at the position. On the longitudinal direction plane trusses are designed with the goal to create rigidity for the cantilevers on that direction. The final cover will be a light one. At the roof level joints are created by placing simply supported longitudinal elements. The steel structure of the roof is divided into four parts and the cantilever trusses are placed on the reinforced concrete frames. In the figure 9 one may find an opening from T2. The expansion joints are placed in correspondence with the joints created in the concrete structure. In these joints the longitudinal elements are simply supported to the cantilevers.

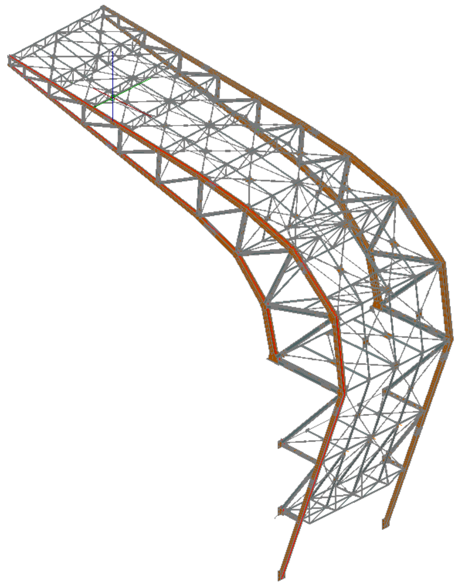


Fig.9. The steel structure of the roof. Opening from T2

These trusses are made of a column and a cantilever beam. The truss is fixed in the reinforced concrete column having the section of $1.20 * 0.80\ m^2$ in four points. The joints where created screwed with base plates, their dimensions where computed according to the corresponding efforts. The truss is made of four sections in order to be transported and put together. The longitudinal parts of the cantilever are made of steel plates welded together in the form of "H", the diagonals and the vertical elements of the truss are made of two "U" profiles placed face to face and connected with steel plates. The joints between the four sections are made of SIRP. The joints between the elements of each section are welded.

The structure was made rigid by introducing diagonals in the roof plane made of pre stressed bars. These are of two types: longitudinal and transversal diagonals. The longitudinal ones were placed at the end parts of the truss and in the connection area between the column and the beam, in the free part of the cantilever and on the lower side of the column. The transversal diagonals are placed two pieces at each end of roof section, parallel to the truss. The longitudinal diagonals that aren't in the roof plane are made of pipe profiles for the longitudinal elements and vertical elements and have pre stressed bars as diagonals.

The longitudinal elements of the roof are pinned to the trusses in the upper nodes and they are also trusses made of pipe-type profiles. Some of these longitudinal trusses are linked with ties with the lower node of the main truss, too. For transversal fixing the lower side of the longitudinal trusses, transversals are used.

* <u>*Special features (particularities)*</u>

\- <u>Steel types</u>

Choosing the steel was made according to the necessities of the objective in question. Thus, for each type of item the following types of steel were chosen:

a. *Steel sheets*: S355J2+N with secured KV at $-31^0C$ or S355K2+N with secured KV at $-31^0C$.

b. *Laminated elements:* S355ML – fine granulated structural steel obtained by thermo-mechanic lamination, with secured KV at $-31^0C$.

c. *Steel sheets and the round steel, the material for the concrete embedded bolts* will be made of S355J2+AR steel, with secured KV at $-31^0C$.

d. *Round steel in windbreak ties* S460ML - fine granulated structural steel obtained by thermo-mechanic lamination, with secured KV at $-31^0C$.

e. *The assembly parts:* the material according to EN 10083/2004, eith secured KV at -31$^0$*C.*

- <u>The welding</u>

Quality conditions are established on the principle of making some equal minimum resistances inside the welded connection with those prescribed for the basic steel.

The following welded connections are performed:

a. The splices from truss soles parts, that parts of the heart, will be prepared for welding according to SR EN ISO 9692, depending on the welding procedure, thickness and direction of the pieces (parts) that form the welding (from one side or both sides of the piece (element)).

b. Welding on the one side it is for thin or thick elements where the second side is not accessible.

c. But seams (in depth) are made on the full items` thickness and width. To avoid areas with incomplete submissions are used extension elements from the beginning to the end of the welding (or another approved procedure that ensures the welding a bearing capacity, equal to that of welded parts after processing the elements).

d. Diagonals and columns from U profiles are connected to the truss soles with depth welding, with root re welding. Gripping the overlapping soles over the truss soles is done with landscape welding.

e. Landscape welding will be made with $\mathbf{a} = 0{,}7\mathbf{t}_{min}$ thicknesses ($\mathbf{t}_{min}$–pack pieces` minimum thickness) and length throughout the elements` overlapping parts.

Designing welding connections entails:

◦ Avoiding as much as possible the installation welding connections at the dynamic required elements and locating them in sections with low stress.

◦ Butt-welded connections offset in different sections of the element.

◦ Avoiding compositions ways of the elements that conduct to the appearance of remaining efforts or deformations above the accepted limits during welding procedures.

◦ Avoiding welding upside and vertical downward position , through a composition of elements that allows the making of assembly connections horizontally (at the table or in the butter), horizontal in a vertical level or in upward vertical position.

Here are the measures taken to reduce residual stresses caused by welding:

◦ Wide thickness welding will be performed in several layers.

◦ Welding the long cords in sequences or leaps.

◦ Stiffness will be welded after combining parts.

◦ The usage of a fastening that prevents parts twisting.

The presets for quality referring to parts cutting, the presets for edge processing and connections for welding (the shape of welded connection), are given in Table 3 from C150-99 " Standard regarding steel welded connections` quality of civil, industrial and agricultural engineering", depending on welded connections quality levels. Connections types will be the first ones used according to SR EN ISO 9692-2004 "Welding and allied processes. Recommendations for connection`s preparation".

Performing (creating) steel elements at the stadium domes was set within the A Execution Category of the element.

* <u>*Loadings*</u>

Loadings are outside forces pressing on the structure. They divide into static and dynamic loadings. Static loadings are those whose intensity is independent of time, so it is unnecessary to introduce inertial forces into calculations. Dynamic loadings are those whose intensity switches so fast in time that introducing inertial forces into calculations becomes necessary.

Stadiums roofs are structures that are sized mostly from wind`s and snow`s action (pressure).

- <u>Permanent and technological loadings</u>

Permanent loadings were evaluated taking into account the weight of building elements, with the technical loadings given by manufacturer. The load on reinforced concrete plates was assessed according to each element`s component layers resulting values between 3.31 and 6.20 $kN/m^2$. The permanent loadings pressuring on the steel truss were evaluated to 0.60 $kN/m^2$, representing the weight of the roof elements (folded board + the cover). According to regulations, appeared the 0.20 $kN/m^2$ loading, made of technologic equipment weight.

- Wind load

Wind is air movement towards ground, directed by forces resulted from atmospheric pressure differences, also resulted from differences of heating from various world areas and the general force generated by earth rotation.

For the steel structure following loadings (charges) were considered: 1.99 $kN/m^2$ field suction and 2.99 $kN/m^2$ on console`s end, 0.96 $kN/m^2$ field pressure on the console and 1.55 $kN/m^2$ on the end of the console, 0.66 $kN/m^2$ suction against the wall, 2.15 $kN/m^2$ wall pressure.

For a better interpretation of wind action has been conducted a simulation, with a program that analyses fluid dynamics. Figure 10 shows the wind speed.

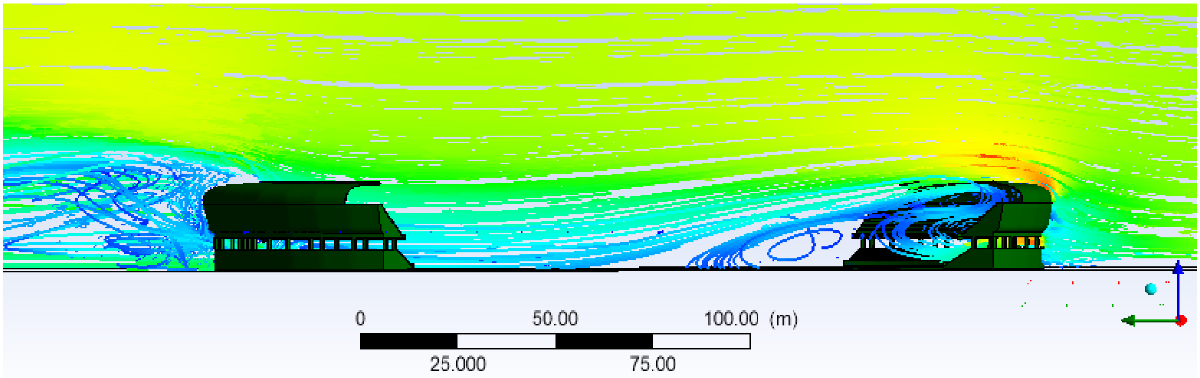



Fig.10. Wind action against the lawns

- Useful loadings

Loadings, due to operating (exploitation) process were determined taking into account building`s purpose and the those operating (exploitation) conditions. These are the maximum values in present operating(exploitation) conditions. Calculation values are 5 $kN/m^2$ on communication channels (means) and 6 $kN/m^2$ on tribunes. The vertical loadings on rails located on spectator`s radar increased to 2.5 $kN/m^2$. The horizontal useful loading is considered distributed on the same surface where the vertical useful loading acts, without being cumulated to wind or earthquake loading.

- Snow loadings

The characteristic value of snow loading on ground is defined by 2% probability of exceeding in one year or with an average recurrence interval IMR = 50 years. This characteristic value has a higher achievement probability of 50% during building`s lifetime.

Snow loading is considered acting vertically on roof surface horizontal projection.

The characteristic loading $S_k$ is given by

$$S_k = \mu_i C_e C_t s_{0,k}$$

with:

$C_e$ = 1- partial exposure, according to Tabel 2.1. from CR 1-1-3 - 2005,

$C_t$ = 1- common waterproofing roof,

$s_{0,k}$ =1.5 [$kN/m^2$] - according Tabel A1 from CR 1-1-3 - 2005,

$\mu_i$ = 0.8 – plat roof

and then

$$S_k = 1 \cdot 0{,}8 \cdot 1 \cdot 1{,}5 = 1{,}20\ [kN/m^2]$$

Snow overcrowding on the edge of a roof is calculated with

$$s_c = K s_k^2 / \gamma$$

with:

$K$ = 2.5 – coefficient that depends on the irregular snow shape,

$\gamma$ = 3 $kN/m^3$ – specific snow weight,

$s_k$ = 0.96 $kN/m^2$ – characteristic value of snow loading on the roof in the worst case on snow deposition.

Performing calculations, the result is:

$$s_c = \frac{2.5 \cdot 0.96^2}{3} = 0.768 kN/m$$

- <u>Seismic loadings</u>

According to P100-1/2006 Standard for anti seismic buildings design, to the designed building ground acceleration, for designing, is $a_g$ = 0.08g, for seismic events with the average recurrence interval IMR = 100 years. Response spectrum`s control time $T_C$ is the border between the area with maximum values in the absolute acceleration spectrum and the area with minimum values in the relative speed spectrum. $T_C$ is expressed in seconds. For the designed building $T_C$ = 0.7 *s*. The building is in the 2nd importance level (importance factor $\gamma_I$ = 1.2), public building with 400 persons in the exposed area. For buildings located in seismic areas characterized by values of $a_g \leq$ 0.16g, may be adopted a design that endows the structures with high ductility capacity, with a corresponding resistance increase (spor?). In this case, buildings are in a high ductility level (class). Building`s importance category is “B” (Very important building, according to the Governmental Decision no.261, O.G. nr.2 /1994).

The normalized elastic answer spectrum for accelerations, for ground`s movement horizontal components, in the area defined by time of control $T_C$ = 0.7 *s* is described in figure 11.

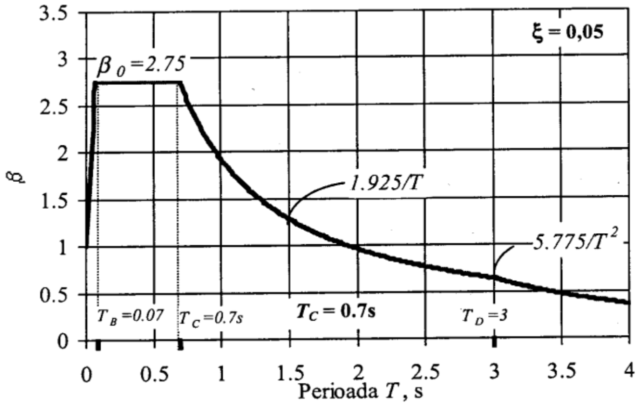



Fig.11. Normalized answer spectrum

* <u>*Cluj-Napoca Stadium`s particular elements*</u>

- <u>Monolithic column merge and stairs` precast support beam</u>

The items that intersect in this connection are the monolith top-forked column and a prefab precast concrete beam inclined with step-shaped top whose purpose is to support the stair. This connection is represented in figure 12. The connection is provided with a steel element (a pin ). Monolith column`s dimensions are: $80*80\ cm^2$ and in the peak is fork-shaped. Prefab bream`s dimensions are $50*65\ cm^2$ , and at connection level has a spur for a good transmission of the compression effort. The pin, made of a circular pipe whose size is $\Phi 219*10$, is designed to take the effort`s horizontal component out of the prefabricated beam and to send it through the monolith precast concrete column fork. The pin was calculated at shearing and bending. The fork was also checked at crushing and pressure on wall holes.

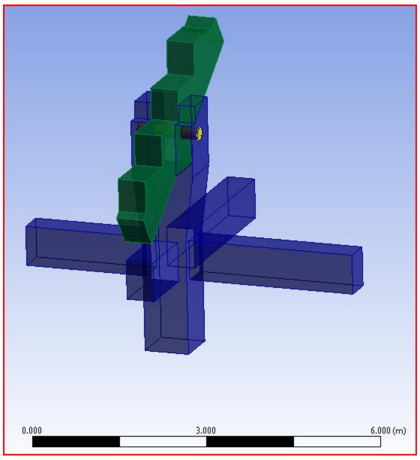



Fig.12. Monolith column connectioning and prefabricated beam that supports the stairs

- Precast concrete structural connectioning (connecting) with steel structure

The steel structure connects with the precast concrete column through steel plates with bolts anchored in concrete and spurs for taking over the sheer force from knot level. Figure 13 shows a graphic representation of the connection. Highly resistant screws were used for a good connection.

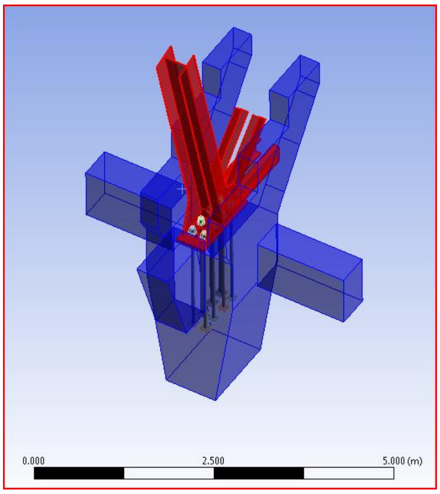



Fig.13. Steel and concrete structures connected

- Steel knots

The steel structure presents two types of connections. In the sections these are connections with screws and clips, and between the elements of a section profiles` connections are designed with welding.

- Structure sectioning

Both steel and precast concrete structures were sectioned according to the applicable regulations (standards). The tribunes were provided with three 5 *cm* thick expansion connections, and the lawns were sectioned in a single place. The connection to the precast concrete structure was made by inserting two intermediary precast concrete frames on both sides of the connection, and to the steel structure the connection was made by releasing the movement restriction on the long direction of the wedge at one end.

- The conception of taking the groundwater charge

The foundation is 25 *cm* thick and was calculated and made with articulated grip to the columns` pads. This articulation was made by installing a beam high as the foundation on its edge, this beam attached with lining`s and foundations` left mustaches. For groundwater not no enter the building a waterproofing was provided under the foundation.

## 3. REFERENCES

[1] Burrows, S., Ecker, K., Guesgen, J., Lutz, R., Miehe, B., Moerland, P., Parrish J., Reinardy, R., Schenk, F., Thompson, I., Uys, E., *The Allianz Arena:A new football stadium for Munich,* Germany, The Arup Journal, no.1, 2006.

[2] Furtado, R., Quinaz, C., Bastos, R., Associados, A., Gaia, N., *New Braga Municipal Stadium, Braga,* Structural Engineering International, no.2, 2005.

[3] Mateescu, D., Caraba, I., *Construcții Metalice,* Editura Tehnică, 1980.

[4] Petrina, M., Socaciu, N., Rus, P., Pop, G.I., Ionascu, A., Petrina, T., Hulea, R., *Structural Concepts of the Municipal Stadium from Cluj-Napoca,* Civil Engineering – Towards a Better Environment and the Concrete Future, Coimbra, 2009.

[5] Petrina, M., Socaciu, N., *Proiect tehnic Stadionul Municipal, la standarde internaționale,* Cluj-Napoca, 2010.

[6] Petrina, M., Socaciu, N., Rus, P., Petrina, B., Hulea, R., Zoicaş, R., Mojolic, C., Petrina, T., *Researches on Design Optimiyation of the Resistance Structure of the Cluj-Napoca City Stadium in the Case of Loads due to Temperature Variation,* Analele Universității din Oradea, Fascicula Construcții şi Instalații hidroedilitare, Vol. XIII-2, 2010.

[7] Petrina, M., Socaciu, N., Petrina, B., Hulea, R., Zoicaş, R., Mojolic, C., Petrina, T., *The Conception of the Resistance Structure of the City Stadium of Cluj-Napoca under Accidental Loadings,* Analele Universității din Oradea, Fascicula Construcții şi Instalații hidroedilitare, Vol. XIII-2, 2010.

[8] Siminea, P., Negrei, L., *Construcții metalice. Calcul prin metoda starilor limită,* Editura Didactică si Pedagogică, 1982.

[9] Socaciu, N., Petrina, T., Hulea, R., Moldovan, I.F., Vidrean, R., Mojolic, C., *Conceptual Structural Solutions for the Resistente of the Stadium Roof,* Building Services, Mechanical and Building Industry Days, Debrecen, 2009.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# Precast Reinforced Concrete Elements of the Structure of the City Stadium of Cluj-Napoca

Tudor Petrina [*1], Mircea Petrina[2], Nicolae Socaciu[3], Bianca Parv[4], Monica Nicoreac[5]

[1,2,3,4,5]*Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*This study describes the precast reinforced concrete elements of the Structure of the City Stadium of Cluj-Napoca. These elements were realized in order to obtain a smooth concrete surface, because the concrete remains apparently. The first part of the study contains the detailed description of the structural solution, in the second chapter is about precast under-plates and precast beams for the slabs. In chapter 3 the stepped beams that sustain the tiers are presented, then in chapter 4 we have the tiers of the grandstand. In chapter 5 other precast elements are mentioned. At last, in chapter 6 we have the conclusions.*

## Rezumat

*Aceasta lucrare descrie elementele prefabricate din beton armat ale structurii Stadionului Municipal Cluj-Napoca. Aceastea s-au realizat din dorinta de a avea o suprafata cat mai neteda a betonului, deoarece betonul ramane aparent. Prima parte a lucrarii cuprinde descrierea detaliata a solutiei structurale pentru obiectivul studiat, in capitolul 2 se prezinta predale si grinzi prefabricate pentru solutia de planseu cu predala. In capitolul 3 se descriu grinzile inclinate ce sustin gradenele, in capitolul 4 se prezinta gradenele, iar in capitolul 5 se mentioneaza si alte elemente prefabricate. In fine in capitolul 6 sunt prezentate concluziile.*

**Keywords:** stadium, precast, grandstand, tiers, beams, trusses

## 1. The presentation of The City Stadium of Cluj-Napoca

The stadium is situated in Cluj-Napoca in the Central Park, on the south side of the Somes River. The new stadium is under construction now – see figure 2, it will have an approximately 30500 capacity, will follow all the standards imposed by FIFA and UEFA and also will follow the codes that rules athletics tracks of “A” category. (See figure 1) [1,10]. The levels on the height will be: two underground levels, ground level and two stories in the north and south parts and with five stories in the main parts. The maximum height is 36.60m and at the cornice is 33.60m [1, 10]. (See figure 3 and 4).

[*] Corresponding author: E-mail address: Tudor.Petrina@mecon.utcluj.ro

Figure 1. City Stadium of Cluj-Napoca

Figure 2. The stadium is under construction

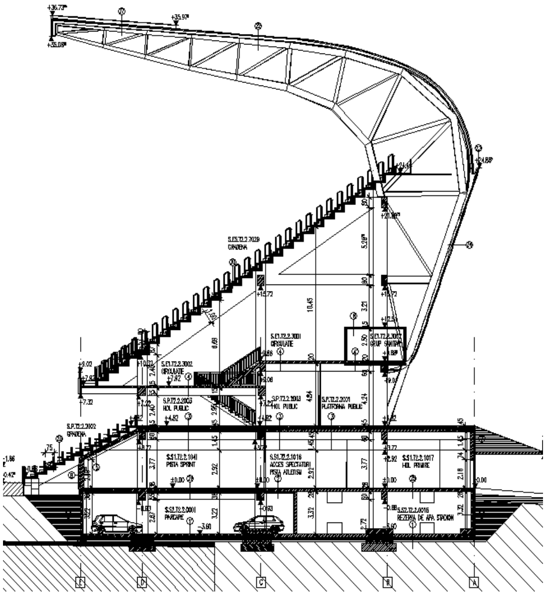


Figure 3. Transversal section of the grandstand T2

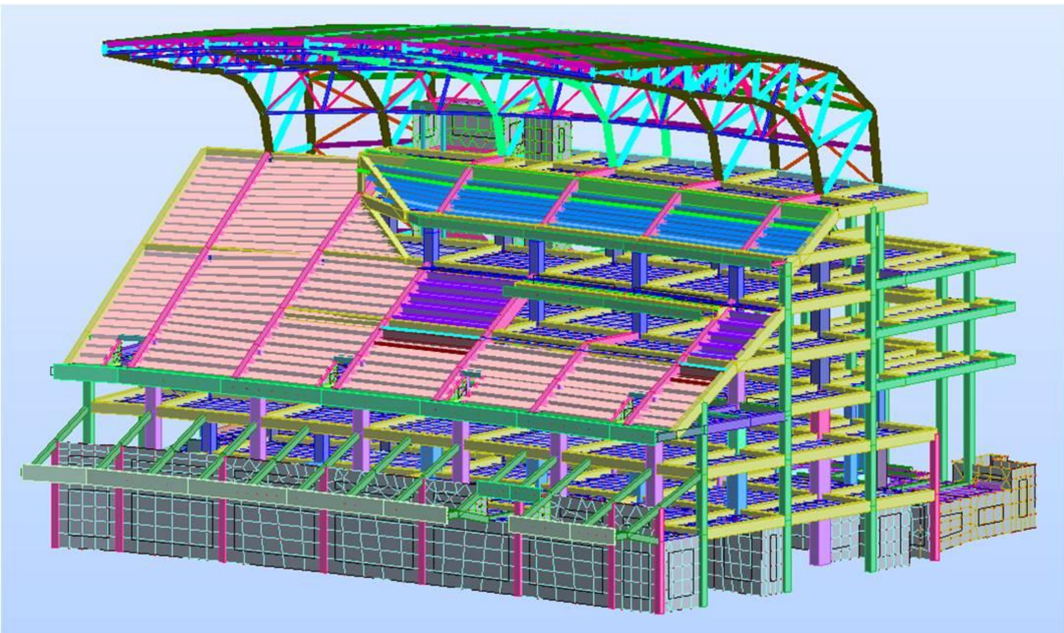


Figure 4. Grandstand T2.1

The foundation solution chosen for all parts is isolated foundation under the columns and continuous foundation under the reinforced concrete walls [1, 10].
The isolated foundations are made of a plain concrete part plus an upper part made of reinforced concrete. The continuous foundations are also with a part made of plain concrete plus an upper part made of reinforced concrete. The isolated foundations go to a depth of -5.15m to -6.55m (measured from the level 0.00) and at -8.05m in the area where the depth of the underground is bigger. These foundations are connected by reinforced concrete beams on both directions. The foundations will be placed for all parts T1 and T2 in the layer of ground called diorite sand with a conventional base pressure of $P_{conv}$ = 750kPa and for the parts P1 and P2 in the layer made of sand and grabble with a conventional base pressure of $P_{conv}$ = 450kPa [1, 10].
The resistance structure of the stadium is made of frames with reinforced concrete columns and beams. The slabs are made of reinforced concrete cast on site, with or without precast under plates ($h_p$ = 20cm) and are computed to have adequate horizontal rigidity to undertake horizontal loading. The slabs contain beams (cast on site or precast). The stepped slab where the seats are placed lay on oblique frame beams [1, 10].
On the contours of the underground levels, reinforced concrete walls are designed to have tie beams on their base and also upper side. These walls are hydro-insulated with thermo-welded membranes protected with a Tefond-type layer. In the joint zone special joint pieces are placed. Because the water level is rather high, the slab on the ground will have a thickness of 25cm, to resist the water pressure. This slab is anchored to the isolated and continuous foundations. The reinforcement is designed to undertake water pressure. First, an equalizing layer of concrete will be placed and then the hydro-insulation made of thermo-welded membranes is realized. At the joints, special pieces and plastic taps will be placed. The insulation will be protected with a thin layer of concrete on which the reinforcement of the slab will be realized. In the columns and walls zones, rigid hydro-insulations will be realized that will be connected to the membranes. The vertical insulation will be connected with the horizontal one, developing a sealed bowl of the underground levels [1, 10].
The resistance structure of the roof is made of a plane cantilever truss that covers the seats. (See figure 5). The steel structure for the roof will be made of parts pre-assembled or assembled on site on the ground or directly at their final position. On the longitudinal direction plane trusses are designed with the goal to create rigidity for the cantilevers on that direction. The final cover will be a light one.

Figure 5. The steel structure of the roof: under construction.

The main truss is fixed in four points in the reinforced concrete column having the section of 1.20m x 0.80m. The longitudinal elements of the roof are pinned to the trusses in the upper nodes and they are also trusses made of pipe-type profiles. Some of these longitudinal trusses are linked with ties with the lower node of the main truss, too. For transversal fixing the lower side of the longitudinal trusses, transversals are used [1, 10].

## 2. Precast slab elements – under-plates and beams:

In some parts of the stadium, the slabs were realized by using precast under – plates. Their thickness is equal to 7cm and they have stiffening trusses inside them. (See figure 6).

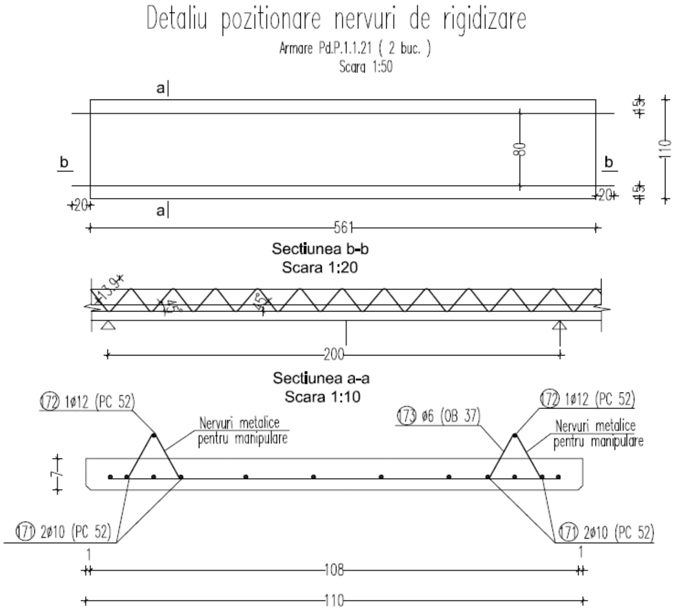



Figure 6. Each panel (reinforced concrete plate) has stiffening trusses

After the fixing in the right position they were reinforced on the upper side, and another 8cm of reinforced concrete was poured. (See figure 7)

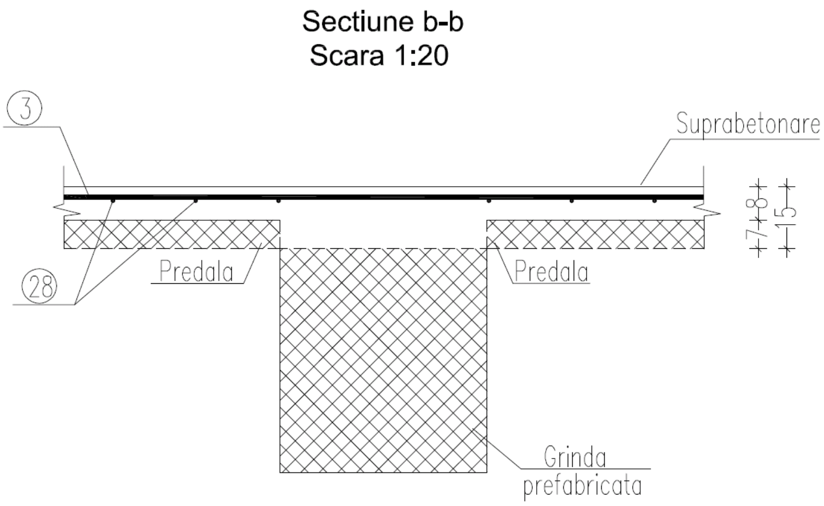



Figure 7. Slab realized using under-plates (predale)

The beams are realized in the same manner, by realizing precast beam, fixing them at their position in the slab and then reinforcing the upper part of it and pouring the concrete in the upper 15 cm of the beam together with the slab. (See figure 8). [1,10]

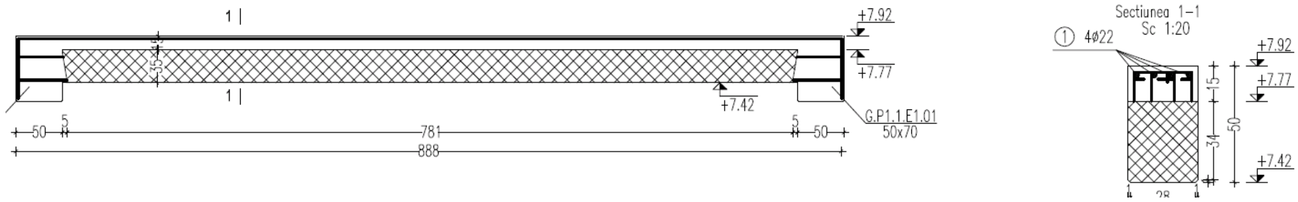



Figure 8. Slab beams: precast and monolithic parts

## 3. Precast stepped beams to sustain the inclined tiers of the grandstand

They were used on the entire structure.

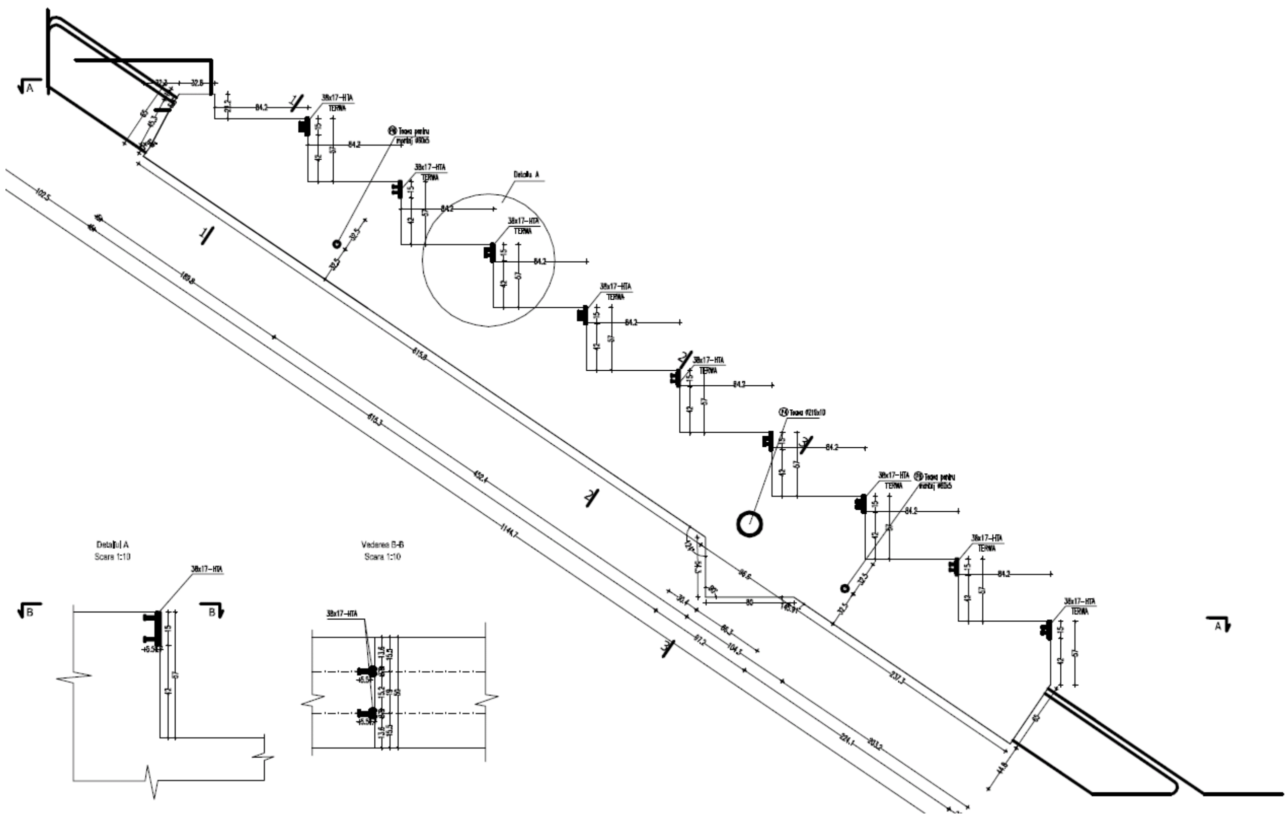


Figure 9. Stepped beams

Their dimension is from 40x65cm to 50x70cm, depending on the length of the opening and they are linked together with the structure by means of monolithic joints. At a distance of approximately 3 meters from the lower part of the beam, a pinned joint is realized in correspondence of the column. (See figure 10) The hinge is made up of a steel pipe and it was realized in order to have continuity with the cantilever inclined beam.

Figure 10. Hinged link between the inclined beam and the column

The montage scheme is presented in the below picture.

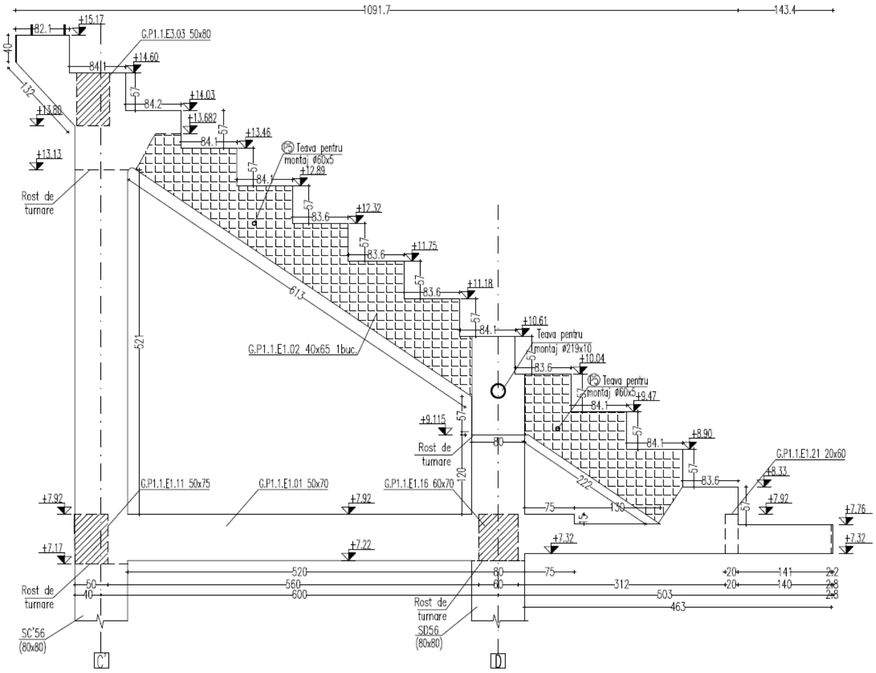
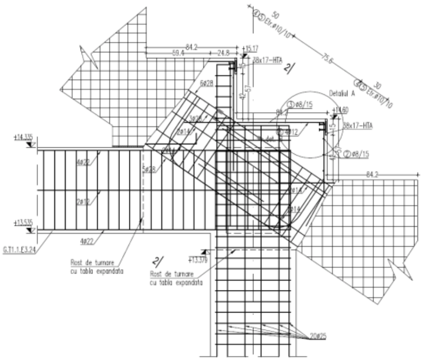
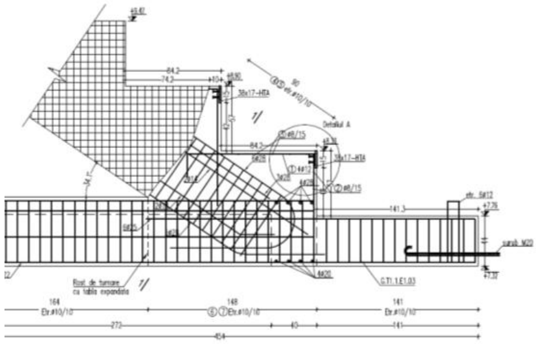



Figure 11. Montage scheme

The joints are realized like in the below figures:

in the design:



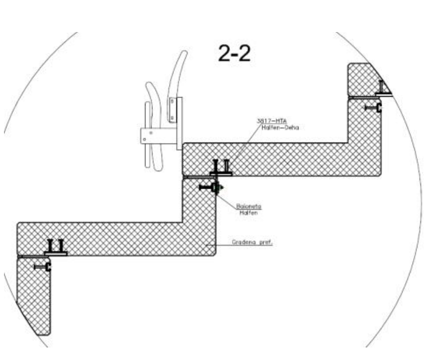


and on site:

Figure 12. The monolithization of the joints of the inclined stepped beam

## 4. Precast tiers of the grandstand

The tiers are the elements on which the chairs are placed. (See figure 13)



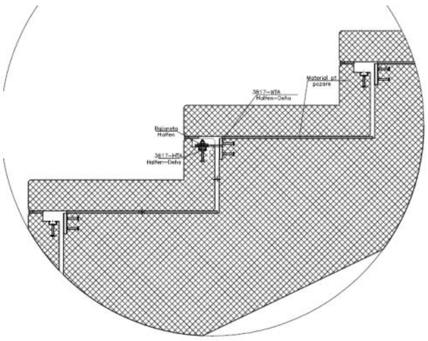



Figure 13. Cross section of the tiers on the opening and on the support zone, respectively

The dimensions of the section of such an element are presented in the figure below. This type of element has a thickness of 15cm.

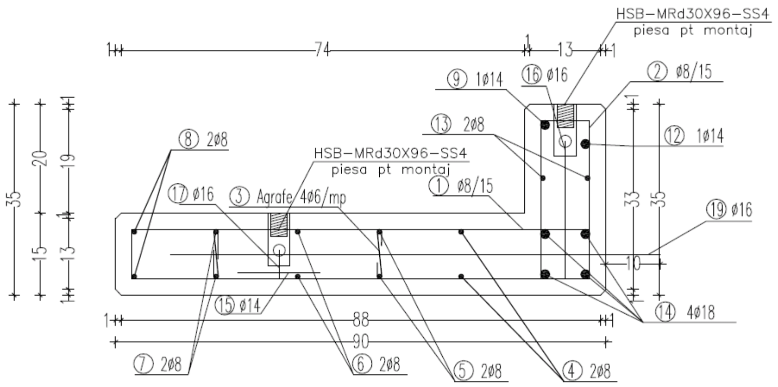



Figure 14. Cross-section dimensions of the tiers

These elements are fixed on the inclined stepped beams by means of HALFEN-DEHA type connectors and they are also linked together in the opening zones by two connectors. [1,10]

## 5. Other precast elements:

- parapets:

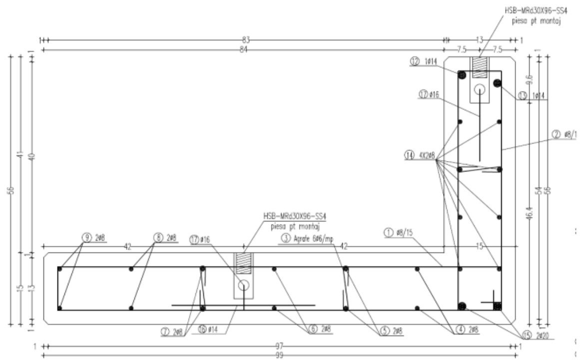


Figure 15. Parapet as designed

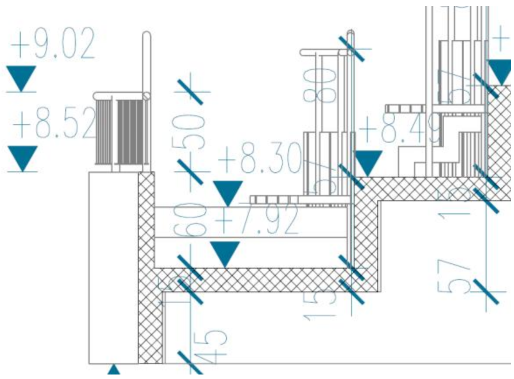



Figure 16. Position of the parapet

- Precast stairs and stair stepped beams between floors:

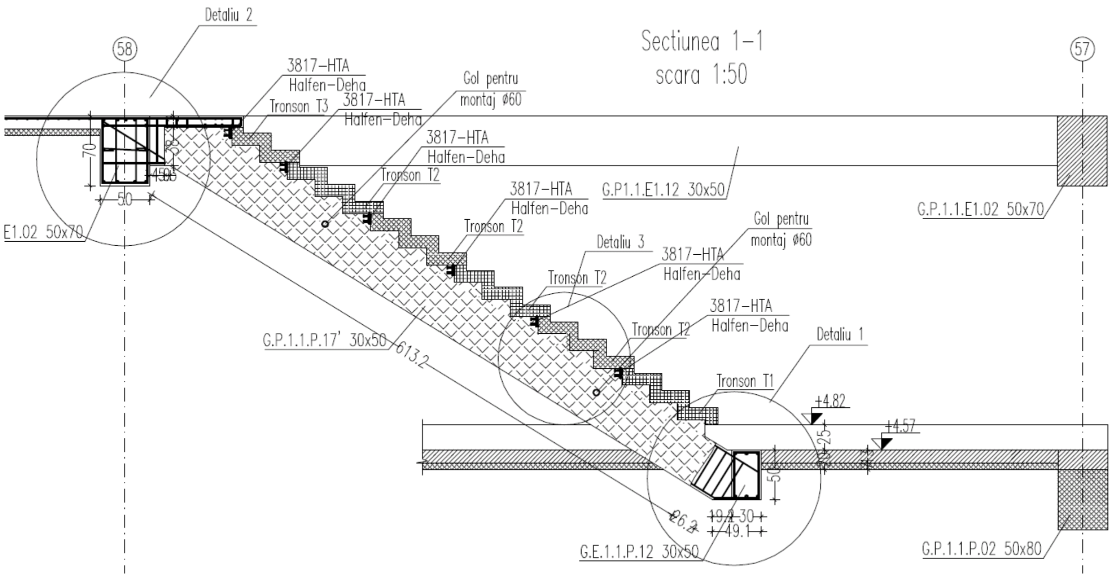



Figure 17. Montage scheme for the precast stairs that links different floors

## 6. Conclusions

By using the precast technology, we obtained a very good quality of the reinforced concrete elements. By reducing the formwork, the reinforcing and concreting works on site we obtained an important reduction of the total time of execution.

## 7. References

[1] PETRINA M., SOCACIU N., PETRINA B.,RUS P.,KOPENETZ L., POP G. I., CATARIG A., PACURAR V., HULEA R., ZOICAS M. R., *„Probleme privind conceperea structurilor de rezistenta a stadioanelor”,* Construieste cu STEEL Seminar 2010, 21 mai 2010, pp. 29, Cluj-Napoca, Romania. ISBN 978-973-713-271-0

[2] SR EN 1991-1-7 *„Actiuni generale – Actiuni accidentale”*

[3] SR EN 1991-1-4 *„Actiuni generale – Actiuni ale vantului”*

[4] V. Pacurar, P. Moga, S. I. Gutiu, *„Bazele proiectarii elementelor din otel”*

[5] M. Petrina, N. Socaciu, T. Petrina, Hulea Radu, Zoicas M. Radu, *„Numerical attempts and optimizing the structure of the roof stadium in Cluj Napoca”,* Sanghai, 8-12 noiembrie 2010;

[6] PETRINA M., SOCACIU N., RUS P., POP G. I., IONESCU A., PETRINA T., HULEA R. *„Structural concepts of the municipal stadium from Cluj-Napoca”,* Civil Engineering – Towards a Better Environment and The concrete Future, Conferince Published by CI-Premier Conference 2009, 17-19 June 2009, pp. 207, Coimbra, Portugal. ISBN 978-981-08-3242-1

[7] SOCACIU N., PETRINA T., HULEA R., Moldovan I. F., Vidrean R., Mojolic C., *„Conceptual structural solutions for the resistance of the stadium roof*” Building Services, Mechanical and Building Industry Days, 15-16 October 2009, pp. 103, Debrecen, Hungary. ISBN 978-963-473-315-7

[8] Dan Mateescu, Ioan Caraba, *Construcții Metalice,* Editura Tehnică 1980

[9] P. Siminea, L. Negrei, *Construcții metalice. Calcul prin metoda starilor limită,* Editura Didactică si Pedagogică 1982

[10] Mircea Petrina, Socaciu Nicolae, *Proiect tehnic Stadionul Municipal la standarde internaționale, Cluj-Napoca,* UTCN, 2010.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# The Steel Structure for the Roof of the City Stadium of Cluj-Napoca

Mircea Petrina [*1], Nicolae Socaciu[2], Tudor Petrina[3], Monica Nicoreac[4], Bianca Parv[5]

[1,2,3,4,5] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*This study presents the steel elements of the roof' structure of the City Stadium of Cluj-Napoca. These consist in main cantilever trusses fixed on reinforced concrete columns, longitudinal 2D trusses and bracings. The first part of the study includes introductory elements about the steel structure of the stadium and in the second chapter the describing of the design of each type of elements was presented. Chapter 3 is about the composition of the elements: main cantilever trusses, longitudinal trusses – simple and with stiffen task and bracing systems.*

## Rezumat

*Aceasta lucrare descrie elementele structurii metalice a acoperisului Stadionului Municipal din Cluj-Napoca. Acestea s-au realizat din ferme principale plane fixate pe stalpi din beton armat, pane alcatuite din ferme plane si contravantuiri.Prima parte a lucrarii cuprinde o introducere in structura metalica a acoperisului stadionului, in capitolul 2 se descrie dimensionarea structurii metalice. Capitolul 3 se refera la alcatuirea elementelor si anume: grinzile principale, panele simple, panele cu rol de rigidizare si sistemele de contravantuire. Concluziile sunt prezentate in capitolul 4.*

**Keywords:** stadium, roof, steel, trusses, bracing.

## 1. Introduction

The City Stadium of Cluj-Napoca was designed for a number of 30500 sits approximately; it will be used for football, athletics and cultural events. (See figure 1)
The structure of the grandstand is made up of reinforced concrete cast on site or precast, having 3 to 5 floors. (See figure 2)
Under the grandstand (the tiers), there are spaces designed for the functionality of the stadium. At the underground level, parking and different other spaces were designed. [1,10]

[*] Corresponding author: E-mail address: Mircea.Petrina@mecon.utcluj.ro

Fig. 1 City Stadium of Cluj-Napoca

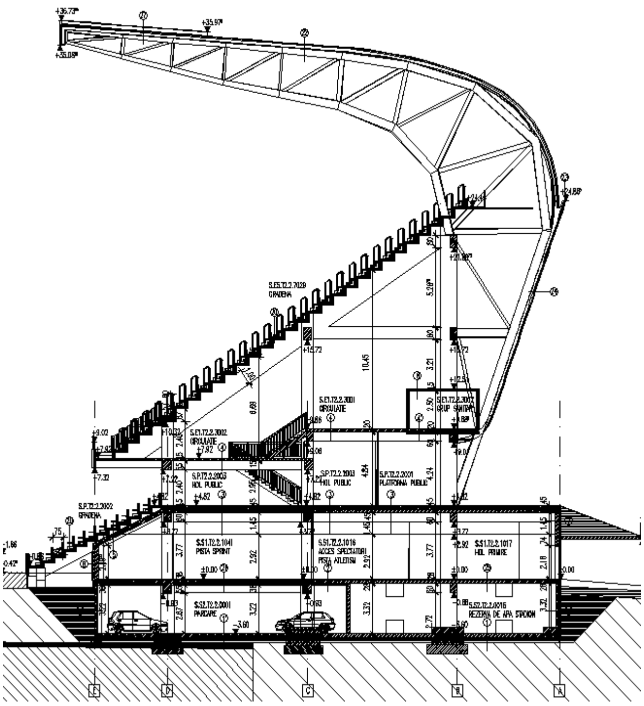


Fig. 2. Transversal section of the grandstand T2

The sits are covered on a percentage of 90% with a roof cover made of steel sheets and partially policarbonate sheets. This cover is sustained by a steel structure made of main steel cantilever trusses, fixed by means of bolts in 4 points on the reinforced concrete structure. On the longitudinal direction there are trusses supported on the main cantilever trusses. In the top and in the supports zones of the main trusses, the longitudinal trusses were designed with a height equal the height of the main truss in order to stiffen the structure. [1,10]

In order to insure the spatial rigidity of the structure, bracing systems were applied in the plane of the roof and between elements. (See figure 3) [1,10]

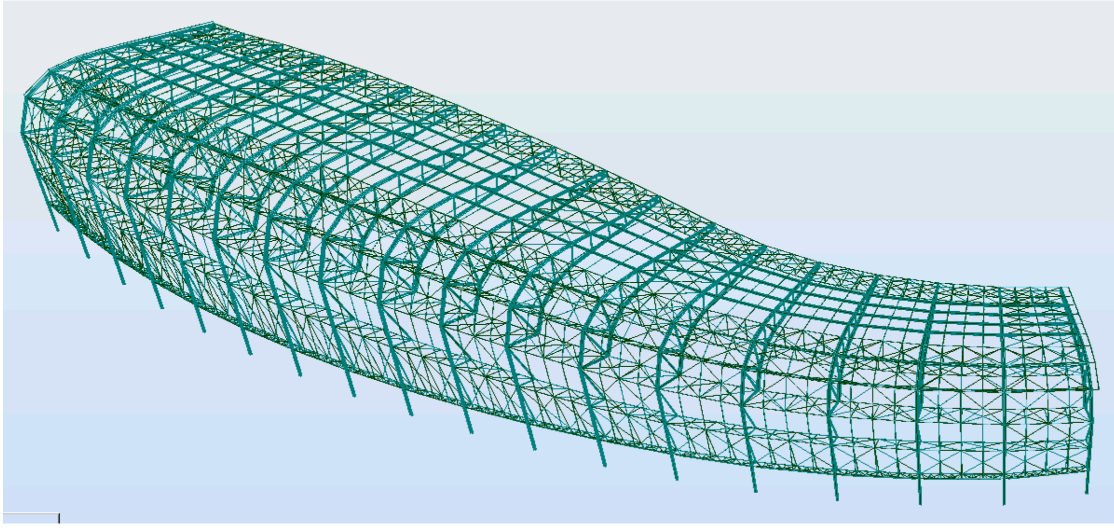


Figure 3. Steel structure for the grandstand T2

The structure is divided for the limiting of the stresse due to temperature variation, the maximum length of the sections being of approximately 60m, in the correspondence of the lower reinforced concrete structure. The expansion joints were not realized by doubling the structure's elements but by simply supporting the longitudinal trusses of one opening (situated above the expansion joint of the reinforced concrete structure). [10,11]

## 2. The design of the steel structure

The structure was computed to undertake the folowing actions:

- the dead load of the structural elements;
- the dead load of the finite cover;
- technological loads of the ligthing installations, sound system and also from the display panel;
- snow load;
- wind load wich can act from up towards down or viceversa.

The maximum internal forces were computed on the spatial (3D) structure obtained from the most unfavorable loading combinations and therefore the sections of the bars were computed from resistance conditions and stability conditions, too. A verification of the structure was realized under an exceptional situation, i.e. when a steel main truss collapses; this effect was studied on the rest of the structure.

Due to the fact that the internal forces are rather big, and the elements are situated outdoors, a superior quality steel was chosen, having the folowing characteristics: S355,J2 by ensuring the minimum yield energy of 27J at a minimum temperature of -31$^{o}$C, possible temperature in the area of the stadium. [1,11]

## 3. The composition of the elements

### *3.1. Main cantilever trusses*

The main trusses are 2D trusses, having a length of the cantilever between 15 – 33m (See figure 4), according to the width of the construction to be covered. Every main truss is fixed in 4,5 points on the reinforced concrete columns of the main structure. [1,10]

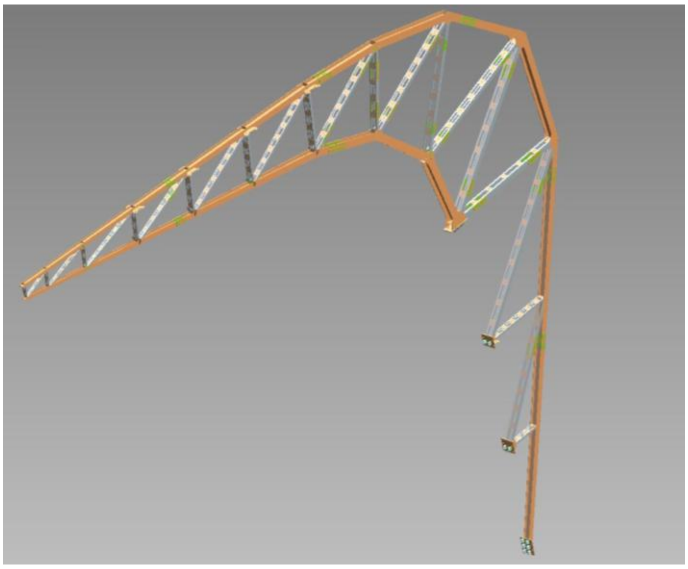


Figure 4. The main truss

The links are realized by bolts of anchoring, embedded in the concrete to which the base plates of the truss' nodes are linked. (See figure 5). These plates have the thickness of 30mm, on them the bars of the main truss are welded. The shears in the supports are undertaken by "spurs" having an I type section fixed on the base plate which enter the holes left in the reinforced concrete column. After the putting in place of the main truss, the space between the base plates and the reinforced concrete and the space between the "spurs" and the holes are injected with an expandable cementitious mix to ensure the working together of elements. (See figure 5) [1,10]

Figure. 5 Base plates with "spurs"

The main truss extends on a portion of the external wall of the stadium, and the height of the truss is variable, depending on the values of the internal forces.
The longitudinal parts of the trusses are made of an H beam produced from welding together steel sheets. The section is variable continuously on the horizontal and on the vertical direction, too.
The diagonals and the transversal elements are made of two U profiles, stiffen together by steel plates. The distance between the two profiles is variable, being equal with the width of the longitudinal element in that node. [1,10] (See figure 6)

Figure 6. Main truss node

In these nodes, the longitudinal elements of the truss become larger in order to become gusset plates to weld on the diagonals and the verticals. The end of the U profiles are processed in order to be welded head-to-head to the gusset plates from the nodes, and the longitudinal elements of the truss will be welded by welding relief seams on the interior face of the node.

On the main trusses a series of pieces are welded in order to fix the other elements of the steel structure. A special attention was made to the head-to-head welding of the steel sheets from which the longitudinal elements of the truss are made, for these weldings to have the same bearing capacity as the basic pieces. (See figure 6)

The superior (upper) longitudinal element of the main truss is fixed against lateral buckling by the longitudinal trusses placed at this level. For fixing the lower longitudinal element of the main truss, inclined ties were used, linked on the longitudinal trusses nodes and in the top and support zones, linked on the longitudinal trusses with bracing task, that have the height equal the height of the main truss, being fixed on the main truss on both upper and lower parts.

Because the length of the trusses is rather great, they were made of transportable sections, the joints between them were designed as bolted joint with pre-stressed bolts. (See figure 7)


Figure 7. Main truss of P2

The main trusses having a big opening and having the height over 3.5m, a sectioning on the vertical direction was also made by diagonals and verticals. (See figure 8)

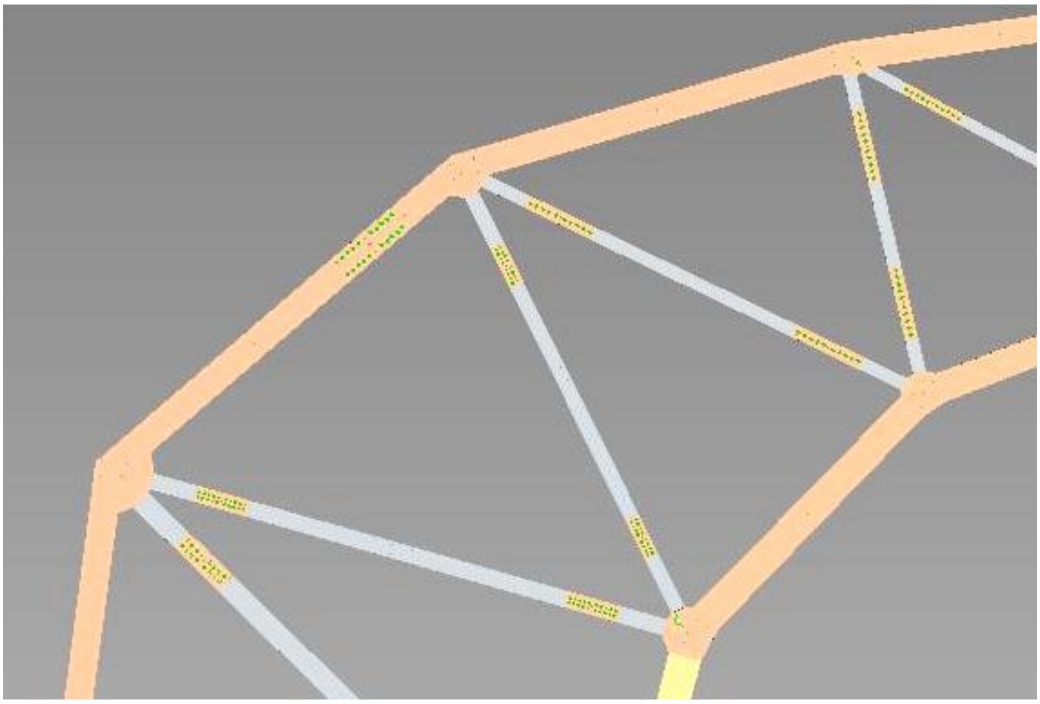


Figure 8. Main truss with a height over 3.5m

### *3.2. Simple longitudinal trusses*

In the central zone of the main trusses' opening, the longitudinal trusses are made of 2D trusses, (see figure 9) hinged on the upper elements of the main trusses. These elements have variable length due to the in-plane shape of the roof, having values between 8 to 10m.

Figure 9. The longitudinal trusses

Figure 10. Connection of the longitudinal truss on the main cantilever truss

All the elements of this longitudinal truss are made of circular pipes, cut in nodes by the line of intersection and welded by relief welding seams. In the bearing of the longitudinal truss, the elements are fixed in a horizontal gusset plate and a vertical rigidity (See figure 10), which may stay on the supports welded on the main truss. The joint is made by prestressed bolts because the longitudinal trusses undertake the longitudinal forces in the 3D structure of the roof.
The upper fixing of these trusses against lateral buckling is done by the elements that sustain the finite cover of the roof. Because the wind pressure from below to above may exceed the weight of the cover of the roof, the lower elements of the trusses my be also compressed and their transversal fixing is made by prestressed ties, linked on the supperior elements of the adjacent longitudinal trusses. (See figure 11)

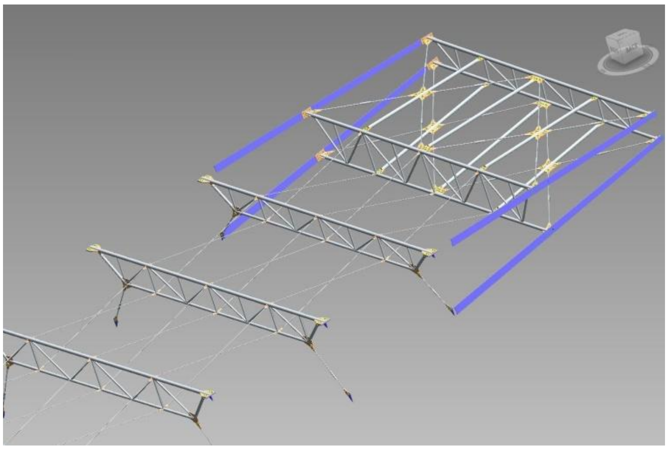


Figure 11. Elements of one opening

On the first node of the lower element of the longitudinal truss, the ties that are used for transversal fixing of the lower element of the main truss are linked. [1,10] (See figure 11)

### 3.3. Longitudinal trusses with stiffen task

The longitudinal trusses situated on the support zone of the main truss, that have also the role to fix the lower element of the main trusses, and role in the spatial longitudinal stiffening of the roof structure have the folowing characteristics: the longitudinal elements are made from circular pipes, fixed by end plates and bolts to the main truss' longitudinal elements. The verticals of these trusses are also made by circular pipes (See figure 12) and the diagonals are in X and are made from round steel, hinged by bolts in the nodes and with a sleeve having a left-right thread. (See figure 12) This system was adopted in order to have a more facile labor and to compensate any assembling error.

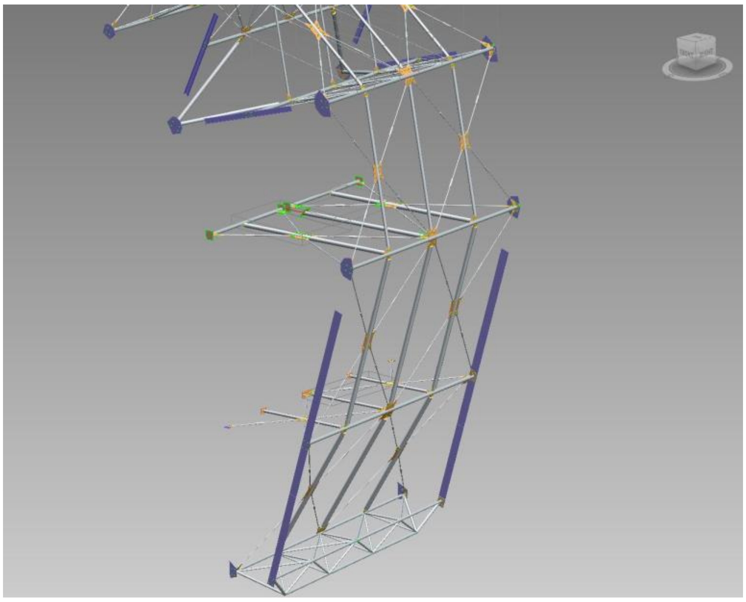


Figure 12. The elements of one opening from the vertical zone

### 3.4. The bracing systems

In order to ensure the spatial stiffness of the structure, bracing systems were provided in the roof plane and also between the main trusses.

These systems undertake the horizontal wind or seismic loads or the tendence of some elements of the main truss to buckle.

Two transversal bracing for every opening, in the plane of the cover of the roof were designed, they were placed in the first and last opening of each section, adjacent to the hinged opening above the expansion joint. (See figure 3, 12) The internal forces taken by these transversal bracings are transmitted to the reinforced concrete structure by means of bracings between the main trusses, and they are realized by trusses created for this task. [1,10]

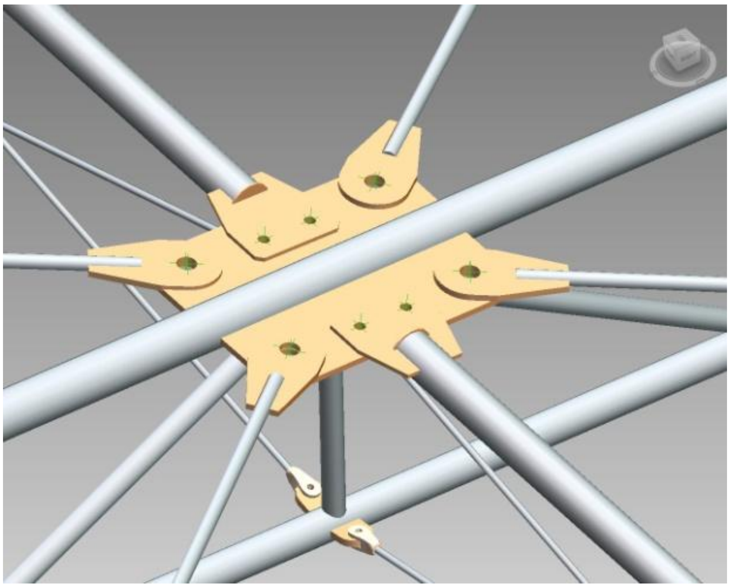


Figure 13. Bracings – truss node

On the longitudinal direction, each section has bracings at the level of the finite cover of the roof, between the longitudinal trusses from the top of the main truss and between the special longitudinal trusses from the support zone, and these bracings are placed on the entire length of the steel structure section. These bracings, together with the transversal ones and with the longitudinal elements of the trusses assure the planar rigidity of the steel structure of the roof. The longitudinal elements of the bracings are in fact the longitudinal elements of the trusses, the verticals are made from circular pipes bolted together in the nodes, (see figure 13) the diagonals are disposed in X, being made of round steel bars hinged in the nodes and stressed with sleeves. This constructive solution permits to compensate any fabrication error.

## 4. Conclusions

The structural system is relatively simple, being suitable for the geometry and the composition of the stadium. The elements from which the structure is made of may be executed in the workshops, all the assembling operations on site are designed with bolts. The assembling operations of the structure is also simple and there is no need for temporary sustaining the elements. The systems with longitudinal trusses and tie bracing permits to compensate assembling errors and also to introduce initial stresses by prestressing which ensures a better co-working of the elements.

## 5. References

[1] PETRINA M., SOCACIU N., PETRINA B.,RUS P.,KOPENETZ L., POP G. I., CATARIG A., PACURAR V., HULEA R., ZOICAS M. R., *„Probleme privind conceperea structurilor de rezistenta a stadioanelor”,* Construieste cu STEEL Seminar 2010, 21 mai 2010, pp. 29, Cluj-Napoca, Romania. ISBN 978-973-713-271-0

[2] SR EN 1991-1-7 *„Actiuni generale – Actiuni accidentale”*

[3] SR EN 1991-1-4 *„Actiuni generale – Actiuni ale vantului”*

[4] V. Pacurar, P. Moga, S. I. Gutiu, *„Bazele proiectarii elementelor din otel”*

[5] M. Petrina, N. Socaciu, T. Petrina, Hulea Radu, Zoicas M. Radu, *„Numerical attempts and optimizing the structure of the roof stadium in Cluj Napoca”,* Sanghai, 8-12 noiembrie 2010;

[6] PETRINA M., SOCACIU N., RUS P., POP G. I., IONESCU A., PETRINA T., HULEA R. *„Structural concepts of the municipal stadium from Cluj-Napoca”,* Civil Engineering – Towards a Better Environment and The concrete Future, Conferince Published by CI-Premier Conference 2009, 17-19 June 2009, pp. 207, Coimbra, Portugal. ISBN 978-981-08-3242-1

[7] SOCACIU N., PETRINA T., HULEA R., Moldovan I. F., Vidrean R., Mojolic C., *„Conceptual structural solutions for the resistance of the stadium roof*” Building Services, Mechanical and Building Industry Days, 15-16 October 2009, pp. 103, Debrecen, Hungary. ISBN 978-963-473-315-7

[8] Dan Mateescu, Ioan Caraba, *Construcții Metalice,* Editura Tehnică 1980

[9] P. Siminea, L. Negrei, *Construcții metalice. Calcul prin metoda starilor limită,* Editura Didactică si Pedagogică 1982.

[10] Mircea Petrina, Socaciu Nicolae, *Proiect tehnic Stadionul Municipal la standarde internaționale, Cluj-Napoca,* UTCN, 2010.

[11] MOGA Petru, CAMPIAN Cristina Mihaela, PETRAN Ioan, MOGA Catalin, PETRINA Bogdan, URIAN Gabriel, *Proiectarea elementelor metalice : calcul dupa SR EN 1993-1, Cluj-Napoca,* UTCN, 2008.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Similitude Theory and Applications

Monica P. Nicoreac [*1], Bianca R. Pârv[1], Mircea Petrina[2], Tudor Petrina[3]

[1,2,3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering, Department of Structural Mechanics*
*15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*Experimental techniques are vital for the development and validation of new analytical models and they are used extensively to determine complex loadings such as wind and earthquake, but also to analyse the behavior under these loading conditions. This paper presents some aspects regarding dimensional analysis and the Buckingham theorem. Similitude requirements for both static and dynamic loadings are synthetized and a simple example is presented for a 5-story plane steel frame loaded with static forces.*

## Rezumat

*Încercările experimentale sunt esențiale pentru dezvoltarea si validarea modelelor analitice noi dar sunt folosite tot mai mult si pentru determinarea încărcărilor complexe cum sunt cele date de vânt sau seism. Totodată se poate analiza comportamentul structurilor sub aceste încărcări. Lucrarea conține cateva aspecte legate de analiza dimensională si teorem Buckingham, iar legile de modelare pentru încărcări statice dar si dinamice sunt prezentate sintetic. Un exemplu simplu pentru un cadru metalic plan cu 5 nivele acționat de forțe orizontale este de asemenea prezentat.*

**Keywords :** dimensional analysis, Buckingham theorem, similitude requirements, experimental models.

## 1. Introduction

Experimental models have an important role in the structural engineering. Tests made on full-scale models can often be uneconomical or impractible. Reduced-scale models are a good solution for studying various phenomena by being economic solution to full-scale tests. In order to carry out any experimental test on a structural model it is important to have a good inside on the phenomen studied.

The Buckingham ($\pi$) theorem is a key theorem for dimensional analysis and it states : *any dimensionally homogeneous equation involving certain physical quantities cand be reduced to an equivalent equation involving a complete set of independent dimensionless products*, [2].

[*] Corresponding author: Tel. : 0040729859618 (Monica P. Nicoreac)
E-mail address: Monica.NICOREAC@mecon.utcluj.ro

Let us consider a certain physical phenomen which depends on n physical variables, that can be put in the form of a meaningful equation, [1]:

$$f ( A_1, ...,A_k, B_{k+1}, ..., B_n ) = 0 \tag{1.1}$$

where $A_1...A_k$ are basic physical variables, and $B_{k+1}...B_n$ secondary physical variables.

The above equation which describes the phenomena studied is dimensionally homogeneous, that is all its terms have the same dimension (the equation is valid regardless of the units we choose to measure the physical variables).

If we consider $\alpha_1, \alpha_2, ..., \alpha_k$ the units for measuring the basic variables, then we can rewrite equation (1.1) with the numerical values (quantities) of the physical variables $A_1,..,A_k$, $B_{k+1},..., B_n$ in terms of this measuring units, [1]. The units of the secondary variables, $\beta_{k+1}, ..., \beta_n$ cand be expressed as a combination of the basic units $\alpha_1, \alpha_2, ..., \alpha_k$.

$$f ( a_1, ...,a_k, b_{k+1}, ..., b_n ) = 0 \tag{1.2}$$

Equation (1.1) can be expressed in the form of the following equation of dimensionless products [1]:

$$\theta ( \pi_1, \pi_2, ..., \pi_{n-k}) = 0 \tag{1.3}$$

where the terms $\pi_1,..., \pi_{n-k}$ are the dimensionless products constructed from the numerical values $a_1, ...,a_k, b_{k+1}, ..., b_n$, [1].

$$\pi_1 = \frac{b_{k+1}}{\prod_{i=1}^{i=k} a_i^{m_i}}, \dots, \pi_{n-k} = \frac{b_n}{\prod_{i=1}^{i=k} a_i^{p_i}} \tag{1.4}$$

A dimensional matrix is defined as being composed of a number of columns equal to the number of physical variables from the equation (1.1) and of a number of rows equal to the number of basic units. The number of dimensionless products ($\pi$- products) is the number of physical variables minus the rank of their dimensional matrix.

The theory of structural models is based on the fact that the physical laws of nature are the same when applied to the prototype or model. Complete similarity between model and prototype is achieved when the dimensionless products $\pi_1, ..., \pi_{n-k}$ are identical both on model and prototype, [1].

$$\theta_p / \theta_m =1 \rightarrow \pi_{1,p} = \pi_{1,m}, \dots , \pi_{n-k,p} = \pi_{n-k,m} \tag{1.5}$$

From the above equations we can deduct the scale factors for the physical variables, which can be fundamental scale factors ( for the basic dimensions) or secondary scale factors (for the secondary dimensions).

In the field of static structural analysis there are 2 fundamental scale factors : $S_L$ for linear dimension and $S_m$ for mases (or $S_F$ for force), and in dynamics one more fundamental scale factor is added, St for time. Scale factors are multipliers that convert the quantities from the model "$i_m$" into corresponding quantities for the prototype "$i_p$", [3].

$$i_p = S_i \cdot i_m \tag{1.6}$$

There are some difficulties in obtaining complete similarity (using a material with the same Poisson's ratio for both model and prototype, discrete loading that replaces continuous loading), and we need to relax the restriction $\theta_p / \theta_m =1$. If the difference between the value 1 and the actual value of the ratio $\theta_p / \theta_m$ can be willingly neglected, then we have a model with first-order similarity. Distorted models are those in which the resulting ratio $\theta_p / \theta_m$ is unknown. For example if the Poisson's ration of the model is not the same with that of the prototype, then this type of disstorsion will not influence the models stresses, reactions or bending moments, but it will distort it's strains.

Structural models can be either elastic models when only the elastic response of the structures is required, or strenght models when we want to load it until failure. Wheter we want to find the actual strain displacements for a given load or only the influence diagrams, we have different types of models.

Elastic models have geometric similarity with the prototype and are made of elastic, homogeneous materials not necessary similar to the prototype's material. This models will predict only the elastic behavior of the prototype and it cannot be used when we want to study the nonlinear behavior of postcracking concrete or of postyielding structural steel.

Strength models (replica models) are used for studying the nonlinear behavior until failure of the prototype. The material used for the replica model has to be similar to the one used in the prototype. This type of models are indirect models by definition, and they can be used for reinforced concrete structures as well as for steel and timber structures. The problem with these models is finding the right material to predict the behavior of the prototype until failure.

Direct models require that the prototype and model have geometric similarity and that they are loaded in a similar manner. We can measure the response directly on the model and by using the scale factors we get the response needed on the prototype. Direct models can be used even for analysis in the postelastic range.

Indirect models are used only when we are looking for the influence lines from the model in order to calculate the prototype's response to certain loads. The loads applied on the model have no direct relation to the ones on the prototype. Geometric ressemblance is also not required.

Wind effect models can be classified as rigid models for measuring the wind pressure on the structure or fluidelastic models which simulate the geometric and rigidity properties of the structure in order to measure the stresses and displacements caused by wind action.

Dynamic models are used for analysing the effects of vibrations and dynamic loads on structures. Earthquake loads, wind loads, impact or blasts are the dynamic loadings of interest for experimental testing, [2].

## 2. Similitude requirements for static elastic modelling

If we consider an elastic system made of a homogeneous and isotropic material with modules of elasticity E and G, specific weight γ and Poisson ratio μ, the characteristic equation that describes the elastic system is, [4] :

$$f(\sigma, l, F, E, \gamma, \delta, \tau, G, (r\varphi), \mu) = 0 \tag{2.1}$$

where l – linear dimension, δ – linear strain, (rφ) – angular displacement, F – characteristic force, and σ, τ – (normal and shear) stresses.

Using the π theorem we can derive the following dimensionless terms, [4] :

$$\theta\left(\frac{F}{\sigma l^2}, \frac{E}{\sigma}, \frac{\gamma l}{\sigma}, \frac{\delta}{l}, \frac{\tau}{\sigma}, \frac{G}{\sigma}, \frac{(r\varphi)}{l}, \mu\right) = 0 \tag{2.2}$$

If Hooke's law is satisfied, then the dimensionless product $\frac{E}{\sigma} = \frac{\delta}{l}$ and $\frac{(r\varphi)}{l} = \frac{\tau}{G}$ , [4] :

$$\theta\left(\frac{F}{\sigma l^2}, \frac{E\delta}{\sigma l}, \frac{\gamma l}{\sigma}, \frac{G(r\varphi)}{\tau l}, \mu\right) = 0 \tag{2.3}$$

The scaling law for static elastic modelling can be resumed as, [4] :

$$S_\sigma = \frac{S_F}{{S_L}^2} \;;\; S_\delta = \frac{S_F}{S_L S_E} \;;\; S_\gamma = \frac{S_F}{{S_L}^3} \;;\; S_\tau = S_G \;;\; S_\mu = 1 \tag{2.4}$$

Similitude requirements for static elastic modelling can also be put in another form like in table 2.1, [2]. We can see that the stresses on the prototype are $S_E$ times larger than those on the model, while the strain on both model and prototype are the same. The loadings on the model are scaled by a large factor $S_E {S_L}^2$ .

Table 2.1 Similitude requirements for static elastic modeling

| Material Properties | Dim. | Scale Factors | Geometry | Dim. | Scale Factors | Loading | Dim. | Scale Factors |
|---|---|---|---|---|---|---|---|---|
| Stress $\sigma$ | $FL^{-2}$ | $S_E$ | Linear dimension l | L | $S_L$ | Concentrated load Q | F | $S_E {S_L}^2$ |
| Modulus of elasticity E | $FL^{-2}$ | $S_E$ | Linear displacement $\delta$ | L | $S_L$ | Unif. distrib. load w | $FL^{-1}$ | $S_E S_L$ |
| Poisson's ratio $\mu$ | - | 1 | Angular displacemnt | - | 1 | Pressure/surface load q | $FL^{-2}$ | $S_E$ |
| Specific Weight $\gamma$ | $FL^{-3}$ | $S_E/S_L$ | Area A | $L^2$ | ${S_L}^2$ | Moment M or torque T | FL | $S_E {S_L}^3$ |
| Strain $\varepsilon$ | - | 1 | Moment of inertia I | $L^4$ | ${S_L}^2$ | Shear force V | F | $S_E {S_L}^2$ |

## 3. Similitude requirements for reinforced concrete and structural steel models

For reinforced concrete structures it is difficult to model the inelastic behaviour and the failure mode. Also the composite nature of concrete needs to be taken in account, especially the interaction between concrete and reinforcement. Therefore we need two materials to simulate the properties of the prototype material : model concrete and model reinforcement. Steel has been shown to be the most viable material to model the reinforcement in model concrete [2]. This will lead to a unity stress scale factor ($S_\sigma$ = $S_E$ =1) and the rest of the scalling factors for material properties as according to table 3.1, and scale factors for geometry and loading identical to those in table 2.1. If the model concrete doesn't have $S_\sigma$ = $S_E$ =1, distorted models have to be used.

Another material difficult to model is structural steel. Because of its property of yielding and strain hardening, many authors recommend using structural steel for both prototype and model for best results.

| Material Properties Concrete | Dim. | Scale Factors | Material Properties Reinforcement | Dim. | Scale Factors | Material Properties Structural Steel | Dim. | Scale Factors |
|---|---|---|---|---|---|---|---|---|
| Stress $\sigma_c$ | $FL^{-2}$ | $S_\sigma$ | Stress $\sigma_r$ | $FL^{-2}$ | $S_\sigma$ | Stress $\sigma_s$ | $FL^{-2}$ | $S_\sigma$ |
| Modulus of elasticity $E_c$ | $FL^{-2}$ | $S_\sigma$ | Reinforcement strain $\varepsilon_r$ | - | 1 | Modulus of steel $E_s$ | $FL^{-2}$ | $S_\sigma$ |
| Poisson's ratio $\mu_c$ | - | 1 | Modulus of elasticity $E_r$ | $FL^{-2}$ | $S_\sigma$ | Poisson's ratio $\mu_c$ | - | 1 |
| Mass density $\rho_c$ | $FL^{-3}$ | $S_\sigma/S_L$ | Bond stress u | $FL^{-2}$ | $S_\sigma$ | Mass density $\rho_c$ | $FL^{-3}$ | $S_\sigma/S_L$ |
| Strain $\varepsilon_c$ | - | 1 | | | | Strain $\varepsilon_c$ | - | 1 |

Table 3.1 Scale factors for reinforced concrete models and structural steel models

## 4. Similitude requirements for dynamic modelling of structures

The similitude requirements for dynamic relationships between model and prototype depend on the material and geometric properties of the structure, and on the type of loading acting on the structure. If we want to achieve dynamic similitude we have to have geometric similarity together with kinematic and dynamic similarity. The fluid flow of both model and prototype are similar from the kinematic point of view, if the fluid streamlines are similar and if they have similar time rates of changing motions. Dynamic similarity is achieved when the two fluid flows have kinematic similarity and mass distribution similarity, [4].

Let us consider an elastic system made of a homogeneous and isotropic material with the following governing variables and their units : lenght l (L), force Q (F), modulus of elasticity E ($FL^{-2}$), Poisson ratio $\mu$, mass density $\rho$ ($FT^2 L^{-4}$), deflection $\delta$ (L), stress $\sigma$ ($FL^{-2}$), frequency *f* ($T^{-1}$) and acceleration g ($LT^{-2}$). Using the Buckingham $\pi$ theorem we can determine the dimensionless $\pi$ terms, and the scaling law follows as shown in table 4.1, [2]:

$$f\,(\,l, Q, E, \mu, \rho, \delta, \sigma, f, g) = 0 \rightarrow \theta\left(\frac{\delta}{l}, \frac{\sigma}{E}, \frac{f^2 l}{g}, \frac{\rho g l}{E}, \frac{Q}{El^2}, \mu\right) = 0 \tag{4.1}$$

If the gravity forces can be neglected, then the time scale will be $S_L$ and the frequency scale will be $S_L^{-1}$, which implies higher frequencies for the model. There are two important models for dynamic testing: the fluidelastic model for wind tunnel testing and the dynamic model for shaking table testing.

Table 4.1 Similitude requirements for elastic vibrations

| Loading | Dim. | Scale Factors | Geometry | Dim. | Scale Factors | Material Properties | Dim. | Scale Factors |
|---|---|---|---|---|---|---|---|---|
| Force Q | F | $S_E S_L^2$ | Linear dimension | L | $S_L$ | Modulus of elasticity E | $FL^{-2}$ | $S_E$ |
| Time t | T | $S_L^{1/2}$ | Displacement $\delta$ | L | $S_L$ | Stress $\sigma$ | $FL^{-2}$ | $S_E$ |
| Gravit. acc. g | $LT^{-2}$ | 1 | Frequency *f* | $T^{-1}$ | $S_L^{-1/2}$ | Poisson's ratio $\mu$ | - | 1 |
| | | | | | | Specific weight $\gamma$ | $FL^{-3}$ | $S_E/S_L$ |

(a) Fluidelastic models are necessary for studying the wind effects on buildings with complex shapes. This elastic models are applicable for scales between 1/50 to 1/300. The modeling of the structure alone for testing is not enough for wind tunnel and needs to be completed with scaling of wind flow parameters like : velocity, pressure and roughness. Complete similarity of the atmospheric boundary layer is not possible to achieve and so the restrictions need to be relaxed ; it is sufficient to simulate the topographic relief, the roughness of the terrain and the temperature of the surface together with the simulation of mean and fluctuating velocity and temperature fields of the approaching flow [2]. One important aspect of modeling the wind effects on structures is the ability to study the interference effect of the surrounding and upstream buildings or other obstructions.

The fluidelastic model used in wind engeneering has to be designed according with some regulations, [5] :

- Mass modeling : complete similarity of inertia forces of the structures and of the flow;

$$\left(\frac{\rho_s}{\rho}\right)_{\text{model}} = \left(\frac{\rho_s}{\rho}\right)_{\text{prototype}} \tag{4.2}$$

Where $\rho_s$ , $\rho$ – structural bulk density and air density;

- Damping : similarity of damping forces requires that the critical damping ratio in a vibration mode is the same in the model and prototype ;

$$\xi_{\text{model}} = \xi_{\text{prototype}} \tag{4.3}$$

- Stiffness scaling : requires complete similarity between the forces which resist structure's deformation and between the inertia forces.

$$\frac{V_m}{V_p} = \left(\frac{E_{effm}}{E_{effp}} \times \frac{\rho_p}{\rho_m}\right)^{1/2} \tag{4.4}$$

where $E_{eff}$ – effective modulus, $\rho$ – air density, V – characteristic wind speed

$$\left(\frac{f_0 L}{V}\right) = \text{constant} \; - \text{Strouhal number} \tag{4.5}$$

Where $f_0$ – natural frequency of a mode of vibration;

(b) Earthquake modeling of structures

Because of the catastrophic nature of earthquakes, this type of loading needs to be taken seriously into consideration in the design of buildings. Replica models for shake table testing must satisfy both the Froude and Cauchy scaling requirements which implies the simultaneous replication of inertia, restoring and gravitational forces, [3].

- Froude number : $Fr = v^2/l \cdot g$
- Cauchy number : $Ca = \rho v^2/E$

Since the value of gravitational acceleration (g) must equal one, and from the dimensional analysis we get the dimensionless product of $S_a/S_g = 1$ (a is the imposed acceleration), the following scaling law is derived, [3] :

$$S_{E/\rho} = S_L \tag{4.6}$$

This is difficult to realize because it requires that the model material has large mass density or small modulus or even both. A good alternative is to increase the density of the structure with additional nonstructural material.

Table 3.2 Similitude requirements for fluidelastic and earthquake modeling

| | | Scale Factors | |
|---|---|---|---|
| Fluidelastic Model | Dim. | Reynold no. neglected | Froude no. neglected |
| Loading | | | |
| Force Q | $MLT^{-2}$ | $S_\rho S_L^{3}$ | $S_\rho S_L^{-1}$ |
| Pressure q | $ML^{-1}T^{-2}$ | $S_\rho S_L$ | $S_\rho S_L$ |
| Time t | T | $S_L^{1/2}$ | $S_L S_v^{-1}$ |
| Gravit.l acc. G | $LT^{-2}$ | 1 | 1 |
| Velocity v | $LT^{-1}$ | $S_L^{1/2}$ | $S_v$ |
| Geometry | | | |
| Linear dim. L | L | $S_L$ | $S_L$ |
| Displacement δ | L | $S_L$ | $S_L$ |
| Frequency ω | $T^{-1}$ | $S_L^{-1/2}$ | $S_v S_L^{-1}$ |
| Material prop. | | | |
| Modulus E | $ML^{-1}T^{-2}$ | $S_\rho S_L$ | $S_\rho S_L$ |
| Stress σ | $ML^{-1}T^{-2}$ | $S_\rho S_L$ | $S_\rho S_L$ |
| Poisson's ratio μ | - | 1 | 1 |
| Mass density ρ | $ML^{3}$ | $S_\rho$ | $S_\rho$ |

| | | Scale Factors | | |
|---|---|---|---|---|
| Earthquake modeling | Dim. | True Replica Model | Artif. Mass Sim. | Grav. forces neglected |
| Loading | | | | |
| Force Q | F | $S_E S_L^{2}$ | $S_L^{2}$ | $S_L^{2}$ |
| Pressure q | $FL^{-2}$ | $S_E$ | $S_E$ | 1 |
| Acceleration a | $LT^{-2}$ | 1 | 1 | $S_L^{-1}$ |
| Time t | T | $S_L^{1/2}$ | $S_L^{1/2}$ | $S_L$ |
| Gravit.l acc. G | $LT^{-2}$ | 1 | 1 | neglected |
| Velocity v | $LT^{-1}$ | $S_L^{1/2}$ | $S_L^{1/2}$ | 1 |
| Geometry | | | | |
| Linear dim. L | L | $S_L$ | $S_L$ | $S_L$ |
| Displacement δ | L | $S_L$ | $S_L$ | $S_L$ |
| Frequency ω | $T^{-1}$ | $S_L^{-1/2}$ | $S_L^{-1/2}$ | $S_L^{-1}$ |
| Material prop. | | | | |
| Modulus E | $FL^{-2}$ | $S_E$ | $S_E$ | 1 |
| Stress σ | $FL^{-2}$ | $S_E$ | $S_E$ | 1 |
| Poisson's ratio μ | - | 1 | 1 | 1 |
| Mass density ρ | $FL^{-4}T^{2}$ | $S_E/S_L$ | a * | 1 |
| Energy EN | FL | $S_E S_L^{3}$ | $S_E S_L^{3}$ | $S_L^{3}$ |

* $(g\rho l/E)_{model} = (g\rho l/E)_{prototype}$

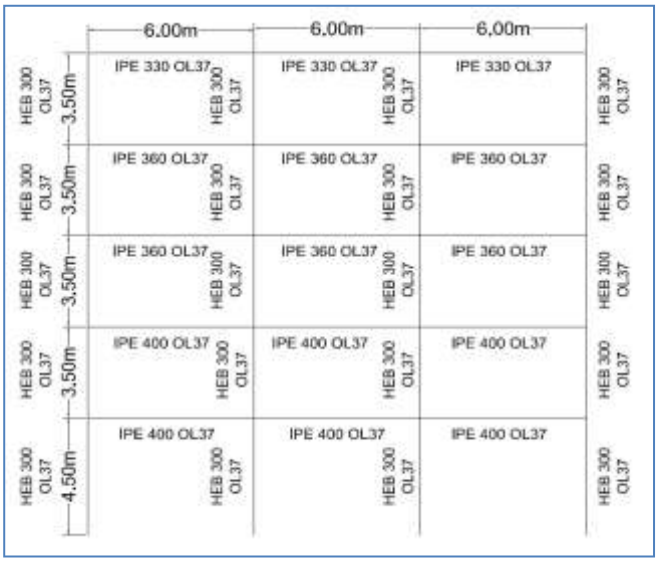
Another option of scaling laws which can be applied only to certain types of structures where stresses induced by gravity are negligible when compared to the stresses induced by earthquake. There is an additional requirement for the model and prototype materials to have identical properties so the failure conditions can be simulated, [3].

## 5. Exemplification

The building chosen for the exemplification purposes is a plane steel frame shown in figure 5.1 and described in “P100-1/ Building seismic design, volume 2-B. Comments and calculus examples”[6].

Using the similitude requirements from section 2, we will model the prototype at a scale of 1:4, using two types of models : the first one made from the same material as the prototype, and the second one made from timber.

The earthquake loadings are of particular interest, and the concentrated loads from the equivalent static analysis are shown in figure 5.2.



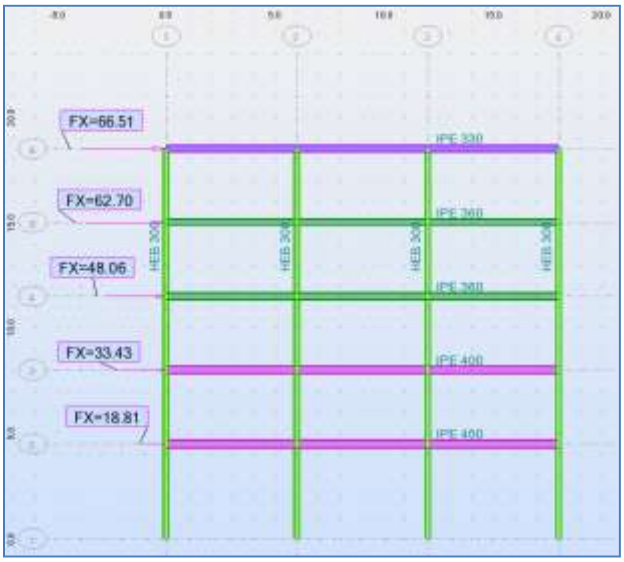




Fig.5.1 Plane steel frame

Fig.5.2 Static equivalent seismic forces

a. First we will consider the model made from the same material as the prototype (steel OL37), and the scaling relationships are according to section 2:

- geometric properties : $S_L = 4$ (linear dimension); $S_A = S_L^2 = 16$ (area);
  $S_I = S_L^4 = 256$ (moment of inertia); $S_W = S_L^3$(modulus of resistance);
  $S_\delta = S_L = 4$ (linear displacement)
- material properties : $S_E = S_\sigma = 1$ (stress); $S_\mu = 1$ (Poisson ratio);
  $S_\gamma = S_E/S_L = 1/4$ (specific weigth); $S_\varepsilon = 1$ (strain);
- loading : $S_F = S_E\, S_L^2 = 16$ (concentrated load)

According to the above relationships, the stresses in the model and prototype must be the same. This will be verified by scaling the concentrated loads with $1/S_F = 1/16$, and applying them on the reduced scale model using a structural analysis program. The result for both the model and prototype are given in the tables 5.1 and 5.2 for the first two columns. The scale for the bending moment is $S_M = S_F \cdot S_L = 64$.

Table 5.1 The concentrated loads aplied to model and prototype and the corresponding axial stresses for the columns in the first and second axes

| | Concentrated Loads | | Axial stresses in columns [kPa] | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Axis 1 | | Δ | Axis 2 | | Δ |
| | Model | Prototype | Model | Prototype | [%] | Model | Prototype | [%] |
| Level 1 | 1.18 | 18.81 | 9345.62 | 9351.6 | 0.064 | 782.35 | 810.61 | 3.486 |
| Level 2 | 2.09 | 33.43 | 6432.10 | 6440.69 | 0.133 | 397.32 | 420.32 | 5.472 |
| Level 3 | 3.00 | 48.06 | 3768.9 | 3778.17 | 0.245 | 172.24 | 186.77 | 7.78 |
| Level 4 | 3.92 | 62.70 | 1899.35 | 1905.14 | 0.304 | 70.14 | 79.65 | 11.94 |
| Level 5 | 4.16 | 66.51 | 588.65 | 591.12 | 0.418 | 23.96 | 28.22 | 15.1 |

Table 5.2 The linear displacements in the model and prototype for the first and second column and maximum bending moments in the first column.

| | Linear displacement [mm] | | | | | | Maximum bending moment | | |
|---|---|---|---|---|---|---|---|---|---|
| | Axis 1 | | $S_{\delta}=$ | Axis 2 | | $S_{\delta}=$ | Axis 1 | | $S_M=$ |
| | Model | Prototype | $\delta_p/\delta_m$ | Model | Prototype | $\delta_p/\delta_m$ | Model | Prototype | $M_p/M_m$ |
| Level 1 | 3.59 | 14.38 | 4.01 | 3.59 | 14.37 | 4.00 | 2.37 | 152 | 64.14 |
| Level 2 | 6.8 | 27.13 | 3.99 | 6.79 | 27.03 | 3.98 | 1.06 | 67.89 | 64.05 |
| Level 3 | 9.75 | 38.96 | 4.00 | 9.74 | 38.81 | 3.98 | 0.89 | 57.21 | 64.28 |
| Level 4 | 12.18 | 48.71 | 4.00 | 12.17 | 48.52 | 3.99 | 0.83 | 52.91 | 63.75 |
| Level 5 | 13.74 | 55.01 | 4.00 | 13.73 | 54.75 | 3.99 | 0.43 | 27.52 | 64.00 |

This model can be used not only in the elastic range of loading, but also for testing until failure, because the model and prototype are made of the same material and they have similar behavior and identical shapes of stress-strain diagrams.

b. The second type of reduced scale model will be considered to be made from timber with the following material properties of both model and prototype and the scaling relationships according to section 2:

$E_m = 9\cdot10^6$ kPa ; $E_p = 2.1\cdot10^8$ kPa $\rightarrow S_E = E_p/E_m = 23.33$

$\gamma_m = 6.38$ kN/m$^3$; $\gamma_p = 77.01$ kN/m$^3$ $\rightarrow S_\gamma = \gamma_p/\gamma_m = 12.07$

$S_L = S_\delta = 4$ ; $S_F = S_\gamma \cdot S_L{}^3 = 12.07\cdot4^3 = 772.54$; $S_\delta = S_F / S_L\cdot S_E = 772.54 / (4\cdot23.33) = 8.28$

$S_\sigma = SF / S_L{}^2 = 772.54 / 4^2 = 48.28$; $S_M = S_F\cdot S_L = 3090.16$

The result for the model and prototype are given in the tables 5.3 and 5.4.

Table 5.3 The concentrated loads aplied to model and prototype and the corresponding axial stresses for the columns in the first and second axes

| | Concentrated Loads [N] | | Axial stresses in columns [kPa] | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Axis 1 | | $S_\sigma=$ | Axis 2 | | $S_\sigma=$ |
| | Model | Prototype | Model | Prototype | $\sigma_p/\sigma_m$ | Model | Prototype | $\sigma_p/\sigma_m$ |
| L 1 | 24.35 | $18.81\cdot10^3$ | 193.99 | 9351.6 | 48.21 | 16.85 | 810.61 | 48.11 |
| L 2 | 43.27 | $33.43\cdot10^3$ | 133.56 | 6440.69 | 48.22 | 8.72 | 420.32 | 48.2 |
| L 3 | 62.21 | $48.06\cdot10^3$ | 78.27 | 3778.17 | 48.27 | 3.86 | 186.77 | 48.39 |
| L 4 | 81.16 | $62.7\cdot10^3$ | 39.46 | 1905.14 | 48.28 | 1.64 | 79.65 | 48.57 |
| L 5 | 86.09 | $66.51\cdot10^3$ | 12.24 | 591.12 | 48.29 | 0.58 | 28.22 | 48.66 |

Table 5.4 The linear displacements in the model and prototype for the first and second column and maximum bending moments in the first column.

| | Linear displacement [mm] | | | | | | Maximum bending moment | | |
|---|---|---|---|---|---|---|---|---|---|
| | Axis 1 | | $S_{\delta}=$ | Axis 2 | | $S_{\delta}=$ | Axis 1 | | $S_M=$ |
| | Model | Prototype | $\delta_p/\delta_m$ | Model | Prototype | $\delta_p/\delta_m$ | Model | Prototype | $M_p/M_m$ |
| Level 1 | 1.74 | 14.38 | 8.26 | 1.74 | 14.37 | 8.26 | 0.0492 | 152 | 3089.43 |
| Level 2 | 3.3 | 27.13 | 8.22 | 3.28 | 27.03 | 8.24 | 0.0221 | 67.89 | 3071.95 |
| Level 3 | 4.72 | 38.96 | 8.25 | 4.71 | 38.81 | 8.24 | 0.0185 | 57.21 | 3092.43 |
| Level 4 | 5.91 | 48.71 | 8.24 | 5.88 | 48.52 | 8.25 | 0.0171 | 52.91 | 3094.15 |
| Level 5 | 6.67 | 55.01 | 8.25 | 6.64 | 54.75 | 8.25 | 0.0089 | 27.52 | 3092.13 |

We can remark that in both cases the results from the analysis made with a commercial software are according to the similitude requirements described in section 2, as expected.

## 6. Conclusions

There have been considerable achievements in structural analysis theoretical methods in the last decades, but structural model testing is still a very reliable method for validating any new developed theory or to study complex, unconventional structures. Nowadays we can find improved testing techniques and instrument systems that give accurate and reliable results. Structural models were found to be educational when teaching the concepts of mechanics where they should be made as simple as possible in order to prove the concept which is studied.

Scaling requirements were presented both for static loading and dynamic loading. Nonlinear behavior of structures is better studied with physical models because they are more realistic than mathematical models.The choice of model material is very important, especially for the inelastic response and failure mode.

The example presents two case of physical reduced scale models, one made of steel and the other of timber. The similitude requirements were presented for the case where both models are reduced to a 1:4 scale. The model of timber can be used only to study the behavior of the model in the linear elastic range, while the model made of structural steel can be used for both elastic and postelastic ranges of behavior.

**AKNOWLEDGEMENT**

This paper was supported by the project "Doctoral studies in engineering sciences for developing the knowledge based society-SIDOC**”**contract no. POSDRU/88/1.5/S/60078, project co-funded from European Social Fund through Sectorial Operational Program Human Resources 2007-2013.

## 7. REFERENCES

[1] Al.A.Vasilescu, G.Praisler “*Similitudinea sistemelor elastice*”, ed. Academiei R.S.R , 1974.

[2] H.G.Harris, G.M.Sabnis “*Structural Modeling and Experimental Techniques*”,CRC Press, 1999.

[3] H.G.Harris “*Use of Structural Models as an Alternative to Full-Scale Testing*” in “*Full-Scale Load Testing of Structures*”, American Society for Testing and Materials, pp. 25-44, 1980.

[4] Al.Vasilescu “*Analiza dimensionala si teoria similitudinii*”, ed. Academiei R.S.R. , 1969.

[5] De Jack E. Cermak,N. Isyumov,American Society of Civil Engineers “*Wind tunnel studies of buildings and structures*”, ASCE Manuals and Reports on Engineering Practice, 1999.

[6] T.Postelnicu “*P100-1/proiectarea seismică a clădirilor.Volumul 2 - b. Comentarii si exemple de calcul*” , ctr. U.T.C.B. nr. 158/02.08.2005, M.T.C.T.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol.53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Results of the Romanian Researchers from Cluj Napoca Concerning High-Rise Structures

Bianca R. Parv[*1], Monica P. Nicoreac[2], Mircea Petrina[3], Tudor Petrina [4]

[1,2,3,4] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*In this article we intend to realize a study on high-rise structure's calculation methods nominated by researchers from Cluj-Napoca. Firstly we will focus on the shear walls-frame structure and after on the perimeter tube structures. We will present the results obtained using the calculation methods recommended by professor Ioan Olariu and also the results experimentally obtained after applying the similitude theory. Considering the perimeter tube structure our starting point -we will present a comparison between the results obtained using the calculation programmes, the results experimentally obtained and those results obtained by using the finite element method.*

## Rezumat

*In acest articol dorim sa facem un studiu asupra modelelor de calcul a structurilor inalte propuse de cercetatorii clujeni. Ne vom axa in primul rand pe calculul structurilor de tip mixt (cadre-diafragme) si pe calculul structurilor de tip tub perimetral. Vom face o prezentare a rezultatelor obtinute prin metodele de calcul propuse de prof.dr.ing. Ioan Olariu dar si a rezultatelor obtinute experimental in urma aplicarii teoriei similitudinii. Pornind de la structura de tip tub perimetral vom realiza o comparatie intre rezultatele obtinute prin aplicarea programelor de calcul, rezultatele obtinute experimental si rezultatele obtinute aplicand metoda elemetului finit.*

**Keywords:** high-rise structure, tall building, finite element method, shear lag effect

## 1. Nominating the tall structures

Dealing with the nomination of the tall structures, it is rather difficult to distinguish between the characteristic elements of a certain building qualified as a tall building, and this is because the term "tall building" it is quite relative. In an area in which exist only 4 levels-maximum buildings, erecting a 10 levels structure might qualify it as a "high-rise building". A 50 levels building erected in a metropolis could be called a "tall building", whereas the inhabitants of a small city would boast about their 6 levels buildings as being "tall structures".

In big United States cities and lately also in United Arab Emirates cities, where exist a large number of tall structures, it got to the point where only the 100-200 levels buildings are classified as "tall structures". Although you would prefer to know exactly when a certain structure is qualified as a "tall structure", there is no clear explanation regarding the elements which certify a tall structure, there is no minimum height or minimum number of levels above which we can call the structure a "high rise structure".

[*] Corresponding author: Tel./ Fax.: 0040742098768 (Bianca R. Parv)
E-mail address: bianca_parv@yahoo.com

From the design engineer's point of view, a structure it's classified as a "high-rise structure" when the designing as well as the structural analysis it is highly influenced by the action of the lateral loads. For example : the taller the structure is the more the lateral loads -caused mainly by the wind – affect the choice of the structural system.

In the wind codes the calculation processes are divided into two categories:

1) Static analysis - which consists in a quasi-static approach where the building is considered as being a rigid body - method not suited to tall, slender structures, prone to be affected by wind loadings' vibrations.

2) Dynamic analysis - suited to tall, slender, vibrations predisposed buildings. The wind codes specify the necessity of such an analysis for those structures that have the height/width ratio above 5 or for those with the frequency in the first mode of vibration below 1 Hz.

Probably the boundary between tall structures and medium structures should be placedwhere the design engineer considers he should make a structural dynamic reckoning instead of a static one-which is to be done in the case of medium or small height structures.

High-rise structures undertake relatively in a simple manner the vertical loads and transmit them to the foundation site, except sometimes the undertaking and transmission of the horizontal loads (earthquake and especially wind loads) might become complicated, depending on the adopted structural system.

There are 4 essential requirements that need to be obeyed during the designing of a tall building: strength, stiffness, stability and also the presently applicable laws. The structural strength is an essential requirement, only it must be observed that the taller the building is the more important it becomes to obey the requirements concerning the structural stiffness and stability which in most of the cases influence the choice of the proper structural system used for the designing of tall buildings.

There are 2 methods used to gratify the demands of stability and stiffness of the structure. The first one would be increasing the strength elements' sections, maintaining however a certain limit in order for them to stay practicable and profitable.

The second method would be adopting a new structural system, much more stable and rigid; this one being considered the favorable one.

In order to justify the previous statements: if we consider the case of a 10 levels building designed to undertake the vertical loads and capable of undertaking the horizontal ones as well without increasing the structural elements' sizes ; knowing that the wind loads increase non-linearly once the height increases, we get to the point where for 50 levels structures or higher, selecting the optimal structural system becomes essential when designing a building in the most profitable and viable manner.

Naming the optimum structural system means selecting the structural elements and disposing them in the most efficient way for undertaking and transmitting the loads, which appear in a structure, to the foundation site.

First of all we must comply with the condition that the horizontal wind loads need to be smaller than those loadings that appear when loosing stability. Secondly we must confine the lateral displacement on each level, in order to avoid the inception of the structure's collapse.

Thus, a lateral displacement's limit it is imposed for each level from 1/400 up to 1/500 from the level's height. An additional reason to limit the lateral displacement for each level would be regarding the architecture, interior and the building inside comfort. Generally for lateral displacement's limit we start with increasing the girder's stiffness.

Besides the above mentioned motives there are a number of factors which affect the selection of the structural system such as: the building's architecture, functionality ( for instance an office building requires large areas), the geometry of the building ( the shape in plane and elevation, the height/width ratio, the taller the structure and the higher the height/width ratio gets the more consequential the selection of the right structural system solution becomes), the geographical situation of the building, extremely important in choosing the right structural system is the existing execution technology, installation systems and not least we must opt for the most profitable structural system possible.

In his Ph.D. thesis entitled:" Multi-staged structures for industrial and social-administrative buildings" – published at the Technical University of Cluj-Napoca, Professor Dr.-Ing. Ioan Olariu performed both a elastic and inelastic seismic analysis on a mixed high structure (frames with shear walls), an experimental study of the tube high structures and an elastic analysis of the tube structures.

The professor chose to study and compare the way shear wall-frame structures and perimeter tube structures behave under the action of the horizontal loadings generated by the earthquake.

The main advantage of the rigid frames structures is the regular disposal of the structural elements which allows the arrangement of the windows and doors with no difficulty.

This type of structure is more economical only for those buildings with a maximum of 25 floors; for those higher than that we can observe a high lateral displacement and as a result of limiting this distortion we obtain structural elements with major and unprofitable sections. Thus for structures higher than 25 floors we can opt for a shear wall-frame structure or the perimeter tube system.

The shear walls have a higher stiffness than the frames therefore the structural system with shear walls becomes more efficient for buildings with a maximum of 35 floors. Shear walls have both an architectural role for partition and a strength role for undertaking the horizontal and vertical loadings. Considering the high strength and stiffness they are ideal for high structures.

The shear walls must entirely take over the lateral loads actions, behaving as vertical cantilevers and are generally disposed around the lift's tube or staircase area.

Unlike frame structures the shear walls have the disadvantage of restraining the interior division which affects firstly the open areas as in the case of the office buildings. They are suited mainly for hotels or residential buildings, in which case the interior division is the same for each floor. The frames and the shear walls interact with each other in order to create a more stiffness structure therefore the shear wall-frame structure is suited for 40-60 levels buildings and even- judging from some authors - for those of 80 levels. Although the shear wall-frame structures are made out of concrete, metal can also be used for braced frames or for rigid metallic ones with a similar behavior.

A major role in designing a high, reinforced concrete structure is played by the use of the tube structures. They came into being as a result of the necessity of guaranteeing under optimal economical conditions, the strength and stability of the minimum 40 levels, reinforced concrete buildings and represent an important step towards the construction of high structures.

Since the goal was to obtain a general rigidity higher than the one present in cases of core structures, a tube structural system was developed.

The perimeter tube undertakes the lateral loads, while the gravitational loads are being undertaken not only by the perimeter tube but also by the interior columns. The perimeter tube system, can be defined as a structure of spatial frames disposed on the facades, with a distance of 1.2-3.0m between the columns, spandrels with a height of 0.6-1.2m and width from 0.25m up to 0.9m.

The way tube structures behave under the action of the horizontal loads, situates/locates somewhere between the behavior, under the same action, of the frame structures – where the stress and deflection produced by the shear force prevail- and the shear walls structures-where the stress and deflection caused by bending prevail.

The proximity to one of the two schemes is given by the tube's components' sizes and the distance between them.

## 2. Researchers from Cluj Napoca

A significant figure among the researchers from Cluj Napoca is Professor Dr.-Ing.Olariu Ioan, the author of 7 scientific treatises, coordinator of 25 research-contracts and numerous scientific works published not only in Romania but also abroad. His Ph.D. thesis [1] presents the results of the researches on high-rise structures. Professor I. Olariu includes in his work the analysis of 2 types of structures: shear wall - frame structure and the perimeter tube structure - affected by the seismic loads.

In the case of shear wall - frame structures, Professor I.Olariu describes the method used to establish the elastic seismic response using the matrixes of flexibility at lateral displacements, assuming the structure is formed out of many vertical substructures. Likewise the author has elaborated, utilizing Fortran programming, a calculation software called SEISCLAD, tested through approximate methods and by comparing theoretical results with those obtained from experimental attempts on a shear wall-frame structure model. The programme's practicability is strengthen by the numerous projects with a shear wall-frame structure, calculated using this method. The author of this thesis makes also an analysis on an existing structure, relying on the acceleration's spectrum of the earthquake that took place in March 1977. Following the elastic analysis we can be observe the appearance of the plastic hinges in the central core's shear wall's lintel. Thus the author notices an adequate behavior of the structure, calculated using the codes of those days.

Considering the inelastic response to the seismic application, the author developed a calculation method based on the matrixes of inflexibility, which allows us to follow the stress-deflection state through elements during the earthquake. Any of the histeretic model of calculation can be adopted. Solving the differential equations of motion is done using the Newmark method (step by step). The laborious calculation method, developed by professor I. Olaru, completes with a calculation programme called SEIS, recommended for standard projects.

The Ph.D. thesis also displays a spatial elastic analysis of the perimeter tube structures, formulated using the matrixes of inflexibility's methods, where we take into consideration the cooperation effects, at intersections, between the vertical elements of resistance. The system of dynamic equilibrium equations may be solved either through modal analysis or through numerical integration methods. The lateral rigidity matrix of the structure can be obtained by summing up the lateral matrixes of rigidity of all the frames, which are obtained but reducing the matrixes of total rigidity of the frames. The author adapted a programme for the spatial dynamic and static analysis of the shear walls - frames structures–ETABS for the FELIX C-256 computer in order to apply the developed method. The results of the experimental attempts on a 20 levels prototype structure, reduced to a 1:30 scale, got compared to those obtained after using the calculation programme. The experimental model's stress and deflection are in good agreement with those calculated using the ETABS calculation programme.

Another semnificant work, when it comes to high-rise strucures field, drawn up at the Technical University of Cluj-Napoca is the Ph.D. thesis of Professor Delia Dragan: " The structure and plasticity of high-rise buildings". Here we find a description of the main architectural styles of high-rise structures from USA, starting with XIX century till the end of the '90s: the early modern style(Home Insurance Building-Chicago, 55m), the modern American style (Second Leiter Building-Chicago; Reliance Building-Chicago, 61m), followed in the second quarter of the XX century by the International Style ( the Rockfeller Center-New York ensemble/pile, 14 skyscrapers with steel resistance structure; Empire State Building-New York, 381m, World Trade Center-New York, 417m(415m)). During the same period the perimeter tube structures appear (concept developed by the ing. Fazlur Khan), exterior diagonal tube, perimeter tube and interior core walls, multiple tube. Starting with the mid '60s we encounter 2 new orientations : neomodernism (John Hancock Tower-Boston, 240m) and postmodernism which includes the high-tech style (Banca Hong Kong & Shanghai Corpotation-Hong Kong, 179m). The '90s architecture in USA is dominated /subordinated by 2 tendencies : modernism and deconstructivism. The latest structural shapes, for high-rise buildings are megastructures(Umeda Sky-Osaka,173m) ; the ecological concerns well known presently determined nature's integration into built up areas [2].

Professor Delia Dragan deals with the problem of correlating the 3 factors that affect the concept of the high-rise structures: architectural functionality-structure-plasticity, presenting numerous examples where the 3 factors are joined together in an harmonious way.

In chapter 4 of her Ph.D, thesis prof.dr.ing. Delia Dragan presents a classification of the tall buildings' structural systems, their behaviour under the actions of the horizontal forces, the height limits up to which they are still efficient.

The tallest building in Romania -Tower Center International, Bucuresti- measures 106.3m, with a dual metallic resistance structure with braced frames and a height condition 3S+P+22E+3Etehn. Both seismic loads and wind loads were determined in sizing the elements, aiming for the erection of a building with a fundamental period large enough to reduce the basis shear force, while maintaining the lateral displacements- caused by the wind's action- within the accepted limits [6]. The static and dynamic calculation was made using the calculation programme ETABS; the elements were sized taking into consideration both the romanian codes P100/92 and P100/2006 and the european codes (EN 1993-1.8, EN 1994-1).

The building's structural performances has been studied using certain procedures based not only on the seismic performance but also on the testing done in the wind tunnel. Three levels of performance have been considered (SLS, ULS, CPLS) which correspond to 3 levels of the seismic imminence, considering the reccurence periods of the earthquakes (frequent, rare, extremely rare). Following an incrementale non-linear dynamic analysis, it can easily be observed inside the structure the shaping of a global plastic mechanism. The structure's behaviour under extreme loads (explosion, impact) it is also observed and its capacity to redistribute the loads following the collapse of some important capacity carrying elements. The procedure consists in removing some of the structural elements, for example the interior columns from one level, followed by the non-linear analysis of the structure determining in this way the number of columns the structure is able to loose until the collapse starts. Those buildings that have a sesmic conformation show abilities like ductility and force to absorb the energy induced by the earthquake, which diminishes the risk of progressive collapse.

Recent studies of the high rise structures draw attention to the inadequate design as a result of using the standard method with seismic forces of equivalent static levels. Mihail Iancovici in his work [5] underlines the importance of using the concept of seismic structural performance (SEAOC, 1995) which involves various structural requests depending on the earthquake's intensity(minor, medium, major).

## 3. Comparing the results experimentally obtained with those obtained using the finite element's method

In his Ph.D. thesis 'Multi-staged structures for industrial and social-administrative buildings", chapter 4, Prof. Dr. Ing. Ioan Olariu presents the experimental testing of a reinforced concrete perimeter tube structure.

The 20 levels structure has a 56m height and dimension in plane of 14,8x20m. The longitudinal elastic modulus En = 265000 daN/cm² and the equivalent shear modulus Gn = 0,4 En. All these values belong to the prototype, the model being sized afterwards using the below values.

For this experiment the author applied the similitude theory using the II theorem(Buckingham).

Not only for the static actions but also for the dynamic actions.

Assigning the dimensions of the model and analyzing the results was done on the basis of the dimensional analysis and on the basis of the similitude theory considering the following hypotheses:

- the material is homogenous, isotropic, elastic linear;
- the deflections are situated inside the area of the minor displacements[Al. Vasilescu]

The similitude theory applied in the case of static stress starts with the physical dimensions function which defines the elastic system:

$$f = (\sigma, l, F, E, \gamma, \delta, \tau, G, (r\varphi), \mu) = 0 \tag{3.1}$$

Using the II theorem , in the case of the minor deflections, the equation becomes :

$$\varphi(\frac{F}{\sigma l^2}, \frac{E\delta}{\sigma l}, \frac{\gamma l}{\sigma}, \frac{G(r\varphi)}{\tau l}, \mu) = 0 \tag{3.2}$$

The scales chosen to obtain the experimental model :

$$S_l = 30;\ \ S_E = 1{,}2;\ \ S_G = 1{,}2;\ \ S_F = 1200;\ \ S_\varphi = 1; \tag{3.3}$$

Thus result the scales below :

$$S_\sigma = 1{,}33;\ \ S_\delta = 1{,}2;\ \ S_\gamma = 1{,}2;\ \ S_\tau = 1{,}2;\ \ S_\mu = 1; \tag{3.4}$$

The experimental model will have a 200cm height with the outlined dimensions of 49.35x66.8 cm, walls thickness 15mm and flat slab thickness 12mm [1].

Details about the model's reinforcement are presented, the material used for the model is micro concrete with a medium cubic strength for 28 days of 325 daN/cm². In order to accomplish this experimental study a testing stand was used composed of a supporting framework for the mechanical comparators which register the horizontal displacements and an ensemble by which use the horizontal forces are applied on the model's height. The structure is activated in the short side direction by the horizontal forces and torsion moments following the seismic loads, applied in 3 successive steps. The experimental model was subject to dynamic loads given by some horizontal displacements and initial bending followed by a sudden relief and a free oscillation.

Solving the structural model using the finite element method would have given the most accurate results, but at the time this Ph.D. thesis was published, this calculation method could not have been realized because that would have required some major capacity computers, unavailable at that time.

Therefore we would like to compare the results obtained using the experimental model from the above mentioned Ph.D. thesis with the results obtained on the same model, this time using the finite element method. The calculation method based on the finite element method is called 'Robot Structural Analysis Professional'.

The dimensions of the model scaled 1:30 remain the same during the structural analysis using the finite element method.

Considering the instructions from P100-2006, the element's stiffness differs. In the case of columns, girders, and shear walls EI=0.5EcIc . Diminishing the element's stiffness, the structure is considered in the plastic stage thus the structure's behavior during the seism is approximated with the requirements from P100-2006.

In order to define the material it has been created in the calculation programme a concrete category having a medium cubic strength for 28 days of 325daN/cm².

In the Ph.D. thesis" Multi-staged structures for industrial and social-administrative buildings", the experimental model is being loaded with horizontal forces brought up by the seism in the short side direction, which is maintained when calculating using the finite element method. The perimeter tube structure under the pressure of the horizontal loads behaves like a vertical cantilever. The perimeter frames, parallel with the wind direction, behave like the 'web' of a cantilever truss while the perpendicular frames in the wind's direction behave like the 'flanges' of a truss.

One disadvantage would be the frame's behavior in the perpendicular direction of wind's loads which is affected by the shear lag effect, caused by the central columns from the 'flange' which are affected in a lesser degree than those from the corner, thus not undertaking the loads that would be distributed to them. The shear lag effect can be reduced by the inclusion of additional columns which would increase the bending efficiency.

In the Ph.D. thesis results experimentally obtained are presented only for level 1 and 4 of the structure.

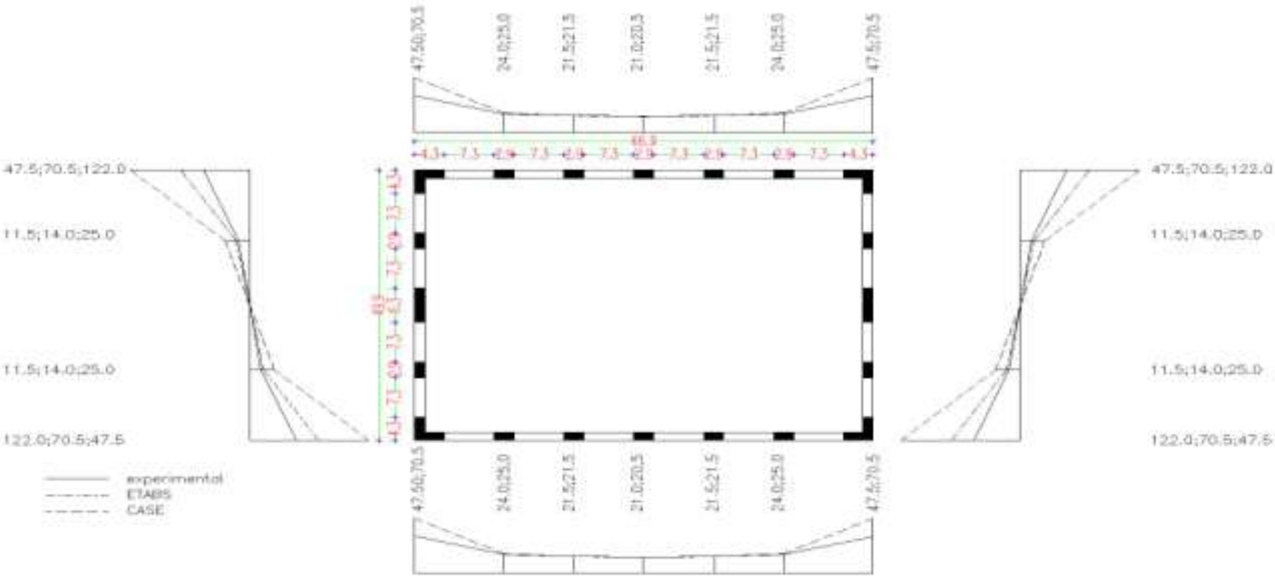



Figure 1. Columns' axial stress determined at level 4

Using the finite element method we obtain axial stress in columns at level 4

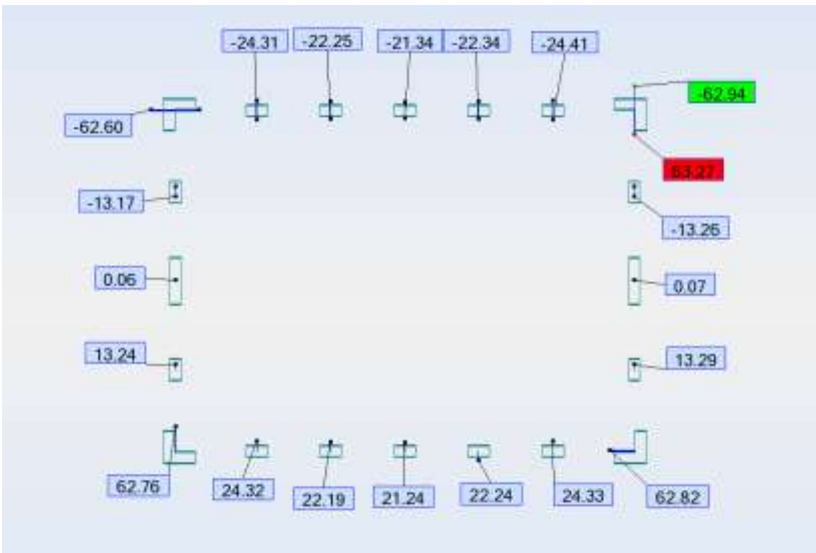



Figure 2. Columns' axial stress (Robot Structural Analysis results)

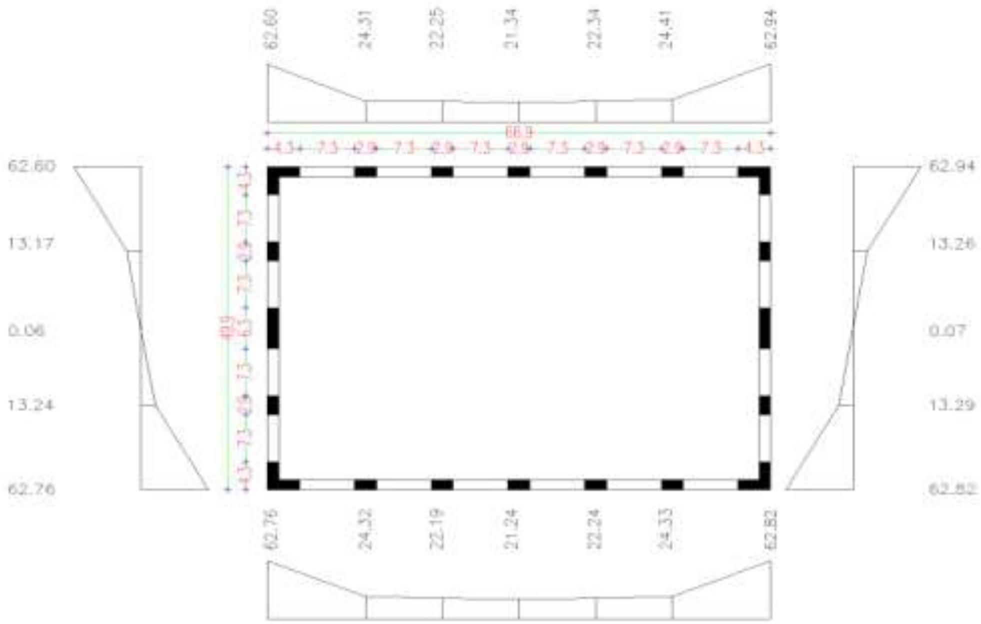



Figure 3. Columns' axial stress determined at level 4

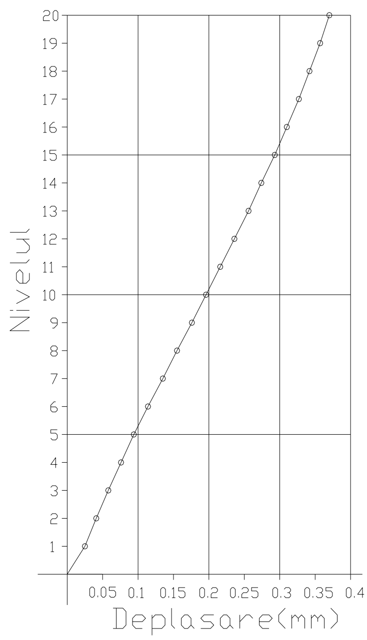



Figure 4. Lateral displacements determined by horizontal loads using the finite element method

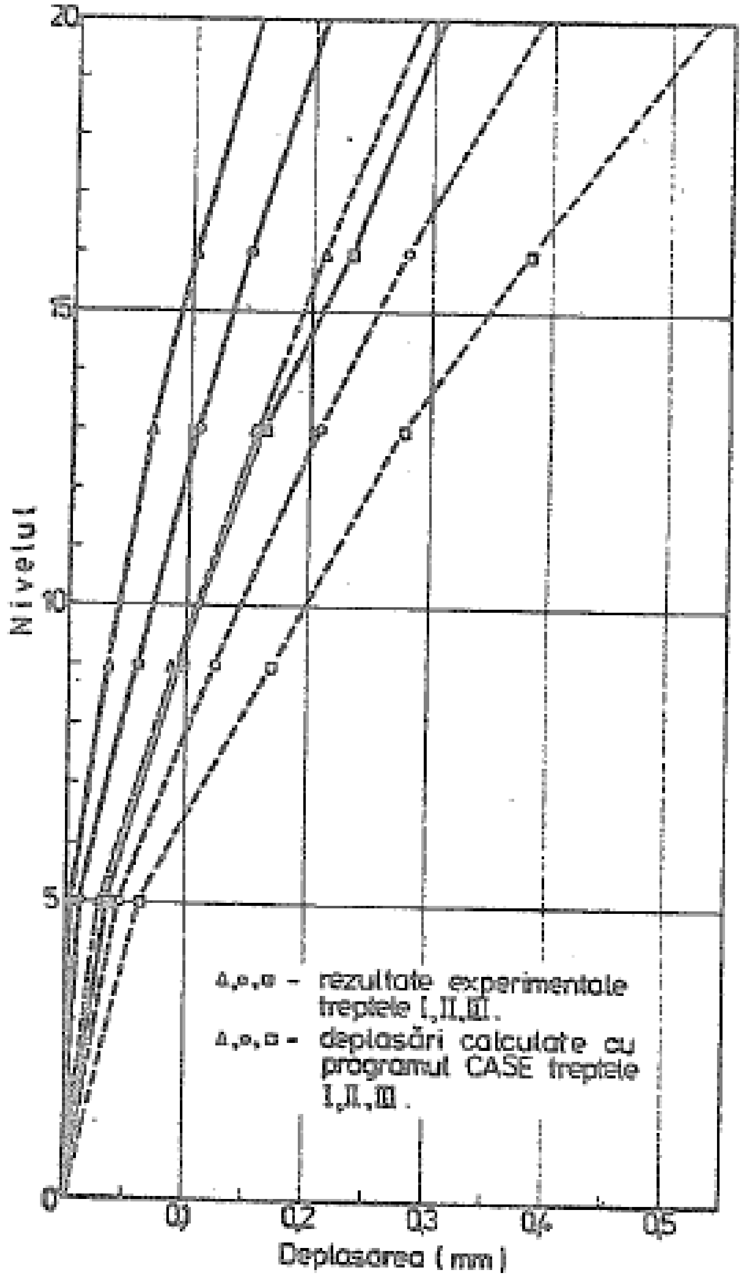



Figure 5. Lateral displacements determined by horizontal loads (results obtained and presented in Ioan Olariu's professor Ph.D. thesis, chapter 4)

## 4. Conclusions

The goal of this work is to accomplish about a study of all the results obtained following the researchers carried out in Cluj-Napoca concerning high-rise structures and particularly reinforced concrete perimeter tube structures.

The Ph.D. theses on which this study is based are :" Multi-staged structures for industrial and social-administrative buildings" of Prof.Dr.-Ing. Ioan Olariu and "The structure and plasticity of high-rise buildings" of Prof.Dr.Ing. Delia Dragan, both under the direction of Prof.Dr.Ing. Mircea Mihailescu.

The first Ph.D. thesis presents a spatial elastic seismic analysis of tubular structures, formulated using the matrixes of stiffness method. The calculation method turned out to be an efficient one, appropriate in designing tall buildings. At the same time, the author of the Ph.D. thesis completed an experimental test on static and dynamic actions, using a model of reinforced micro-concrete tubular structure. Prototype presented on a 1 :30 scale.

Considering that the seismic analysis of the perimeter tube type structure was brought into in a time when the finite element method was not at all accessible for the calculation of structures, "the finite element method would give the most accurate results, but the necessity of utilizing the computers of big capacity makes it prohibitive" Ioan Olariu., the goal now is to accomplish in parallel a certain calculation based on the finite element method, on the same perimeter tube structure, calculated by professor I. Olariu.

Utilizing the same entrance information ( we maintain the tube structure, the geometry , the elements' dimensions, the material and seismic loads) we calculate the structure based on the finite element method, using the "Robot Structural Analysis" programme.

We intend to compare the experimentally obtained results by professor I.Olariu with the results obtained using the calculation based on the finite element method. In figure 3 the axial stress in columns at level 4, is obtained using the finite element method.

Thus we can observe that these results place themselves inside the interval of experimentally obtained results or of those results obtained by the use of ETABS and CASE calculation programmes - figure 1.

Similarly we will compare, at each level, the displacements due to horizontal actions following a seism using the calculation based on the finite element method (figure 4) with the displacements experimentally obtained or obtained utilizing the ETABS and CASE calculation programmes on different levels of loading(figure 5).

What we can notice is that the horizontal displacements values thus obtained are close to those values obtained in the Ph.D. thesis. Therefore we demonstrate the accuracy of applying the finite element method, the preciseness of the results thus obtained and the improvement of the calculation method throughout the years.

**AKNOWLEDGEMENT**

This paper was supported by the project "Doctoral studies in engineering sciences for developing the knowledge based society-SIDOC**"**contract no. POSDRU/88/1.5/S/60078, project co-funded from European Social Fund through Sectorial Operational Program Human Resources 2007-2013.

## 5. References

[1] Ioan Olariu, "*Structuri etajate pentru constructii industriale si social-administrative*", teza de doctorat, Cluj-Napoca, 1980.

[2] Delia Dragan, "*Structura si plastica cladirilor inalte*", teza de doctorat, Cluj-Napoca, 2000.

[3] Bungale S. Taranath, *''Reinforced Concrete Design of Tall Buildings''*, CRC Press, 2010.

[4] Bryan Stafford Smith, Alexx Coull, ''T*all Building Structures''*, Wiley-Interscience Publication.

[5] Mihail Iancovici, Cristian Gavrilescu, '' *Database-assisted Seismic Analysis of Tall Buildings Subjected to Long Predominant Period Vrancea Earthquakes''*, Bucuresti, 2008.

[6] Dan Dubina, Florea Dinu, Aurel Stratan, "*Tower Centre International building in Bucharest- Structural Design*"

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Using the Satellite Positioning System within the Public Transportation Management

Voichiţa A. Roib[*1]
[1] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*The Global Navigation Satellite Systems are systems which make possible the determination with high precision of the position within a geocentric reference system. One of the applications of the satellite positioning technology is that provided for the management of transports, regarding vehicle navigation and fleet monitoring. The technical performance of telecommunication and informatics allow us to address traffic problems arising due to the constant development of mobility. The paper presents the satellite positioning system for determining and monitoring positions in real time, applied within the management of public transportation of Cluj-Napoca city, its implementation, advantages and an informatic program designed for informing the user about the urban transport conditions.*

## Rezumat

*Sistemele Satelitare de Navigaţie Globală sunt sisteme care fac posibilă determinarea cu precizie ridicată a poziţiei într-un sistem de referinţă geocentric. Una dintre aplicaţiile sistemului de poziţionare satelitară este cea oferită în managementul transportului, pentru navigaţia vehiculelor şi monitorizarea flotelor. Performanţele tehnice ale telecomunicaţiilor şi informaticii, permit rezolvarea problemelor apărute în trafic datorită creşterii constante a mobilităţii. În lucrare se prezintă sistemul de poziţionare satelitară utilizat pentru determinarea şi monitorizarea poziţiilor în timp real, a cărui utilizare devine tot mai frecventă, aplicat în managementul transportului public din municipiul Cluj-Napoca, implementarea, avantajele acestuia şi un program informatic conceput pentru informarea în timp real a utilizatorilor asupra condiţiilor de transport urban.*

**Keywords:** Global Navigation Satellite Systems, traffic management, information, urban transport

## 1. Introduction

Location is determined on the basis of satellite observations made in several regions of interest and by acquiring their coordinates within a well-established reference system. Satellite observations are achieved on the basis of measurements performed between the satellite receiver, located on the ground or in its vicinity and one or more satellites which evolve on circumterrestrial orbits.
Satellites send signals which are then received by specialized receivers, which decode them; the information necessary for determining the receptor's position is extracted afterwards.
The Global Navigation Satellite Systems (GNSS) make possible the determination with high precision of the position within a geocentric reference system, with the help of Earth's artificial

[1*] Assistant Eng.Voichiţa A. Roib, Ph.D: Tel./ Fax.:0040264-430428
E-mail address: Voichita.ROIB@ggi.utcluj.ro

satellites, in any point located on the terrestrial surface, in its vicinity or external part. The most known GNSS systems are the NAVSTARP-GPS (U.S.A.) and the GLONASS (Russia) systems. From the point of view of their functioning principles and the used technology, these two GNSS systems are similar. They have 3 segments:
-a space segment (satellites which send users time and navigation information and also messages about the system's state);
-a control segment (monitoring and control stations), which carry out the functioning of the satellite constellation, of the time system attached and the determination of the satellites' orbits;
-a users segment [1].
In order to determine a position by using the GNSS satellite positioning technologies (Global Navigation Satellite Systems), passing from the post processing mode to the determination of position in real time, it's necessary to create some complementary positioning systems on a regional, national or local level. Complementary positioning systems offer additional information ("differential adjustments") besides the data received directly from the GNSS satellites, so that high-accuracy and real-time positioning can be achieved (absolute positions with decimetric or centimetric precision).
Depending on the precision level, positioning systems can be: D-GNSS (Differential GNSS) – decimetric and RTK (Real Time Kinematic) – centimetric. The National Agency for Cadastre and Land Registration in Romania created such a system called ROMPOS (Romanian Position Determination System).
Through this system, precise positioning within the ETRS89 European reference and coordinates system can be achieved, on the basis of the Permanent GNSS Stations National Network. ROMPOS is based on Global Navigation Satellite Systems (GNSS), including GPS, GLONASS and GALILEO and providing complementary data, necessary for the improvement of the positioning accuracy up to the order of several millimeters. The system is available in every moment and for every location in Romania. This system provides modern services, having the following applications [1]:
-ROMPOS DGNSS – Geographical Information Systems (SIG), vehicle navigation, fleet monitoring, maritime and air navigation, supporting public authorities (police, firefighters, ambulance), tourism, etc;
-ROMPOS RTK – Cadastre, Information Systems which are specific to different fields of activity (local government, real estate – urban, public utilities – water, gas, sewers), disaster management, construction and engineering measurements, scientific research, meteorology, bathymetric measurements, etc;
-ROMPOS GEO – Geodetic networks of support and thickening, support networks for tracking and tracing constructions over time, Geographical Information Systems (SIG), geodynamics, air photogrammetry, laser scanning, scientific research, etc.
Principles of the DGNSS (Differential GNSS) and RTK (Real Time Kinematic) are:
-absolute differential positioning – a technique used to determine the position of a receiver, usually mobile, on the basis of direct observations towards satellites and some adjustments (differential) sent (in real time) from another fixed receiver, which is also called reference or base receiver;
-the distances between satellites and receivers, measured by the mobile receiver are adjusted on the basis of the differential adjustments obtained from the base receiver and then there is an absolute positioning (punctual);
-the transfer of the DGNSS/RTK differential adjustments from the reference stations (stations' network) to the user is carried out by various means, the most common being: the transfer by radio waves, through GSM/GPRS mobile communication systems or via Internet.
The DGNSS/RTK services of the ROMPOS are based on the data transfer via Internet. These data are transmitted in a standardized format (RTCM – Radio Technical Commission for Maritime Services) with the help of the NTRIP technology (Networked Transport of RTCM via Internet Protocol). The dissemination of differential adjustments or other types of GNSS data to stationary or moving users is done via Internet [1].

The benefits of using the Romanian System of Position Determination are: enhancing the effectiveness of modern GNSS receivers, increasing labour productivity, reducing costs.
One of the applications of the satellite positioning technology is that provided for the management of transports, regarding vehicle navigation and fleet monitoring.
The management of public transport contains advanced technologies based on GNSS, for public transportation, located inside the vehicle and modern solutions for the planning, organization, efficient operation of vehicles and parks of vehicles and for informing the user about the urban transport conditions.
The conducted studies analyzed the movement of public transportation in Cluj-Napoca city and identified the traffic difficulties related to the geometric elements of streets, the route, the traffic conditions (congestions, illegalities, vehicles parked on the road), the drivers' and pedestrians' behaviour, etc.[2]. The analysis shows that in almost all major intersections there are high levels of public transport vehicle flows.
By examining the configuration of the public transport network, we have encountered many parallelisms and overlaps between the lines, even when there is a big transportation capacity (tram line) or a fixed railway infrastructure (tram, trolleybus) which have to be efficiently exploited, due to the configuration of lines in the intersections on the east-west axle there are turning maneuvres which require special phases within the traffic light cycle.
The difficulties which were encountered lead to the proposal of implementing some advanced technologies for the management of the public transport fleet.

## 2. Implementation of the Vehicle Tracking and Location System and the Informing Program for the Urban Transport in Cluj-Napoca City

One of the advanced technologies for managing the park of vehicles is the automatic vehicle tracking and location system, which leads to the efficiency of services and passengers' safety. The vehicle automatic location technology is based on the real – time determination of the vehicle's geographical position and the data transmission to a central station [3].
Taking into account the actual tendencies of transport development, the Local Independent Urban Passenger Transport Operator of Cluj-Napoca (RATUC) wants to improve the public transport network in Cluj-Napoca. Thus, RATUC Cluj implemented a vehicle location, tracking and management system, by using GPS-GSM technologies [4]. The Global Positioning System (GPS) technology uses signals sent by satellites launched in order to determine position.
The GPS is a modern technology for determining the geographical position in any point on the globe, with the help of a GPS receiver. The tracking and location system consists in the AVL mobile units (Auto Vehicle Location), which are attached to the monitored vehicles, the GSM communication network and the monitoring equipment.

### *2.1 Auto Vehicle Location Module*

RATUC Cluj-Napoca began to exploit an AVL Module (vehicle tracking and location), manufactured by RADCOM Bucharest, which offers real-time information about the position of vehicles. This module is a subsystem of a modular system, which offers to its user a complete instrument for monitoring, dispatching and controlling traffic, which leads to the achievement of a superior management of the urban transport vehicles' fleet, from the point of view of operational costs and of the relationship with passengers [5].
The system's architecture consists of more interconnected subsystems:
-a subsystem for dispatching and coordinating public traffic vehicles;
-a subsystem for acquiring, processing and displaying the vehicle's state parameters (embarked equipment);
-a passenger information display subsystem (inside stations);

-a communication subsystem.
The passenger information subsystem consists of LED displays and screens placed inside stations. It is connected to the subsystem for dispatching and coordinating public traffic vehicles, from which it receives information on the estimated time of arrival of the next vehicle, advertising or general information.
The collection of data regarding the vehicle's position is done by a device, a precise navigation system which uses GPS satellites. The GPS module prepares the information package (point coordination, module identifier) which is transmitted every 30 seconds through the GPRS (mobile phone operator - Orange), through the control server we arrive to a web page which is updated. The soft installed on the server processes the information which is stored at the internet address and shows the position of buses on a linear path, calculates the time which is necessary for arriving to the panels installed inside stations and stores all the information of the day [4].
After the implementation of the monitoring pilot programme in 2005, for bus line 30, by which 28 buses received a GPS device as shown in Fig.1 and 6 panels were installed inside the following bus stations: Memorandumului (roundtrip) as shown in fig.1, Regionala C.F.R. (roundtrip), Petuniei, P-ta Marasti, at present the programme has been extended to all the lines which are adjacent to the Grigorescu node (26, 27, 28, 30, 41, 43), all vehicles which pass through the Grigorescu node (60 vehicles) received a GPS equipment and 5 information panels have been installed, informing passengers about the arrival time of the next vehicle [6].


Figure 1. Bus with GPS device. Informing panel.

***2.2 Informing Program for Urban Transport***

As a consequence of the increase of the number of inhabitants in cities, development of motor vehicle industry, the street networks of major cities are incapable to fulfill the requirements of users, causing a large number of traffic difficulties. Most of the displacements are done on the road network which offers greater freedom for organizing the trip or the transport but with negative consequences in what concerns traffic safety and comfort. To solve such problems traffic surveys are necessary and an adequate management based on new technologies for the collection and transmission of information related to the infrastructure and traffic status.
As a research result, a program that informs about traffic conditions was conceived [2]; traffic data is presented on a site which provides real-time information, as shown in Fig.2.

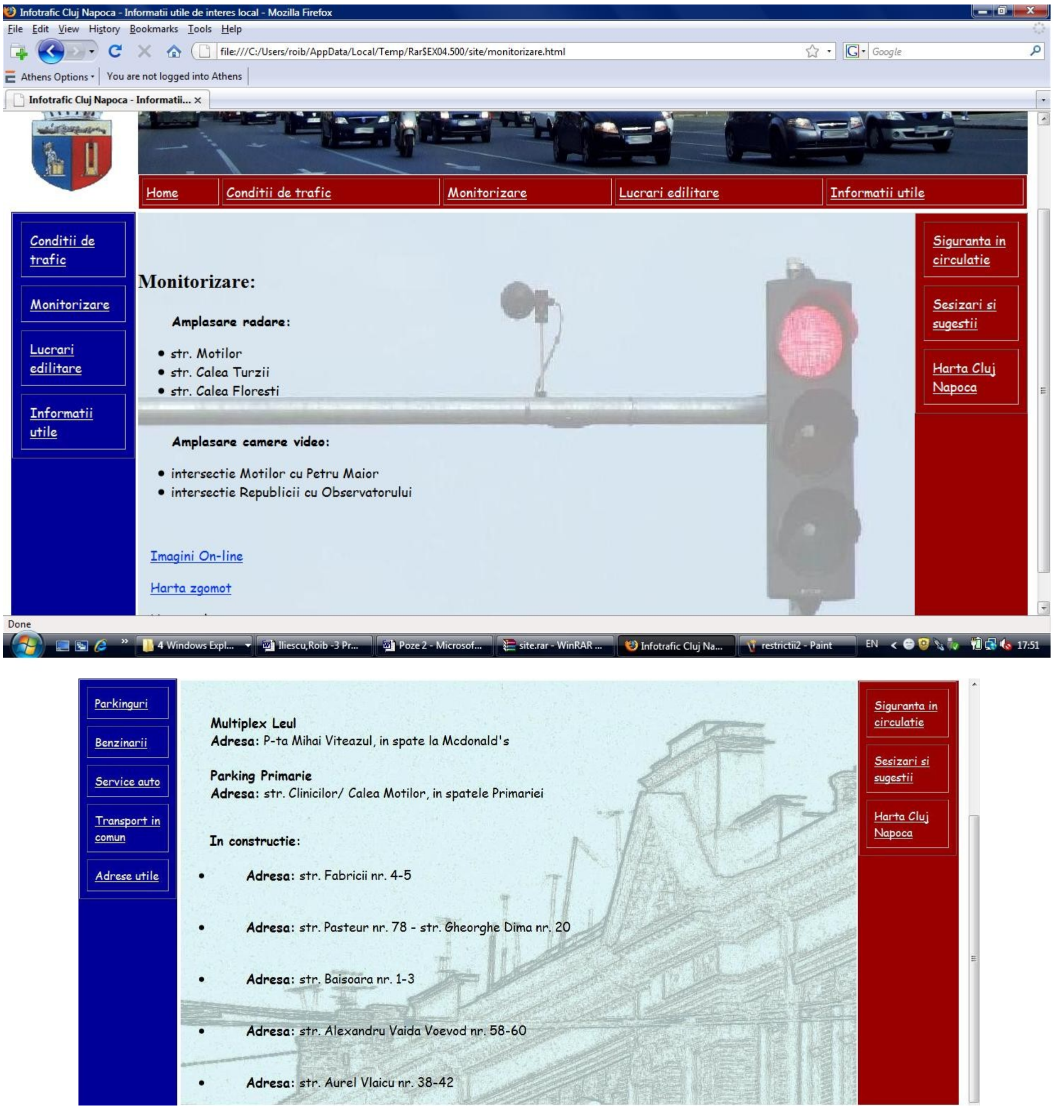



Figure 1. Informing  Program. Useful information page.

## 3. Conclusions

The technical performance of telecommunication and informatics allow us  to address problems arising due to the constant development of mobility. To solve such problems traffic survey are necessary and adequate management based on new technologies for collection, transmission of information related to traffic status. The public transport, through its technologies, has the purpose of leading to the increase of public transportation services efficiency and of meeting the needs of its clients in terms of safety, comfort, economy and environmental protection. The paper presents one of the applications of the satellite positioning system offered for the management of public transport, regarding vehicle navigation and fleet monitoring.

With the help of satellite positioning system and an real time informing program, the following tasks are achieved:

-vehicle positioning through the GPS modules installed on vehicles and data transmission through the GPRS;

-providing citizens with information panels located in key points of traffic and website;

-informing the bus driver about the schedule being respected or not (advance, delay, normal);

-monitoring the evolution of circulating vehicles by the central dispatch and transmitting operative commands if necessary;

-permanent communication between bus drivers and central dispatch;

-default messages sent by the bus driver to the operative command centre;
-operative intervention in case of need (blocks, accidents, etc.);
-storing the data of every day within a database and its ulterior processing for accounting purposes;
-received data allow the analysis and optimization of resource allocation in order to meet the transportation needs.

The use of satellite positioning system for determining and monitoring positions, leads to the improvement of public transport management, to the increase of public transport services, meeting the users demand, increasing profit and optimizing productivity.

## 4. References

[1] ROMPOS- http:// www.rompos.ro

[2] Roib VA. *Contribuţii la fluidizarea circulaţiei rutiere din municipiul Cluj-Napoca, prin utilizarea unui sistem de gestiune modern de trafic*. Teza de doctorat, Universitatea Tehnică din Cluj-Napoca, 2009.

[3] Banciu D, Hrin R, Mihai G, Eşanu A, Alexandrescu M, Anghel L. *Sisteme Inteligente de Transport - ghid pentru utilizatori şi dezvoltatori.* Bucureşti: Editura Tehnică, 2003.

[4] RATUC Cluj-Napoca – http:// www.ratuc.ro

[5] RADCOM Bucureşti – http:// www.radcom.ro

[6] PRIMARIA CLUJ-NAPOCA- http://www.primariaclujnapoca.ro

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage: http://constructii.utcluj.ro/ActaCivilEng*

# The Choice of Boilers Using Global Evaluation Method of Performances

I. Giurca[*1]

[1]*Technical University of Cluj-Napoca, Department for Installations and Constructions. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*In this paper, there is presented the choice for optimum method as concerning boilers, by using global evaluation of performances. In the end there is made a comparison of the results obtained by the author in contrast to those obtained by other authors as a conclusion under the supervision of Electre method. On the base of this study it is clearly presented the choice of technical solution that can be influenced by: chosen Mathematical pattern, type of chosen scale for quality features, importance criteria given to the decisional criteria.*

## Rezumat

*În lucrarea de faţă, este prezentată alegerea variantei optime pentru cazane, utilizând metoda de evaluare globală a performanţelor. În final se face o comparaţie a rezultatelor obţinute de autor, cu rezultatele obţinute de către alţi autori în urma analizei făcute de către aceştia cu ajutorul metodei Electre. Din studiul efectuat rezultă faptul că alegerea soluţiei tehnice poate fi influenţată de: modelul matematic ales, tipul de scală aleasă pentru caracteristicile de calitate, precum şi de coeficienţii de importanţă acordaţi criteriilor de decizie.*

**Keywords:** Global evaluation method; Boilers; Mathematical pattern; Electre method.

## 1 Introduction

### 1.1 General notes

The choice of materials and design devices within installations for constructions is one of the matters less studied among other matters in the field both in our country and abroad. In accordance to these circumstances, it represents a future study direction for the specialists in this field.

In most cases, the person who takes decisions, namely the manager of the company or the chef of supplies department, having not enough time or complete and coherent information, chooses materials and necessary equipments based on a futile analysis, and thus the final results are not the best.

[*] Corresponding author: Tel./ Fax.: 0264 594967

### 1.2. Present phase of knowledge

Nowadays in Romania there are few papers about the choice of boilers by using the global evaluation method of performances.

### 1.3. Recent research in the field

The most important papers in this field are presented in worksheets [1, 2, 4].

### 1.4. The purpose presented in the article

The presented article tries to solve scientifically a classical matter that appears during the process of design made by an engineer specialized in installations for constructions, namely the choice of a boiler. And there is used the global evaluation methods of performances.

## 2. STAGES IN CHOOSING BOILERS

In the case of global evaluation method of the boiler performances, there come up the following stages:

*a) Establishment of choices*

Let's assume, for example, that we have detailed information on four types of boilers and their function is done by using gas supply, all boilers being marketed on installation market in our country. I name these boilers: Cz1, Cz2, Cz3, Cz4 [1].

Table 1 Multitude of boiler types

| $V_i$ | Name of types |
|---|---|
| $V_1$ | Cz1 |
| $V_2$ | Cz2 |
| $V_3$ | Cz3 |
| $V_4$ | Cz4 |

*b) Establishment of evaluation criteria*

They have to choose the criteria that will determine the decisional act. It is recommended that these criteria to be coherent, exhaustive and non-redundant, and their number to be less than ten. In our case the following criteria will be considered (see table 2) when choosing the boiler, all criteria being imposed by the preferences of multiple beneficiaries , but not always the same with the rules made Law no. 10/1995 [1].

Table 2 Multitude of used criteria in selection of boilers

| $C_j$ | Name of criterium | M.U. | Scale |
|---|---|---|---|
| $C_1$ | Life duration | years | maximized |
| $C_2$ | Boiler efficiency | % | maximized |
| $C_3$ | Minimum gas pressure | mbar | minimized |
| $C_4$ | Type of ignition | | maximized |
| $C_5$ | Price | lei; euro | minimized |
| $C_6$ | Emission of noxious | $CO_2$ | minimized |
| $C_7$ | Electricity consumption | kWh | minimized |
| $C_8$ | Warranty | years | maximized |
| $C_9$ | Noise level | dBA | minimized |

*c) Establishment of importance coefficients connected to evaluation criteria.*

Chosen importance coefficients are presented in table 3.

Table 3 Used criteria for importance in the boiler selection

| $C_j$ | Name of criterium | Kj |
|---|---|---|
| $C_1$ | Life duration | 2 |
| $C_2$ | Boiler efficiency | 1 |
| $C_3$ | Minimum gas pressure | 2 |
| $C_4$ | Type of ignition | 1 |
| $C_5$ | Price | 3 |
| $C_6$ | Emission of noxious | 1 |
| $C_7$ | Electricity consumption | 1 |
| $C_8$ | Warranty | 3 |
| $C_9$ | Noise level | 1 |

*d) Choice of measurement*

To express the choice evaluation, considering each criterium, it is usually used a range of ratings instead of one using numbers: very well (VW), well (W), medium (M), satisfactory (S), unsatisfactory (US). Due to the fact that the calculus is done using numbers, there comes the need to change ratings into grades. [1]. The calculus will be done by two methods, namely that first we choose the measurement in ten levels and then the measurement in five levels.

*d.1) Measurement in ten levels, using the replacement of ratings with grades*

Conversion of ratings into grades can be done in respect for the field literature [Maystre], in accordance to table 4 [1, 3].

Table 4 Change of ratings into grades taking into account importance criteria

| | **Very well** | **Well** | **Medium** | **Satisfactory** | **Unsatisfactory** |
|---|---|---|---|---|---|
| Criteria with the average ≥ 3 | 10 | 7,5 | 5 | 2,5 | 0 |
| Criteria with the average 2 | 8 | 6,5 | 5 | 3,5 | 2 |
| Criteria with the average 1 | 7 | 6 | 5 | 4 | 3 |

d.2) Measurement in five levels, by converting ratings into grades

Transformation of ratings into grades can be done in accordance to table 5 [3].

Table 5

Change of ratings into grades

| **Crt. No.** | **Qualification** | **Code** | **Grade** |
|---|---|---|---|
| 1 | Very well | VW | 5 |
| 2 | Well | W | 4 |
| 3 | Medium | M | 3 |
| 4 | Satisfactory | S | 2 |
| 5 | Unsatisfactory | US | 1 |

e) Building up the matrix of consequences

On the base of technical data of boilers there will be done a matrix of consequences.

f) Building up the matrix of ratings

Considering the data from the matrix of consequences there will be done the matrix of ratings, by changing the consequences into ratings. In table 6 it is presented the matrix of ratings.

Table 6 Matrix of ratings

| | | **$C_j$** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **$V_i$** | **Name of types** | **C1** | **C2** | **C3** | **C4** | **C5** | **C6** | **C7** | **C8** | **C9** |
| **$V_1$** | Cz1 | VW | VW | M | W | S | W | M | VW | VW |
| **$V_2$** | Cz2 | W | VW | W | M | M | M | W | M | W |
| **$V_3$** | Cz3 | W | W | M | S | W | M | S | S | S |
| **$V_4$** | Cz4 | M | W | W | W | VW | S | S | M | US |

*g) Building up the matrix of grades*

Ratings will be changed into grades, and these grades are enrolled in the matrix of grades (see tables 7 and 8). In table 7 there is presented the matrix of grades on the 1-10 scale.

Table 7 Matrix of grades on the 1 - 10 scale

| | | $C_j$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| $V_i$ | **Name of types** | **C1** | **C2** | **C3** | **C4** | **C5** | **C6** | **C7** | **C8** | **C9** |
| $V_1$ | Cz1 | 8 | 7 | 5 | 6 | 2,5 | 6 | 5 | 10 | 7 |
| $V_2$ | Cz2 | 6,5 | 7 | 6,5 | 5 | 5 | 5 | 6 | 5 | 6 |
| $V_3$ | Cz3 | 6,5 | 6 | 5 | 4 | 7,5 | 5 | 4 | 2,5 | 4 |
| $V_4$ | Cz4 | 5 | 6 | 6,5 | 6 | 10 | 4 | 4 | 5 | 3 |

In table 8 there is presented the matrix of grades on the 1-5 scale.

Table 8 Matrix of grades on 1-5 scale

| | | $C_j$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| $V_i$ | **Name of types** | **C1** | **C2** | **C3** | **C4** | **C5** | **C6** | **C7** | **C8** | **C9** |
| $V_1$ | Cz1 | 5 | 5 | 3 | 4 | 2 | 4 | 3 | 5 | 5 |
| $V_2$ | Cz2 | 4 | 5 | 4 | 3 | 3 | 3 | 4 | 3 | 4 |
| $V_3$ | Cz3 | 4 | 4 | 3 | 2 | 4 | 3 | 2 | 2 | 2 |
| $V_4$ | Cz4 | 3 | 4 | 4 | 4 | 5 | 2 | 2 | 3 | 1 |

*h) Calculus of global index*

Evaluation of boilers is done with the help of global index. The global index that measure the product quality is expressed as follows:

$$I = \sum Ni \cdot Kj \qquad (1)$$

where:

I is the global index that measure the installation quality, of the products, etc.;

Ni – note or grade obtained by using the performance criterium;

i = 1... n;

n – total number of performance criteria necessary for the chosen installation, and for the product, etc.;

Kj – index of importance for the performance criteria due to the importance of quality evaluation (the group);

j = 1...m;

m – number of groups in witch we recognize the quality criteria due to the importance created by the user demands [2, 4].

*i) The hierarchy of choices*

On the base of calculus (1) there comes up the calculus for global index. The choice with the highest value of the global index is considered to be the optimum solution.

In the case of grades choice by using the 1-10 scale, the dates will be presented in table 9.

Table 9 Calculus of global index in the case of grade choice when using grades on the 1-10 scale

| $V_i$ | Name of types | C1 | C2 | C3 | C4 | C5 | C6 | C7 | C8 | C9 | Global index | Place |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | **$K_j$** | | | | | | | | | | |
| | | 2 | 1 | 2 | 1 | 3 | 1 | 1 | 3 | 1 | | |
| | | **Grade** | | | | | | | | | | |
| **$V_1$** | Cz1 | 8 | 7 | 5 | 6 | 2,5 | 6 | 5 | 10 | 7 | 94,5 | 1 |
| **$V_2$** | Cz2 | 6,5 | 7 | 6,5 | 5 | 5 | 5 | 6 | 5 | 6 | 85 | 3 |
| **$V_3$** | Cz3 | 6,5 | 6 | 5 | 4 | 7,5 | 5 | 4 | 2,5 | 4 | 76 | 4 |
| **$V_4$** | Cz4 | 5 | 6 | 6,5 | 6 | 10 | 4 | 4 | 5 | 3 | 91 | 2 |

On the analysis of data presented in table 9, it is easily seen a clear difference of choices, because the values of the global index vary between 76 and 94,5. In the case of the grade choice by using the 1-5 scale, the data will be presented in table 10.

Table 10 Calculus of the global index in the case of grade choice on the 1-5 scale

| $V_i$ | Name of types | C1 | C2 | C3 | C4 | C5 | C6 | C7 | C8 | C9 | Global index | Place |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | **$K_j$** | | | | | | | | | | |
| | | 2 | 1 | 2 | 1 | 3 | 1 | 1 | 3 | 1 | | |
| | | **Grade** | | | | | | | | | | |
| **$V_1$** | Cz1 | 5 | 5 | 3 | 4 | 2 | 4 | 3 | 5 | 5 | 58 | 1 |
| **$V_2$** | Cz2 | 4 | 5 | 4 | 3 | 3 | 3 | 4 | 3 | 4 | 53 | 2 |
| **$V_3$** | Cz3 | 4 | 4 | 3 | 2 | 4 | 3 | 2 | 2 | 2 | 45 | 4 |
| **$V_4$** | Cz4 | 3 | 4 | 4 | 4 | 5 | 2 | 2 | 3 | 1 | 51 | 3 |

On the analysis of the data presented in table 10, it is clearly seen a difference of choices, because the values of the global index are between 45 and 58.

## 3. Conclusions

a) Conclusions as the result of the preceding study. The results of the calculus are centralized in table 11.

Table11 Final table that centralizes the boiler choices

| $V_i$ | Name of types | Method: Global evaluation method of performances | | Method: Electre Method [1] |
|---|---|---|---|---|
| | | Grade1-10 | Grade 1-5 | |
| | | Place | | |
| **$V_1$** | Cz1 | 1 | 1 | 1 |
| **$V_2$** | Cz2 | 3 | 2 | |
| **$V_3$** | Cz3 | 4 | 4 | |
| **$V_4$** | Cz4 | 2 | 3 | 1 |

The following conclusions are made in table 11:

- when using grades on the 1-10 scale, the best boiler is Cz1;
- when using grades on the 1-5 scale, the best boiler is Cz1;
- when using Electre method (see paper 1), the best boilers are Cz1 and Cz4;
- using different methods brings different results;
- using different scales can also bring different results;
- when using different scales the final classification can be almost the same for all factors involved.

b) The use range of the method presented in the above study

The presented method can be used for any kind of products, and the number of the products of the same type can be theoretically unlimited.

## 4. References

[1] Badea, G. ş.a., *Alegerea echipamentelor de instalații - o problemă multicriterială.* A XXXIII-a Conferință Națională de Instalații. 20-23 octombrie 1998, Sinaia, România. Pagina 15-20;

[2] Berbecaru, D., *Elemente de evaluare pentru un top al echipamentelor de instalații*. Pagina 35-37. A XXX-A Conferință de instalații. Sinaia 16-18 octombrie, 1996;

[3] Munteanu, A., *Optimizarea constructivă a rețelei electrice de distribuție din incinta consumatorului*. Teză de doctorat. Universitatea Tehnică din Cluj-Napoca – Facultatea de Construcții. Martie 2003. Pagina 6-60;

[4] ***, Ordinul nr. 937 din 02.07.2002 al Ministerului Lucrărilor Publice, Transporturilor şi Locuinței, pentru aprobarea reglementării tehnice „*Ghid pentru determinarea performanțelor energetice ale instalațiilor de încălzire şi de apă caldă de consum din clădirile social-culturale existente, în vederea reabilitării şi modernizării acestora*'', indicativ GT 038-02. Pagina 115-117

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Structural Behavior of Corrugated Web Cold-formed Girders

Gîlia Lucian-Dumitru[1], Nagy Zsolt[2], Păcurar Vasile[3]

[1,2,3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

## Abstract

*The objective of this paper is present a new structural solution for the trusses of the pitched roof portal frames of industrial buildings. Cold-formed steel members are used in a wide variety of applications, both residential and industrial, due to their high strength to weight ratio. The girder solution proposed in this paper is composed of back to back cold-formed C cross-sections acting as top and bottom chords, and corrugated sheeting acting as the girder web. The girder components are described, as well as the stress state of each member of the girder. Also several finite element based analyses have been carried out. These analyses point out the serviceability of some truss configurations. Several remarks are stated resulted from the FEM-based analyses. The paper concludes by briefly stating the technologic and economic advantages of the proposed cold-formed girders.*

## Rezumat

*Scopul acestui articol este să prezinte o solutie structurală inovatoare pentru structura principală a acoperisului halelor metalice parter. Profilele subtiri formate au o aplicabilitate largă, atât in domeniul rezidential cât si în cel industrial, în mare parte datorându-se raportului mare rezistentă-greutate. Ferma propusă în acest articol este compusă din profile C formate la rece asezate spate în spate atât la partea superioară cât si la partea inferioară, si fasii de tablă cutată cu rol de inimă a grinzii. Se prezintă componentele grinzii, rolul lor în subamsamblul structural si starea de eforturi a fiecărui component. De asemenea au fost efectuate câteva analize bazate pe metoda elementului finit, fiind relevate starea limită de serviciu pentru diferite configuratii de ferme. Articolul concluzionează prin enuntarea succintă a avantajelor tehnico-economice a fermelor propuse.*

**Keywords:** cold-formed steel, steel girders, stressed skin sheeting, finite-element analysis, modified Riks analysis, serviceability limit

* PhD Student, Tel./ Fax.: +40722266923
E-mail address: lucian.gilia@bmt.utcluj.ro

# 1. Introduction

Structural steelwork has two main families of members. The first family is comprised of members that have hot-rolled shapes and/or built-up sections extensively used structural steel framing. The second category, less familiar but of growing importance, is comprised of members that made of by cold forming thin steel sheets, strips, flat bars etc. either by folding, cold rolling or by press braking [1]. The main advantages of cold-formed members are their lightness, high strength to weight ratio, ease of fabrication and mass production, practically unlimited number of sections, fast erection and installation, more economic transportation and handling, due to compact packaging possibilities, etc.

Cold formed steel structural members can be divided in two main categories:

- Individual structural framing members – thickness raging from 1mm to 8mm
- Plates and decks – thickness raging from 0.5mm to 5mm[2]

The trusses proposed in this paper combine both categories of members, in a way that both are used for their specific purpose. The proposed girders are made of back to back cold C-sections acting as top and bottom chords and first generation trapezoidal sheeting acting as the girder's web. The connection between the chords and the web is made by self-tapping SD3 screws with either steel or neoprene washer in every rib of the sheeting, also the overlapping connection of the sheeting is made by self-drilling, self-tapping SL3 screws, complying with technical catalogue recommendations [3].

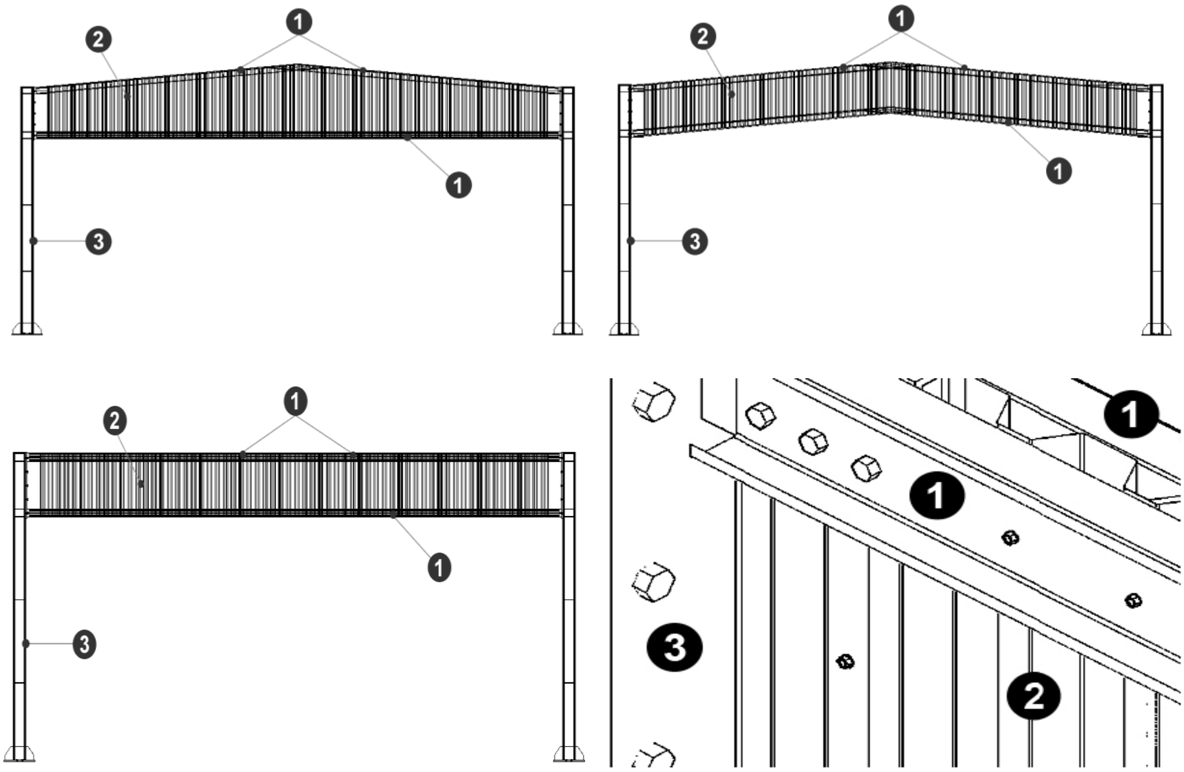



Figure 1. Several configurations of the proposed corrugated web trusses:
1 – back to back C-section cold-formed members; 2 – trapezoidal sheeting; 3 – hot rolled section columns.

The corrugated web cold-formed girders presented in this paper have the same applicability as the classical cold formed trusses that are used for the roof structure of low-rise buildings, spanning from 12 to 24 meters. Previous studies about cold-formed trusses have focused on load bearing [4, 5, 6, 7, 8] and influence of joints stiffness [9, 10] making this system more familiar and easier to use by structural designers.

Due to high stress concentrations in the joints near the supports, some members are designed according to connection related requirements – the member has to be large enough to accommodate a desired number of screws. Also because of their considerable height, the compressed lower chord is highly prone to instability; this can also lead to large cross-sectional members. Therefore the use of trapezoidal sheeting connected in every corrugation to both top and bottom chords prevents the afore mentioned issues. Thus, a more even stress pattern is achieved throughout the girder and smaller cross-section members are used for the bottom chords.

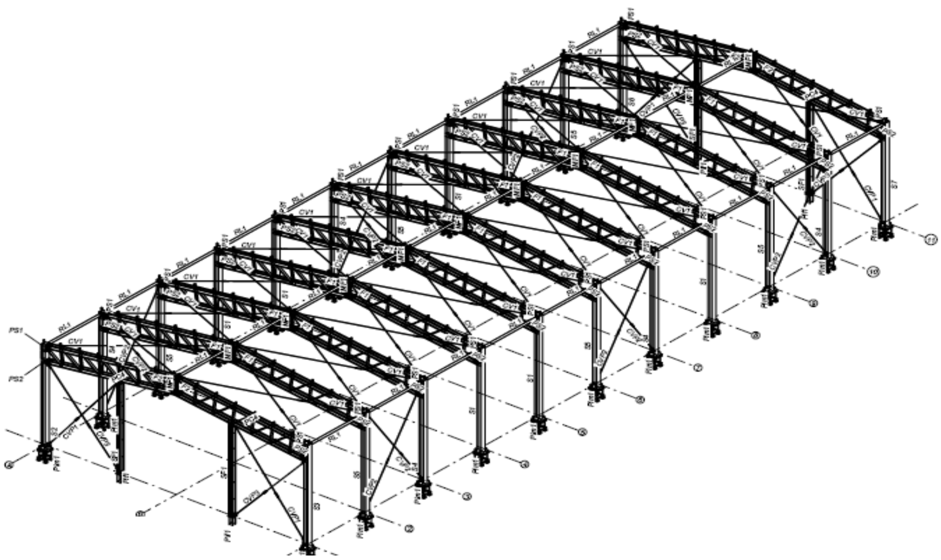


Figure 2. General view of a industrial structure with cold formed steel trusses and hot rolled steel columns

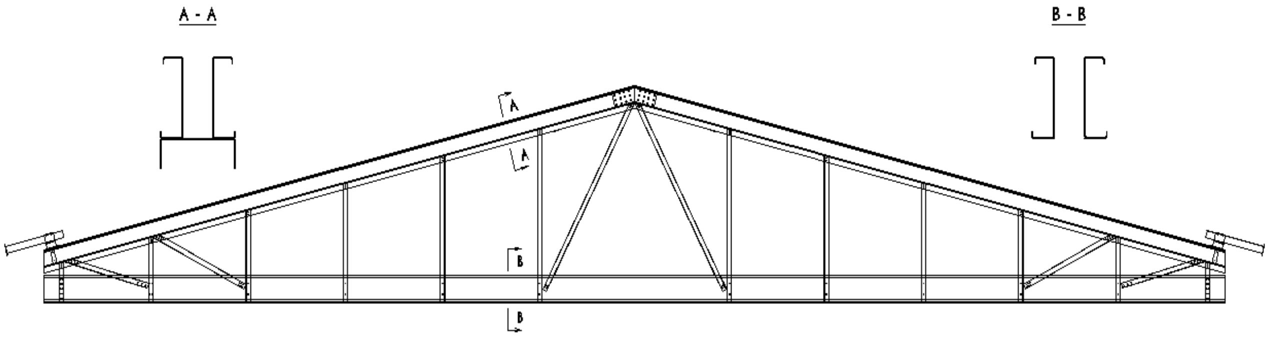



Figure 3. Triangular cold-formed steel truss made of both C and U cross-sections

## 2. Short literature survey

Corrugated sheets have been used in building construction since about 1974 and are one of the oldest types of cold-formed steel products. Applications of corrugated sheets include roofing and siding in buildings, flooring systems, curtain walls, steel pipelines, guardrails, conveyer covers, bridge constructions and other purposes.

In many cases they are used as shear walls to enhance or replace conventional bracing of entire structures or individual members – the effect of stressed skin diaphragm. Corrugated or trapezoidal sheets have also been used as web elements for built-up girders, in order to increase the web stiffness, thus avoiding the use of thicker plates, or additional stiffeners. Such is the case of Macomber Panlweb girder [11] shown in Figure 4, which is similar to the girders proposed by the authors.

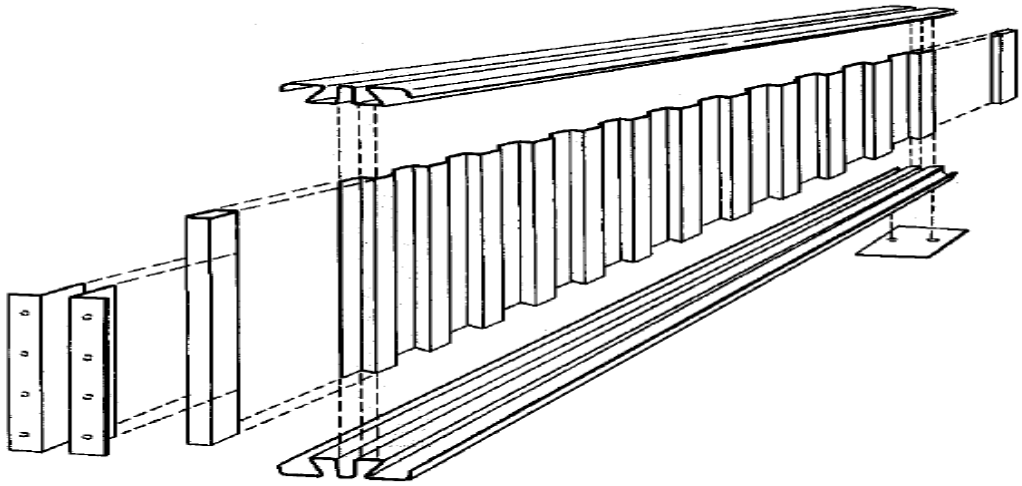


Figure 4. Macomber Panlweb Girder

The structural performance of steel diaphragms depends on sectional configuration of panels, connection arrangement, strength and thickness of the sheeting, span length, load function and is regarded as a non-trivial and complex topic. The shear strength and stiffness can be evaluated by analytical or by test procedures. Research on the subject started in 1947 with numerous tests conducted by a number of researchers and engineers; work up until 1960 was summarized by Nilson [12]. From 1962 a research project was conducted at Cornell University under the sponsorship of AISI (American Iron and Steel Institute) culminating in 1967 with the publication of “Design of Light Gage Steel Diaphragms” [13].Additional research on this subject the work of Winter, Luttrell and Apparao [14], Easley and McFarland [15], Chern and Jorgenson [16], Fisher and LaBoube [17] and others.

In Europe, the main research projects have been conducted by Bryan and Davies. The shear diaphragm action of steel panels in framed buildings has been well illustrated in Davies and Bryan’s book on stressed skin diaphragm design [18]. Other recent research on steel shear diaphragms includes the work of Elgaaly [19, 20], Pekoz [21, 22] and others.

Analytical relations to evaluate the design strength of the presented trusses have been summed up in previous studies by Neagoie, Ungureanu and Dubina [23]. These relations along with several relations, from the design code of corrugated sheet diaphragms [24, 25], have been used by the author to design the trusses analyzed by finite-element.

## 3. Finite element analyses

### *2.1 Geometry and mesh definitions and element types*

Finite element method or FEM is a numerical method used mainly to solve a number of differential equations which describe a physical phenomenon. From an engineering standpoint, FEM is a method for solving engineering problems such as stress analysis, heat transfer, fluid flow, electromagnetics, etc. by computer simulations.

This paper presents the results of several FEM stress analyses of the previously mentioned corrugated web girders. General purpose, FEM based package ABAQUS [26] was chosen to carry the present analyses. All analyses were geometrically nonlinear and static using modified Riks method. In the analyses three main types of finite elements have been used to mesh the steel members used in the studied model:

- S8R – a 8-node, doubly curved thick shell element with reduced integration, used for the all cold formed members and the trapezoidal sheeting

- C3D20R – a 20-node, quadratic brick with reduced integration used for the plates that connect the girder to the columns
- C3D8R – a 8-node, quadratic tetrahedron used where structured meshing was not an option

Also line spring elements have been used to model the self-tapping and self-drilling screws between the cold-formed members.

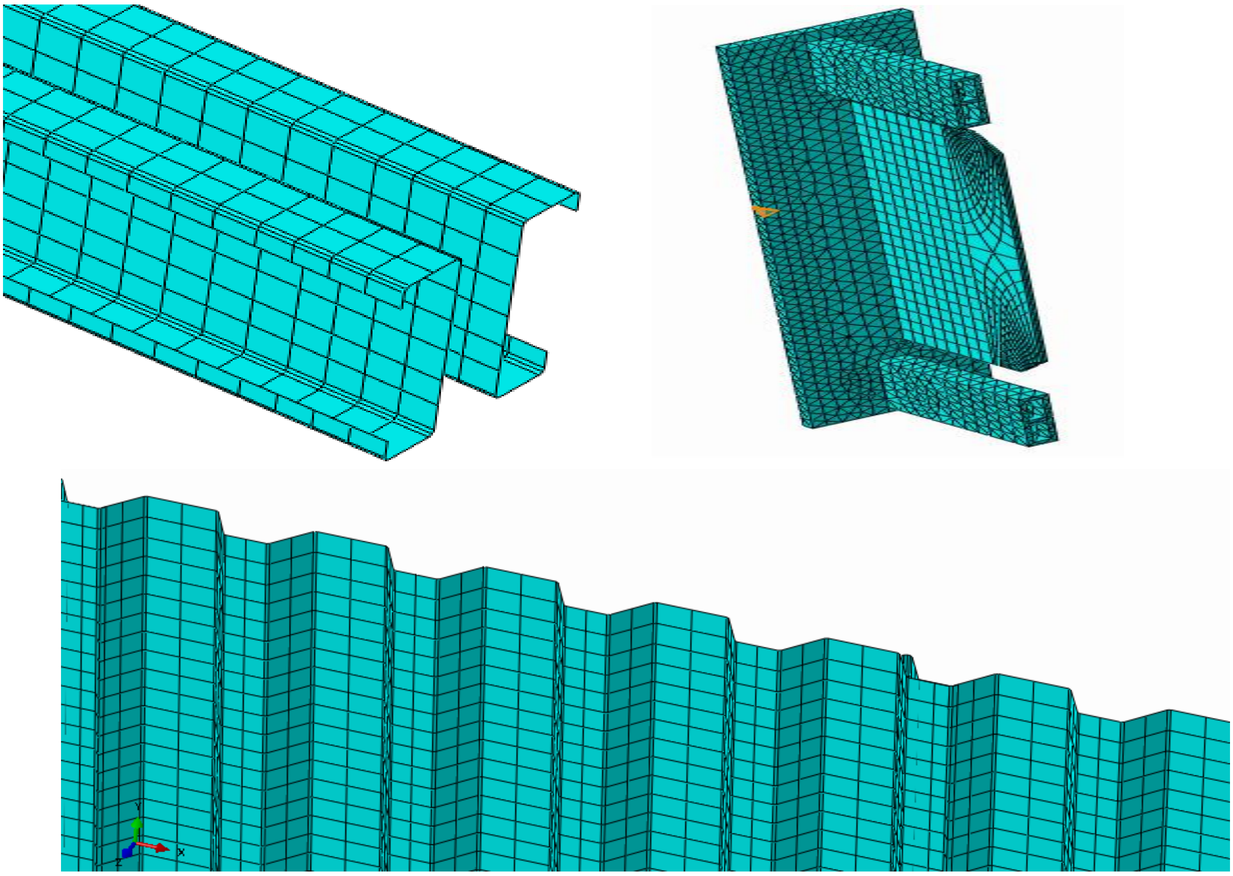


Figure 5. Meshed parts of the model

The model studied in this paper is the general serviceability of a girder acting as primary beam of a single-storey industrial frame. All analyzed girders had constant cross-section height. Discrete rigid parts have been used in order to apply the desired loads on the girder. These rigid elements represent the roof purlin action and are modeled by shell linear elements. The active width of the rigid parts are in accordance to actual cross-section with of Z250/2.5 designed for 6 meters bays.

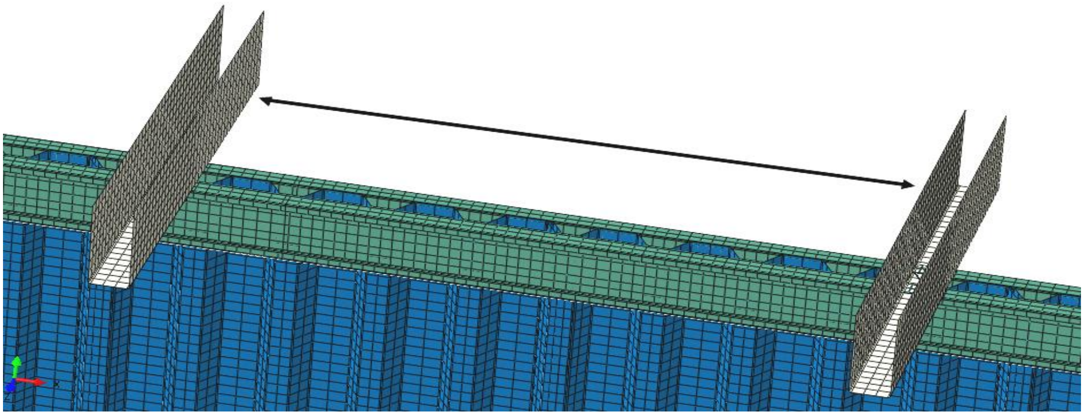


Figure 6. Discrete rigid purlins used to load the girder

### *2.2 Material properties*

S355G+D steel grade was used for all parts. Because the finite element program uses real stresses and real strains, a conversion of pre-measured engineering stress and strains had to be carried using the following formulas, and have been presented in Table 1. The material used is considered isotropic, with elastic modulus of 210.000 MPa and Poisson ratio of 0.3. Work-hardening due to the cold-forming process has not been considered in this paper.

$$\sigma_r = \sigma_e(1+\varepsilon_r) \qquad \text{Eq. (1)}$$

$$\varepsilon_r = \ln(1+\varepsilon_e) \qquad \text{Eq. (2)}$$

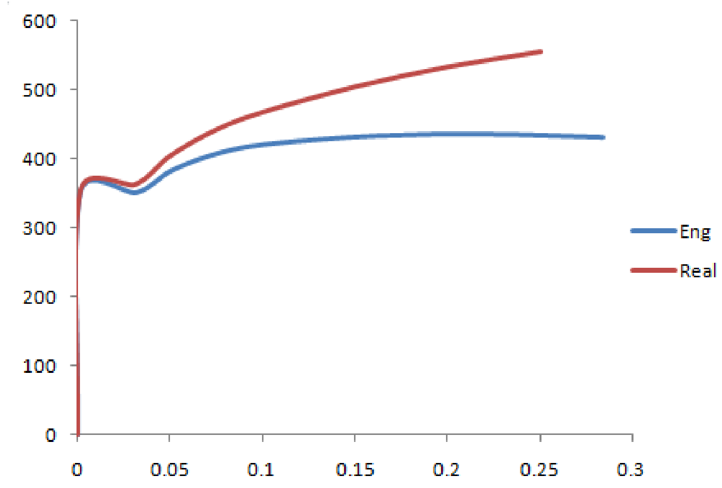



Figure 7. Engineering and real stress-strain S355G+D used in this FEA

### *2.3 Miscellaneous input data*

Fixed boundary conditions have been used for both end-supports on the outside of each end-plate Also pinned boundary conditions were used at the mid-span of the bottom chord to prevent lateral-torsion buckling. General contact property with both normal and tangential penalty behavior was used. Self-drilling and self-tapping screws were modeled by spring element with elastic and failure behavior provided by experimental testing [27, 28]. To trigger the correct non-linear response from the loaded girder, imperfections were seeded to the web and chords with respect to the guidelines of Schafer and Pekoz study [29] and in accordance to essential manufacturing tolerances for cold formed members and profiled sheets stated in EN 1090, Part 2 [30].

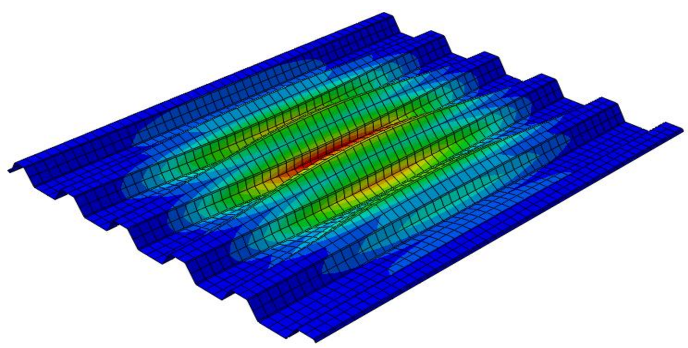


Figure 8. The shape of the imperfection seeded to the sheeting corresponding to the 1st Eigen mode

The effect of membrane and bending residual stresses is not considered into this analysis.

### *2.3 FEM analysis and results*

A total of 9 analyses have been carried on using arc-length, modified Riks method, using artificial damping. The modified Riks method is suitable for geometrically nonlinear static problems that involve buckling or collapse behavior, where the load-displacement can exhibit negative stiffness and the structure must release strain energy to remain in equilibrium.
Corrugated web girders have been set up on 3 different spans – 12,15 and respectively 18 meters and height to span ratios of 1/10, 1/15 an respectively 1/20.
Results of the FEM analyses have been summarized in the table below.

| Nomenclature | Span [m] | Cross-section height [mm] | Peak Load [kN] | Midspan Vertical Deflection at Peak Load [mm] |
|---|---|---|---|---|
| GCW1 | 12,00 | 1150 | 451.10 | 21.48 |
| GCW2 | 12,00 | 850 | 409.12 | 24.67 |
| GCW3 | 12,00 | 650 | 369.18 | 25.85 |
| GCW4 | 15,00 | 1450 | 577.62 | 26.61 |
| GCW5 | 15,00 | 1050 | 503.17 | 29.96 |
| GCW6 | 15,00 | 750 | 440.64 | 34.06 |
| GCW7 | 18,00 | 1750 | 693.85 | 31.99 |
| GCW8 | 18,00 | 1150 | 608.37 | 43.43 |
| GCW9 | 18,00 | 850 | 535.20 | 55.48 |

Table 1. Peak loads and vertical deflections of the analyzed girders

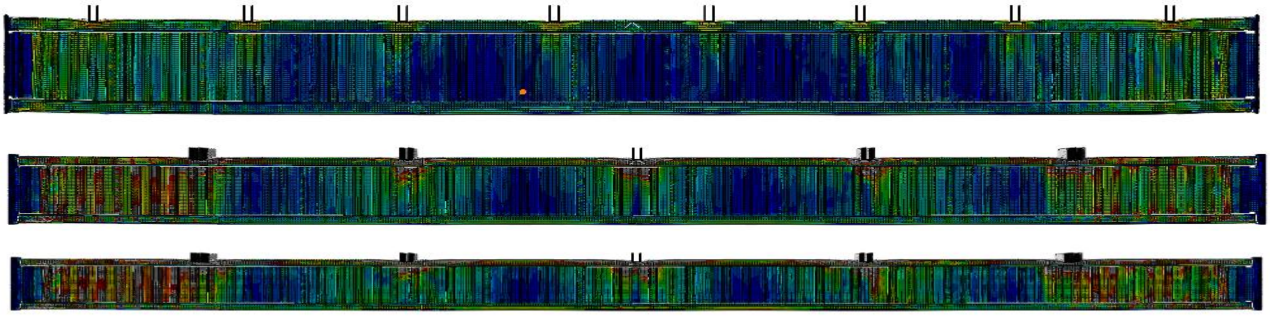


Figure 8. Overviews with the stress patterns of 3 of the analyzed girders

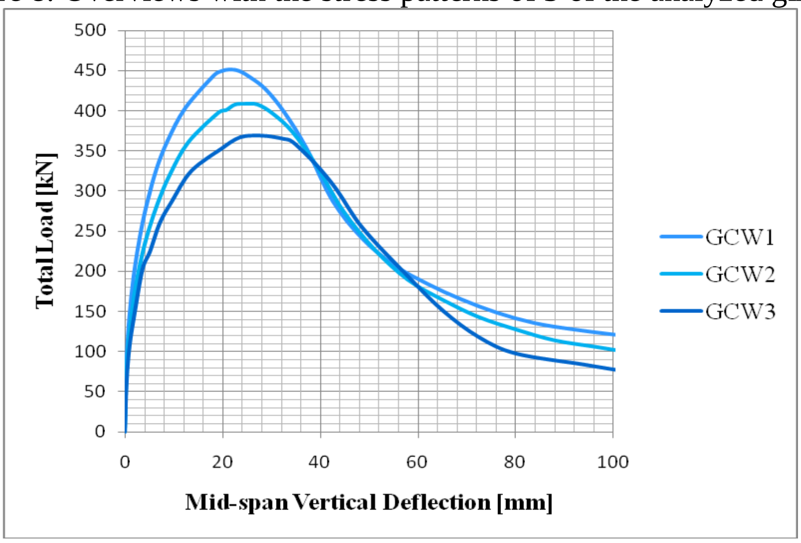



Figure 9. Load-deflection curves resulted from FEA of 12 meters span girders

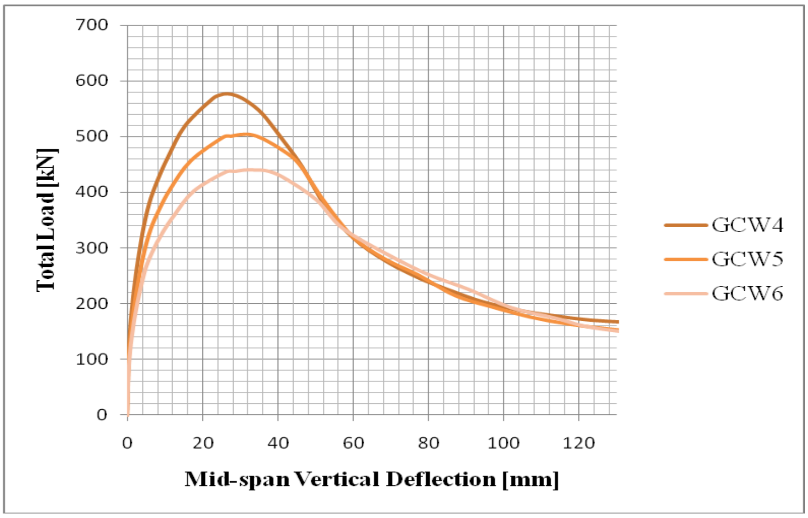



Figure 10. Load-deflection curves resulted from FEA of 15 meters span girders

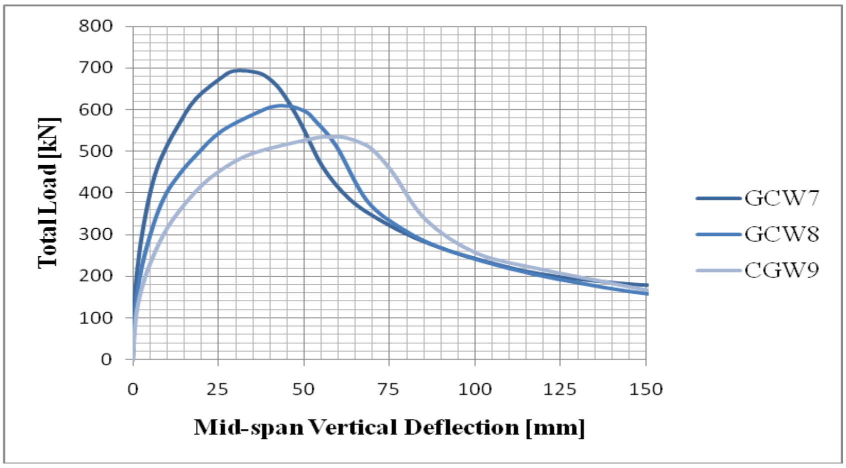



Figure 11. Load-deflection curves resulted from FEA of 18 meters span girders

The three main failure modes discovered are:

- failure due to the distortion of the web sheeting near the supports combined with the web-crippling of the upper chords
- failure due to the shear buckle of the sheeting
- failure along the first line of seam fasteners near the supports

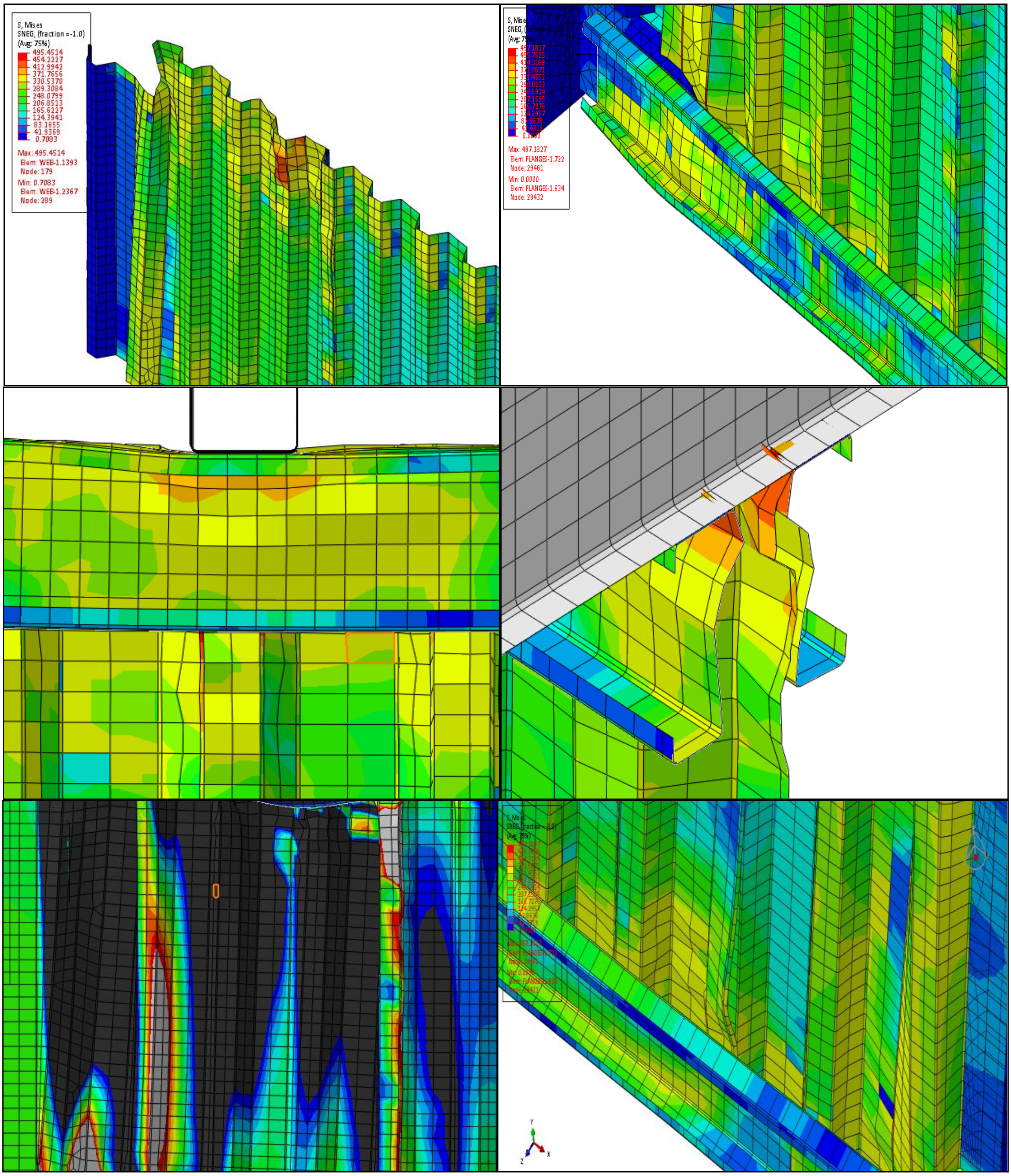



Figure 12. Overviews of the failure modes resulted from FEA: sheeting section distortion, web crippling of the upper members, shear buckle of the sheeting, failure along seams

## 5. Conclusions

The assumption that corrugated web girders have satisfactory load bearing capacity is partly proven with the help of finite element analyses. The studied solution can be an alternative to the classic cold-formed trusses, and if properly designed can reduce the steel use from 15% to 25% at the roof structure. The results of the analyses are in accordance with adapted analytical design calculations. Future work will include several parametric studies with FEA and influence of the effect of residual stresses and work-hardening at the corners present due to the cold-forming process, which was not accounted for in the present study.

## 6. References

[1] Yu WW. "Cold-formed Steel Design", 3rd ed. Willey, 2000.
[2] Dubina D, Ungureanu V, Zaharia R, Nagy Zs. "Calculul si proiectarea constructiilor din profile metalice cu pereti subtiri formate la rece", Vol. I. AMM, Colectia Lindab, Bucuresti, 2004.
[3] SFS Intec. "Technical Values for Fastener from SFS Intec", 2005.
[4] Wood JV. "Full-Scale Test Behavior of Cold-Formed Steel Roof Trusses", ASCE Journal of Structural Engineering, Vol. 132, No. 4, April, 2006.
[5] Blumel S, Fontana M. "Load-bearing and deformation behavior of truss joints using thin walled pentagon cross-sections", Thin-Walled Structures Vol. 42, 2004.
[6] LaBoube RA, Yu WW. "Recent research and developments in cold-formed steel framing", Thin-Walled Structures Vol. 32, 1998.
[7] Pedreschi RF, Sinha BP. "An experimental study of cold-formed steel trusses using mechanical clinching", Construction and Building Materials Vol. 22, 2008.
[8] de Andrade SAL, da S. Vellasco PCG, da Silva JGS, de Lima LRO, D'Este AV, "Tubular space trusses with simple and reinforced end-flattened nodes – an overview and experiments", Journal of Constructional Steel Research Vol. 61, 2005.
[9] Zaharia R, Dubina D. "Stiffness of joints in bolted connected cold-formed steel trusses", Journal of Constructional Steel Research Vol. 62, 2006.
[10] Lutz DG, LaBoube. "Behavior of thin gusset plates in compression", Thin-Walled Structures Vol. 43, 2005.
[11] Macomber Inc."The Panlweb Girder", Canton, Ohio.
[12] Nilson, A H "Shear Diaphragms of Light Gage Steel", Journal of the Structural Division, ASCE Proceedings Vol. 86, 1960.
[13] Luttrell, L D "Structural Performance of Light Gage Steel Diaphragms", Report 319, Department of Structural Engineering, Cornell University, 1965.
[14] Apparao, TVSR. "Tests on Light Gage Steel Diaphragms", Report 328, Department of Structural Engineering, Cornell University, 1966.
[15] Easley JT, McFarland DE. "Buckling of Light-Gage Corrugated Metal Shear Diaphragms", Journal of the Structural Division, ASCE Proceedings, Vol. 95, 1969.
[16] Chern C, Jorgenson JL, "Shear Strength of Deep Corrugated Steel Panels", Proceedings of the 2nd Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, 1973.
[17] LaBoube RA, Fisher JM. "Diaphragm Strength of Composite Roof System", Proceedings of the 6th International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, 1982.
[18] Davies JM, Bryan ER. "Manual of Stressed Skin Diaphragm Design", Granada Publishing, New York, 1982.
[19] Caccese V, Elgaaly VM, Chen R. "Experimental Study of Thin Steel-Plate Shear Walls under Cyclic Load", Journal of Structural Engineering, ASCE, Vol. 119, 1993.
[20] Elgaaly M, Liu Y. "Analysis of Thin-Steel-Plate Shear Walls", Journal of Structural Engineering, Vol. 123, 1997.
[21] Kian T, Pekoz TB. "Evaluation of Industry-Type Bracing Details for Wall Stud Assemblies", Final Report, Cornell University, 1994.
[22] Miller TH, Pekoz TB, "Unstiffened Strip Approach for Perforated Wall Studs" Journal of Structural Engineering, ASCE, Vol. 120, 1994.

[23] Neagoie B, Ungureanu V, Dubina D. "Grinzi cu talpi din profile de otel cu pereti subtiri formate la rece si inima din tabla cutata", Stabilitatea si ductilitatea structurilor metalice - Preocupari actuale, Editura Mirton, Timisoara, 2006.
[24] NP 012/1997. „Normativ pentru calculul elementelor din profile de otel formate la rece", Buletinul Constructiilor, Vol. 15, 1998.
[25] NP 041/2000. "Normativ pentru calculul pentru constructii metalice cu diafragme din tabla cutata" Buletinul Constructiilor, Vol. 19-20, 2001.
[26] Hibbit, Karlsson, Sorrensen, Inc. "Abaqus/Standard User's Manual".

[27] Berry JE. “Sheeting Connections with mechanical fasteners”, 1976.
[28] Fullop LA, Ruff A, Dubina D. “Studiul experimental al imbinarilor profilelor de otel formate la rece folosind suruburi autofiletante” , Stabilitatea si ductilitatea structurilor metalice - Preocupari actuale, Editura Mirton, Timisoara, 2006.
[29] Schafer BW, Pekoz T. “Computational Modeling of Cold-Formed Steel: Characterizing Geometric Imperfections and Residual Stresses”
[30] BS EN 1090 – 2:2008, “Execution of steel structures and aluminum structures – Part 2: Technical requirements for steel structures”, 2008.

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Research and It's Place in Economical Development

Mircea I Rus*[1]

[1] *National Institute of Research-INCERC, Cluj-Napoca, Romania*

## Abstract

*Scientific research plays an important role in the evolution of the society in all of its aspects, therefore its interference with the innovation process should be also taken into consideration Both, the scientific research and the innovation process take part in the technological progress. The outcome of this progress has positive effects on the economical and financial activities at a global level, thus encouraging the spreading and the consolidation of globalization, as well as the development of the multidisciplinary aspect of scientific research. Technological progress generated by scientific research has led to the appearance of new areas in the economical field of activities, but it had also caused the extinction of some others which were obsolete. On a global scale, the activities are constantly changing because of tehnological progress, one of the main goals is finding an alternative power sources, and nevertheless, protect the environment, meaning that these power sources should generate a decrease in the global pollution level.*

## Rezumat

*Cercetarea ştiinţifică are un rol important în dezvoltarea societăţii din toate punctele de vedere: astfel, este bine de luat în considerare şi interferenţa acesteia cu procesul inovării care contribuie, împreună cu cercetarea ştiinţifică, la progresul tehnologic. Rezultatele acestui progres au efecte pozitive asupra activităţii economico-financiare la nivel global, contribuind prin aceasta la apariţia şi consolidarea globalizării, precum şi la caracterul multidiscplinar al cercetării ştiinţifice. Progresul tehnologic generat de cercetarea ştiinţifică a dus la apariţia unor domenii din activitatea economică dar şi la dispariţia altor domenii care au fost depăşite de noile tehnologii. Activitatea la nivel global este într-o continuă schimbare datorită progresului tehnologic, iar unul din domeniile importante este de a găsi noi surse de energie şi, nu în ultimul rând, de a proteja mediul, adică aceste surse de energie să aibă ca efect scăderea nivelului de poluare la nivel mondial.*

**Keywords:** Research, Innovation, Economical development, Technological, Environment

## 1. Introduction

People have been forced by the need to survive and to improve living conditions to become creative, innovative in order to use the material and energy resources, time and labor more efficiently.
Practical needs have led to the emergence and development of certain scientific fields. Thus, astronomy arose from needs in agriculture, arithmetic –for reasons of evidence and exchange, geometry –from the measurement of land and construction, mechanics –from the need to ease labor and increase yields, and so on. Science and technology have known a more or less constant progress in time, contributing to the development of the human civilization. At times over the course of society's development, important

* Corresponding author: Tel./ Fax.: 0744478270
E-mail address: mircearus@yahoo.com

changes have occurred which led to a dramatic transformation of society. The end of migrations and the settlement of lands is the first such change, which set the whole process in motion. The second great change is the industrial revolution, which profoundly changed human society.

The invention of the steam engine by J. Watt (1788) can be regarded as the beginning of the industrial revolution. This invention was the starting point for the construction of more powerful machines, for the introduction of mechanization in the manufacturing process, for the opening of modern factories having a larger number of workers and experts, for the growth of production, and so on.

The current era is influenced by the new scientific and technical revolution in which technologies are rapidly changing, automation is expanding and information is rapidly processed and transmitted. Science and technologies are developing in an exponential rhythm and affect everything, from the household to cosmic space. The new scientific discoveries may contradict some previously established laws, such as the indivisibility of the atom, the theory of the formation of species, and so on. The time between the scientific discovery and its application in practice is decreasing.

Effective economic development requires a continuous process of modernization. Taking into consideration their rate of development, nations progress in terms of their competitive advantage and of their specific methods of competition. In the current era, competitiveness is no longer based only on primary production factors, like cheap labor and access to natural resources or investments. The economy is based on innovation, the dominant source of competitive advantage, meaning the capacity to create products and services that are innovative at the maximum limit of global technology using the most advanced methods.

New materials and structures have been discovered and have replaced traditional ones in mechanic, civil and industrial constructions. Subatomic particles and new chemical substances are being discovered, which will radically change the way of life and will help maintain the health of people.

The current scientific and technical revolution has brought about major changes:

- increased productivity;
- increased product quality;
- lower product costs;
- reduced consumption of raw materials and energy;
- investments directed mainly at the high-tech fields;
- reduced environmental pollution;
- rapid movement of information;
- developed communication between people;
- increased importance of the individual in the new society;
- more people engaged in activities of research;
- people adapting faster through trainings and re-trainings;
- improved quality of life;

The multidisciplinary character of certain phenomena highlights the idea of globalization in all its implications. Thus, globalization brings a series of advantages to the big corporations and consumers, such as:

- significant cost reduction due to the growth of production beyond certain limits;
- flexibility to produce, design, innovate in different countries, thus using labor force and existing resources at their maximum;

- opportunities to invest and move various activities out of the consortium so that the final product will be as competitive as possible regarding quality or price;
- quick delivery of cheap, high-quality products;
- greater mobility of workforce;

Globalization can also mean disadvantages related to:

- loss of state control over the economy and national territory;
- labor force less protected in terms of job security; this disadvantage can be countered through high qualification, increasing the possibility of being relocated to other geographical areas;
- higher polarization between rich and poor countries;
- poor countries can become attractive from the point of view of foreign investments only if they can provide considerable advantages related to work force and material resources;

Knowledge-based economy is considered to be the economy of the future – a broader concept that integrates innovation, informational society and human capital. This presupposes:

- wider use of intelligent systems assisting managers in making important decisions quickly;
- a drastic change in the number of people working in creating information and those working in material goods production;
- information will become merchandise, due to its value and codification;
- training will get better and better. In the future, a person will have to possess knowledge not only in the form of know-how, meaning physical and mental abilities (tuition, training, labor force, stress resistance), but also know-who (the ability to interact with persons who have the information);
- shortened validity time for products;
- cheaper products, more competitive, of better quality, with the lifetime set by the design and not by the materials used;
- all activities, including technical progress, will become more and more efficient;
- an increasing number of products will incorporate intelligent technology (microcomputers, mobile systems of communication, multimedia systems, and so on);
- different products will "communicate" between them, excluding man from uncreative routine tasks;

Due to fierce competition, big companies will form international consortia, which will share the risk of identifying, processing and launching new technologies. The organization of firms will be done after the network model, renouncing the pyramidal pattern. The productive activity will be oriented towards the poor areas, with cheap labor, the technological advanced countries keeping only the activities of invention, design and production of "labor" goods.

Physical work will be less valued, excepting the service system. Employees will be appreciated more for their native qualities like: creativity, spontaneity, rapidity in taking decisions. International, multidisciplinary and multicultural teams will be formed.

Science has begun to play an increasing role in industrial development, not only as a major source of knowledge for new technological activities in numerous industrial sectors (drugs, chemical, electronic industry) but also as an essential input in testing, evaluating and controlling the quality of products. Basic research has become the input of technological development, and the advancement of science is increasingly dependent on the development of the technique. Many of the great contemporary scientific programs have been possible only through the improvement of the working speed and memory capacity of computers. Along with the increasing level of complexity in science and technology, their delimitation has become more and more difficult due to multiple profound interrelations, multidisciplinarity and border domains.

For a long time this reality has been ignored due to the predominance, both in theory and in practice, of the linear model of inter-relation between science and technology. In this model, the scientific activities, autonomous by nature, lead to inventions that, if commercially successful, lead to innovations, which can then be spread to potential adaptors. The strategic implications of this vision have found materialization in the governmental support of research efforts in pure science, whose results were regarded as public goods, while technology was considered a private good.

With the blurring of boundaries between science and technology and the intensification of the reciprocal flow of information, the accent has moved, in theory and practice, from research and development to technological diffusion.

## 2. Research in modern society.

Building a knowledge-based economy has become a rational goal for any state. This is the road to competitiveness, growth and economical prosperity.

Economic science shows that a sustainable growth cannot be achieved only through investments and a stable macroeconomic medium. They need to be doubled by technical progress enhancing the value of capital and labor. Therefore, the shift from resource exploitation to knowledge exploitation represents the testing point for the change from cost-based competition to one based on final value.

Scientific research is a systematic and creative activity meant to increase the volume of knowledge, including knowledge about man, culture and the use of these informations for new applications.

The concept of scientific research is used today also under the name of research-development (R-D). Research, development and innovation represent, for any country, the engine for economical and social development. The common concern of all countries for science and scientific research seems to be a recognition of their role in the wellbeing of the human civilization.

Scientific research produces knowledge, which is largely incorporated in the technological products. In addition, scientific research multiplies by itself (because unlike material values, scientific ones are not exhausted and don't get wasted), it develops education and it educates, thus leading to social improvement.

Some features of scientific knowledge:

- is able to acumulate and multiply in time;
- by being used, knowledge consolidates, improves and completes itself;
- through acumulation it becomes a free source of creative power at the use of people;
- is not alienated by trasmission from one person to another;
- usually remains in the possession of those who created specific knowledge;

Scientific research needs researchers, meaning specialists in various scientific fields, equipments, financial resources and infrastructure. There are two ways in which scientific information can be created and exploited, namely:

- the existence of an organizational framework and of means to collect, store, process and trasmit scientific information;
- the creation of new scientific information through research activity conducted in labs, research institutes and academies;

The results of research can be: inventions, innovations, new materials, computer programs (software), equipments, technologies, modern systems of management, of staff training and so on.

Like numerous other issues, current scientific research must be viewed in the context of the globalization phenomenon. There are real global problems in the field of research as well:

- science and research must take into consideration: the globalization of economic life, the accentuation of the international division of labor, the intensification of international relations, the limited resources and their uneven distribution across the globe, environmental protection and the ensuring of a sustainable development for mankind;

- the need for scientific research is an acute problem and requires great efforts, which sometimes cannot be supported by a single state;
- sustained development issues require international intervention for their solving;

Scientific research that takes place in the Romanian research institutes is integrated, through results, scientific publications and international collaborations in the universal scientific research.
Quality scientific research is a vector for progress and development. The need for long and medium-term national strategies in the field of scientific research is obvious, as obvious as the need for a rigorous assessment of the actual state of scientific research and human potential involved in research.

The development level of society is determined, essentially, by the performance of its educational and research systems, by the education level of its citizens, the quality of the products of the research activity, and by equitable access for all potential users to the services and products of this system.
Universities play an important role in producing, transmitting and using knowledge. Scientific research is a must for any effective educational system and is essential for the development of the high education itself.

The Bologna process has highlighted the need to stimulate performance, competitiveness and excellence, internationalization and globalization of research as well as the need to establish a European Area of Higher Education and Research, based on knowledge, fundamental for a competitive society,.
Scientific research activity represents the best way to develop human resources, both through the continuous training/information that it requires and the results obtained. It is an essential activity for solving the global issues of the society. Econometric analysis confirms the importance of the R-D activity for economic growth, for the ensuring of competitiveness as well as the importance of the macroeconomic environment, the openness of markets and the development of financial markets.
Scientific research is based on the creative potential of the people who are engaged in innovative activities. Innovation requires specific skills (such as the capacity to seize the market opportunities in correlation with the technological evolution, identify technical solutions, evaluate the costs-benefits report and the risks involved, find the necessary resources, and so on), skills to be gained by the employees, managers or to be incorporated in the organization of the company.
The innovation process involves five elements:

1. research system (located at the center of knowledge production);
2. innovative campaigns –the engines (the leaders) of innovation (companies that transform knowledge into market products);
3. infrastructure for innovation;
4. available capital and financing channels;
5. labor resources and educational services (human capital);

In reality, the five elements interweave in such a way that a research unit can also function as an innovation leader, or companies can have their own research units, and so on.

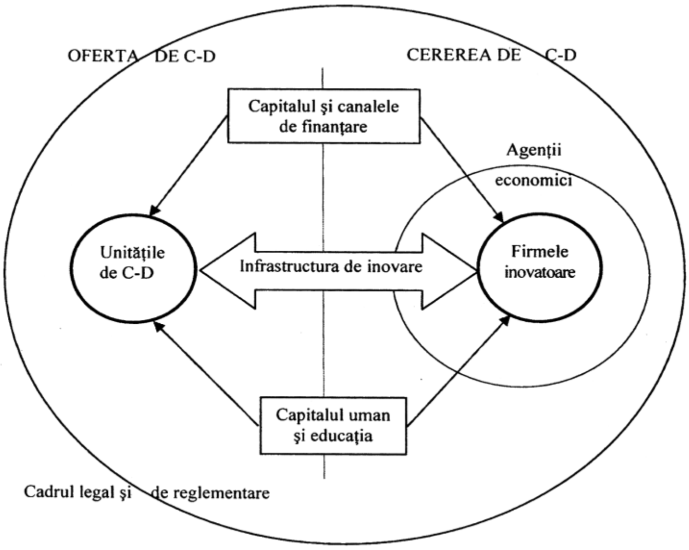



*Image 1.* The interdependece between the R-D activity, economic agents, infrastructure, finance and human capital.

The success of a knowledge-based economy depends on the way it works with the business community and the resources available to generate new products and processes.
To the greatest extent, research fields are orientated towards technologies, followed by natural sciences, exact sciences and human sciences.
Increased competence and efficiency of R-D activity can be achieved by reaching the following objectives:

- obtain interesting results for the economic/social beneficiaries;
- correlate the research topics funded from the budget with the sector strategy on long and medium term;
- promote partnership between researchers and the beneficiaries/users of the research results;
- support the implementation of the results obtained with beneficiaries/users;
- develop the human resources through training, for and by R-D activities, so as to be able to use the results of research and development;
- modernize the research and development infrastructure; form excellence centers in the main domains;
- expand the innovation infrastructure (innovation and business offices, technology transfer centers, offices for technological information);
- establish risk funds for the implementation of research results;
- set up a flexible system for the management of budget funds directed towards R-D programs;

The following types of activities can be financed from the national budget:

- R-D activities;
- technologic transfer activities: knowledge transfer, consulting and technical assistance;
- activities of exploitation/implementation of the results for the beneficiaries;
- horizontal support activities sustaining the achievement of the program by developing human resources coming from R-D activity and from the beneficiaries of the program;
- courses, trainings, disseminations of the acquired knowledge and experience;
- program management activities;

Private funding supports the development of skills in order to be able to act in this domain with chances to succeed in the implementation of innovative projects. This actualization of abilities is a necessary step, taking into consideration the fact that innovation offers higher rates of profitability and has become a necessary condition for long term development. The generation of scientific knowledge takes place in research centers and networks, which must have up to date equipment and software. The integration of Romanian R-D units in the international networks and programs is expected to occur. Research units can develop resources through grant programs for research. In addition to professional satisfaction and modern conditions of work, researchers must be attracted and motivated by a proper wages system. This will ensure the retention of Romanian intelligence and expertise, be they the product of universities, masters, Ph.D.s, and so on.

R-D units will have to increase their capacity to broadcast knowledge, results and experiences by developing marketing services, offices to ensure contact with the industry and through a public activity of promotion (via product catalogs, news, informative and promotional publications, conferences, product/technology demonstrations, audio-video presentations, program/project launching, and so on).

R-D units will have to: increase their capacity to use specific technologic knowledge by augmenting the economic environment's capacity to absorb innovation as a result of certain courses and trainings for the beneficiaries/users of research and development results; sustain the mobility of researchers, specialists and that of students from the research institutes and universities towards companies; improve access for companies to informational facilities and scientific assistance services.

The progress of the potential for research, development and innovation, at company level, can be achieved by elaborating R-D and innovation projects between industrial partners and R-D units. Another possibility could be the co-financing of collaborative projects between economic agents and R-D agencies. The introduction of quality management systems forces firms to improve their activity, and thus to perform activities of R-D and innovation.

## 3. Forms of scientific research

The scientific research activity is an important factor that contributes to the social and economic development. It can be considered the engine of social and economical progress; science and technology are key components of modern life that directly help countries in the achievement of social and economical objectives, in achieving sustainable development.

R&D activity takes the following main forms:

1. fundamental research is an experimental or theoretical activity aimed mainly at accumulating new knowledge concerning the fundamental aspects of observable phenomena and facts without taking into consideration a specific or particular application;
2. applied research corresponds to the work of innovation; it's aim is to acquire new knowledge to be put into practice; so this is an activity directed towards a practical and specific purpose: the creation of new products, processes, services or the significant improvement of existing ones;
3. development research (experimental) is a systematic activity that uses existing knowledge gained from research and/or practical experience to launch, in the fabrication process, new materials, products or devices, new processes, systems and services, or to substantially improve the existing ones.

Therefore, the activity of research aims to produce knowledge expressed mainly in publications. In scientific research a distinction between fundamental research, applied research and research for development and innovation is being operated. Thus, if knowledge refers to rules and principles, we can consider them the result of fundamental research published in publications. If knowledge refers to procedures or to the application of knowledge resulted from fundamental research to specific contexts, then we can discuss about applied scientific research, whose results appear in publications. When knowledge is processed sufficiently, so that it can be expressed in scientific publications which can be doubled by patents and prototypes, the patents and the registration of prototypes ensure commercial security to published knowledge. So, scientific research produces knowledge expressed in publications, doubled by patents and the registration of prototypes that provides commercial security.

Innovation is the activity that leads to the generation, assimilation and exploitation of R-D results in the social and economic sphere. The transformation of knowledge from publications, patents and prototypes (R&D) into technologies and services assimilated economically and socially (innovation) is not the specific purpose of the scientific research linked to the socio-economic field, but the transfer and dissemination of knowledge. In order to facilitate the transfer of knowledge from R&D to the socio-economic field, besides classical instruments linked to education, such as universities, specialized tools have been built, such as:

1. spin-offs;
2. technological platforms;
3. specialized organizations in which researchers and businessmen work together;

But in the latter situation, quite often the work of the researcher is not recorded as research but as an educative or economic one. So, the involvement of researchers in the innovative activity is frequently not quantified as a scientific activity but as a social or economic one from which they have no economic benefit. The three forms of R&D activity are interlinked and have no borders to define them. All of them require substantial financial resources.

The importance of the research activity is expressed in many official documents: thus, the activity of scientific research represents the highest mode of developing human resources, both through the continuous training/information required and the results obtained. Nevertheless, although scientific research is essential for development and for solving the global problems of society, it has been the first to be sacrificed in Romania, being considered a luxury compared to the emergencies of the moment.

The main direction of program development in this area is the creation of a national research and development system, as a resource multiplier factor, capable of supporting the development of the Romanian society.

In this field, the main areas of action are the following:

- reorganize the national scientific research system by defining the strategic fields and offering funds, especially in these fields, through the diversification of funding sources, better exploitation of research results and Romanian inventions; ensure closer links between research and the necessities of the economic agents;
- adapt the national scientific research system and the technological development system to the requirements of the European Union. The opportunities offered in the field by communitarian programs and the optimization based on the requirements of the informational society will be efficiently exploited.
- a hierarchy of priorities set in time for this field and the allocation of national funds will aim to reinforce links between the R-D system and the requirements for national economic progress, for rapid acquisition of R-D results to economic agents;
- well equipped and computerized laboratories to stand at levels comparable with their counterparts in the European Union. To achieve this, measures have been taken in order to transform certain centers and institutes into advanced development platforms, as institutes of national interest that will develop and valorize the existing research potential and will limit the danger of its dissipation in other sectors of activity or abroad;
- for the integration in the European structures, the promotion in Romania of certain projects of collaboration and the creation of scientific research centers resembling similar ones from other European countries has been taken into consideration, which will bring together Romanian specialists as well as foreign ones, especially in those fields in which there is a valuable scientific tradition and scientific and economic advantages are expected for Romania, including the conservation of the national environmental system and the valorization of the country’s natural potential;

- develop technological R-D activities at a regional level by stimulating the innovation and valorization of results obtained in the interdisciplinary territorial centers of R-D;
- ensure the contact between research and industry, at a national and regional level, by developing a specific institutional infrastructure meant to facilitate the transfer and valorize the results obtained through various R-D programs within the industry;

The international community is in a rapid development process. Romania must find a good position in the new international division of labor and must be prepared for the changes that the society must face. Scientific research, technological development and innovation are the heart of the knowledge-based economy, the key factor of development, competitiveness and job creation. Research contributes to the better understanding of human nature, the environment in which he develops and his role in this world.

## 4. The stages of scientific research

Scientific research is generally structured in a large number of phases and stages characteristic for the scientific process, and there is the possibility for other features to occur in accordance with the research field (technical, economic, social-politic, etc.).

The stages of scientific research are:

1. choice of research topic;
2. scientific documentation;
3. fulfillment of tasks;
4. writing;
5. valorization of scientific research results;

### *4.1. The choice of research topic*

The choice of research topic corresponds to the phase of „understanding the problem", of "problematization" or "definition and delimitation of the problem". The attention focuses on the following considerations (principles):

- complex tasks are fulfilled by research teams, sometimes having multidisciplinary membership;
- complex issues can be divided into sub-themes assigned to teams or individual researchers;
- researchers can chose their tasks after their specialization, experience, the resources they can make use of, the importance of the task, other preferences, motivation (the writing of a PhD thesis, of a scientific paper that will be published or presented in the country or abroad, and so on), minimal risk;
- researchers can come up with research topics that can participate in project auctions, which may become contracts in national or international programs, or can help them with their PhD, etc;
- minimum risk of failure (feasible subject);

For the economic research, the topics are selected from the economic problems arising from the confrontation of theory with empirical facts.

### *4.2. Scientific documentation*

Scientific documentation is determined by the need to know the current state of the research in the field, nationally and internationally. For example, researchers in economics must have knowledge of certain concepts, notions, categories, theories, indicators, methods of measurement and analysis.

Stages:

- bibliographic documentation (learning for the economists) is a mandatory step because no research happens on bare land, outside the existing national and international knowledge. Specialized literature should be read; it can be found in textbooks, treatises, encyclopediae, magazines, various studies, volumes on scientific manifestations, online publications, and so on;
- direct documentation aims to acquire information (statistics, facts) about a country, geographical region, domain, business, and so on. Information must be correct and rich in content;
- external expertise can ease the work and can shorten the time of research;

### *4.3. Fulfillment of tasks*

The most important stage of the research, in which:

- specialized works are critically analyzed;
- the economic reality is closely watched;
- hypotheses are formulated;
- experiments are put into practice;
- experimental results are interpreted;
- conclusions are drawn;

Inspiration can come during experiments. It can lead to new hypotheses to be tested and sometimes generalized. The mathematical device helps to better interpret the experimental results.

### *4.4. Writing*

The experimental data obtained are processed in the form of tables, graphics, are put into equations and in-equations and solutions that correspond with reality are searched (solution that are part of a system of values).

The next step is the actual writing of the scientific paper on a previously agreed plan. It should first present the current state of knowledge in the field, the experimental results obtained, their interpretation, conclusions and proposals.

### *4.5. Valorization of the scientific research results*

Scientific research results are handed in as reports that are sent to the research program that finances the research, or to a publisher for publication in the form of a monograph or magazine article, or arrive at the inventions office for publication as an invention patent. They may also be sent to a scientific m, where, as in the case of drafting doctorate theses, a public presentation of the research is held.

The results of research can also be in the form of consultancy conferred to certain beneficiaries, for the evaluation of their performance, proposals for the improvement of their activity, the protection of the environment, and so on.

The research team is organized based on the complexity of the theme. It is composed of specialists (researchers, professors, students, master students, doctoral students) and managers. On the whole, the responsibility rests with the project director.

At the subdivision of the theme, responsibilities are incumbent on the project partners and each individual researcher.

The main advantages of organizing as a team come from the working in parallel, which reduces the research term, contributes to the professional formation of the young, and produces research results that are better substantiated, analyzed and interpreted. It is absolutely necessary in case of major themes, which require inter and multidisciplinary research.The disadvantages have to do with the discipline within the team, which can limit initiative and the creative capacity of individual researchers.

As the case may be, a researcher can belong to more than one research team. Such a team functions only for the duration of the research on a particular theme.

## 5. Conclusions

Scientific research is present in any field of activity, be it economic, technical, sociopolitical, as well as cultural. Its results lie at the bottom of numerous discoveries in recent years, and their valorization has led to the overall improvement of living conditions. An increase in research activity has led to an increased number of researchers, which has led to an increase in available jobs, both in the research and adjacent fields. The standards of living of those involved in research have increased both materially and on a socioprofessional level, owing to the qualifications gained over time. Additionally, the results of the research activity come to be valorized in various branches of the industry (drugs, dyestuffs etc), which also leads to profitability for the industry. But, in order to obtain all these, one must first invest in research activity, and the results will follow. The multidisciplinary character of research is also worth mentioning, especially since it determines the collaboration of researchers across multiple fields of research, so that the results can be used concomitantly in the respective fields. This is one of the reasons which underlie globalization, which must also be looked upon in light of its positive effects, not just the negative ones.

Globalization has given certain researchers the possibility of working in research centers which they otherwise wouldn't have dreamed of, which allowed them to further perfect themselves, as a result of the research infrastructure made available.

The research infrastructure plays an essential role in the research activity - the better the infrastructure, the greater the chances of getting new results, which lead to new discoveries, which may someday materialize in the slowing down of aging, or the eradication of presently incurable diseases, or the use of new energy sources and the protection of the environment etc.

One mustn't forget that the research programs financed by the European Union (the Frame Programs) contribute abundantly to the collaboration between researchers from numerous UE and UE-associated countries, and researchers from other parts of the world, such as countries from Asia and Africa.

## 6. References

[1] Corlan, A. D., Cercetarea ştiinţifică nu este o activitate comercială, 2005, www.ad-astra.ro/journal

[2] Florescu, M. S., *Managementul cercetării ştiinţifice*, Bucharest, ASE, 2006

[3] Gheorghe, I. Gh., *Metodologia cercetării ştiinţifice, dezvoltării şi inovării*, Bucharest, Cefin, 2008

[4] Krausz, S., *Metodologia cercetării ştiinţifice*, Petroşani, Universitas, 2007

[5] Plumb, I., Vişan, S., Botez, L. F., Florescu, M. S., Angelescu, A., *Managementul cercetării şi inovării*, Bucharest, ASE, 2007

[6] Răboacă, Gh.,Comşa, M., Ciucur, D., *Metodologia cercetării ştiinţifice economice*, , Bucharest, Editura Fundaţiei România de Mâine, 2007

[7] Sandu, S., *Inovare, competenţă tehnologică şi creştere economică*, Bucharest, Expert, 2002

[8] Văcărel, I., Bistriceanu, Gh. D., Bercea, F., *Finanţe Publice*, Bucharest, The Vth Edition, Editura Didactică şi Pedagogică, 2006

*[9]* Institutul Naţional de Statistică, (2009), *Cercetare-dezvoltare în România, Colecţie de date statistice (2003-2008)*

*Acta Technica Napocensis: Civil Engineering & Architecture Vol. 53 (2010)*
*Journal homepage:* http://constructii.utcluj.ro/ActaCivilEng

# Fields of Research Activity

Mircea I Rus[*1]

[1] *National Institute of Research-INCERC, Cluj-Napoca, Romania*

## Abstract

*In the last decades the scientific research areas have become more diversified, which has led to an increase in the funds accorded for this kind of activity, and also to finding new solutions to the various problems mankind is facing. Important steps have been taken in finding alternative power sources, treatments for diseases thought to be untreatable, ending world hunger, and not least, ways to protect the environment. All of this wouldn't have been possible if the main scientific research fields hadn't been diversified and set to interfere with each other resulting in the multidisciplinary aspect of scientific research. We must be confident that in the future, the results of scientific research will offer answers and solutions in those fields and to those issues for which mankind has not yet found an answer.*

## Rezumat

*În ultimele decenii domeniile de cercetare ştiinţifică au devenit tot mai diversificate, ceea ce a avut ca efect atât creşterea sumelor alocate acesteia, dar şi apariţia unor soluţii la multiplele probleme cu care s-a confruntat şi se confruntă omenirea. S-au făcut paşi importanţi în găsirea unor surse alternative de energie, tratamente pentru boli considerate pană acum netratabile, găsirea unor metode de eradicare a foamei şi, nu în ultimul rând, posibilităţi de protejare a mediului. Toate acestea nu ar fi posibile dacă domeniile principale ale cercetării ştiinţifice nu ar fi fost diversificate şi să interfereze între ele, de aici rezultând şi caracterul multidiscplinar al cercetării ştiinţifice. Trebuie să fim încrezători că în viitor rezultatele cercetării să ofere răspunsuri şi soluţii la domeniile şi problemele care încă sunt necunoscute pentru omenire.*

**Keywords:** Research, Innovation, Economical development, Technological, Environment

## 1. INTRODUCTION

In any field, finances are essential for carrying out scientific research. Any legal entity, meaning any individual under the incidence of law, can apply to receive financial support.

In practical terms, this means that universities, research institutes, small and medium companies are equally eligible, as potential users of technologies and technologic applications. However, it is mandatory for each of them to meet the essential requirements in the rules of participation.

The sums can vary substantially, depending on the type of project, the number of partners involved, the ambition and the coverage area of the research.

The research-development activity has been credited as the starting point or as a decisive factor in the innovative process. Improved systems of ground, water and air propulsion have been discovered and continually improved, which made man's entrance into space possible; radio broadcasting and television have given new value to transmitting information, giving way to digital and three-dimensional television; the means of communication have improved dramatically, using satellite systems, ground,

[*] Corresponding author: Tel./ Fax.: 0744478270
E-mail address: mircearus@yahoo.com

water and air traffic can be more efficiently directed, terrestrial and sea space can be explored, military technique has greatly benefited.

Electronic systems of calculation are an innovation of the 20th century. Their rhythm of evolution gives a significant picture of the current information revolution. The technologies of information and communication (TIC) have become an integral part of everyday activity.

For the 20th century, studies conducted in developed countries of the European Union, USA and Japan have shown that the following scientific and technologic fields will have an important role: science and the technology of information; genetic technology; the technology of materials; energy; environmental science; brain science.

a). Science and technology will undergo a huge development, due to an increase in communication (simultaneously with the lowering of the costs of access to information), growth of computing capacity, greater use of artificial intelligence and expanding the technology of representation (the expansion of virtual reality to other fields). Virtual simulations and tests will be expanded to domains that are not related to the productive activity.

b). Genetic technology will lead to the eradication of some diseases, or disorders in the family line of compounds, thus correcting defective genes. It is even hoped to increase the IQ coefficient of certain nations by one or two points with each generation. With plants, genetic transformation could eliminate undesired features or add up new characteristics. The extension of cultivated areas to lands considered to be arid would be possible through genetic changes. Additionally, genetic engineering will help to combat incurable diseases from the past century: cancer, AIDS, heart and psycho-nervous diseases.

c) Technology tends to create “smart materials” with almost eligible functions. The concerns of the researchers focus on smaller and smaller construction items: nanoparticles, nanostructures, biologic materials, supraconducting materials at room temperature, and so on.

d) Energy research will focus on energy conservation, finding new sources of energy to replace fossil fuels (solar and nuclear energy are considered to be the future resources). Having this in mind, there is an attempt to reduce household energy consumption, taking into account the population growth (according to some researchers, the population of the globe will remain at the current figure of 6 billion, but according to others, the population will grow to 10 billion in this century and the energy consumption will be 47-70 times greater than now). Efforts are being made to reduce the fuel consumption of cars to 3 liters/km. Materials to ensure the conversion of solar energy into electric energy will be produced by using molecular nanotechnology. In nuclear energetics, studies for nuclear fusion reactions are being continued.

e) Environmental science will develop to “solve” problems like: uncontrollable genetic mutations; the reduced immunologic capacity of the human body; the spread cancer; the greenhouse effect; the destruction of the ozone layer and so forth.

f) Neurology will evolve in the direction of monitoring the functions of the brain, will develop possibilities to adjust, correct and repair brain functions.

An interesting phenomenon of recent years was generated by the spectacular development of science is the multidisciplinary study of certain facts. The trend is to transform structural science into integrative science. Integrative science will be based on new fundamental principles; will have at its disposition an integrative mathematics to represent the new theories in physical and informational reality, taking into account the complexity of life.

Within the globalization process, these changes occur simultaneously as the result of changes in the global economy. Thus, we are witnessing the disappearance of custom barriers, the emergence of multinational firms and increased competition in the internal and external markets.

## 2. Fields of scientific research financed in Romania

In Romania, the research activity is substantiated and financed based on programs, and in these programs the fields of research are explicitly presented.

These programs of research are more amply developed in the National Program for Research,

Development and Innovation 2007-2013 (NP II), legislated by HG 475/2007.
In establishing the programs of NP II, the aim was that of undertaking with priority concrete actions for increasing the number of researchers, improving their performances and increasing the attractiveness of pursuing a career in research. To this end, the Human Resources program was created.

In order to allow the researchers to work using efficient apparatuses, to benefit of an adequate management and to maintain a permanent relation with the socioeconomic needs, the Capacity program was created.

Having in view the importance of fundamental research in the development of knowledge and the fact that it ensures a solid foundation for applicative research and technologic development, both through ideas, and the capacity of training up the highly-qualified personnel required for these activities, the Ideas program was introduced in the Plan. Although for this program no priority fields are established, the accent being on excellence and international visibility, on research on the frontier of knowledge, interdisciplinarity and complex research in frontier fields, and on the participation in international research networks of excellence, as follows, the research fields with potential from Romania will be presented. Concentrating the investment in these areas, the program also sustains new fields, in which research groups from Romania collaborate internationally.

Through the fourth program called Partnerships in Prioritary Fields, which is the amplest program of the Plan, the aim was that of creating the conditions for a better collaboration between the various CDI entities, economic agents and/or units of public administration, in order to offer solutions to the problems identified in the research directions resulted from the large consultation undertaken within the exercise of foresight that took place in the period September 2005-May 2006. The majority of priorities of the public investment in research-development are of interest for the fundamental research as well. The public investment has in view the development of knowledge motivated by the strategic socioeconomic needs, and the research is evaluated with respect to its innovative capacity. Having in view the importance of finalizing researches by practical results, materialized in technical and technologic developments, the Innovation program was introduced in the Plan. This is going to support precompetitive and competitive research programs, as well as development of the infrastructure of innovation.

The Program for Sustaining Institutional Performance establishes mechanisms of institutional financing through competition, which to allow the efficient research entities, be them public or nonprofit, to implement their own strategies of development in accordance with the National Strategy for CDI. The evaluation of institutional performances is made with international participation, at 3-5 year intervals. This program will ensure the concentration of the resources and the institutional development necessary for obtaining performances on an international level.

The main objective of the IDEI (IDEAS) Program is the obtaining of peak scientific and technologic results, comparable to those at European level, reflected by the increase of visibility and international recognition of Romanian research.

The derived objectives of the program are the following:

- the continuous improvement of the performances visible on the international level, in those fields in which Romania has research potential and in which results comparable to those of other EU countries have been obtained;

- the development of those fields which Romania has interest of developing activities of scientific research in, with real contributions to the increase of the quality of knowledge, to technical and technologic development and the improvement of the quality of life.

The directions of action of this program comprise:

1) The sustaining of fundamental scientific research, of frontier and exploratory;
2) The organizing of "exploratory workshops" aimed at identifying unexplored niches of knowledge;
3) The launching of appeals for international collaborations on projects of fundamental research, of frontier or exploratory research.

The fundamental research areas that have potential in Romania are the following:

- biology, genetics and medicine;
- mathematics;

- physics and technologic physics;
- geology and atmospheric physics;
- border domains;

*BIOLOGY, GENETICS AND MEDICINE*

Research in this area concerns: the molecular investigation of viruses and bacteria having a major impact on human health, the molecular mechanism of immuno-genetics and histo-compatibility in organ and stem-cell transplants, the major diseases of the population (cardiovascular diseases, cancer, diabetes, obesity, degenerative diseases –fundamental and clinic research), biodiversity and biotechnology, genetics and physiology of the organisms' resistance to biotic and abiotic stress; genomic, trascriptomic, proteomics and metabolomics in normal and pathologic biologic processes.

*CHEMSTRY, ENVIRONMENT AND MATERIALS SCIENCE*

This domain refers to the science of nanosubstances, nanomaterials and nanotechnology applications, food chemistry, food quality and security, medicines, cosmetics, dyes, biomaterials and biocomposites, environmental quality and security, geochemistry of lithospheric processes, polluting processes, catalysis, catalysts and technique of decontamination, detection and identification of hazardous materials, high-resolution sensors, technologies to reduce and eliminate the contamination with CBRN agents, explosives and heavy metals.

*MATHEMATICS*

Mathematics includes combinatorial logic, theoretical informatics, commutative and non-commutative algebra, categories, number theory, representations of groups and algebras, algebraic and differential geometry and topology, complex geometry, real and complex functions, measurement and integrals, potential, functional analysis and operators, numerical analysis, differential equations with partial derivations, control and optimization, non-linear analysis, mathematical models of mechanics, thermodynamics, astronomy and the theory of particles and fields systems, biomathematics, probabilities, stochastic processes, mathematical statistics, operational and mathematic research in economy.

*PHYSICS AND TECHNOLOGIC PHYSICS*

Here we are dealing with the physics of the atomic nucleus, hadronic material and nuclear astrophysics; atomic, molecular and biomolecular processes; stable and radioactive isotopes, photonics, optics, physical processes and phenomena in condensed matter, quantum fields and elementary particles, particle and radiation interaction with substances, ionized environmental physics, of plasma and nuclear fusion, mathematic physics, information physics and quantum correlations, linear phenomena and chaos.

*GEOLOGY AND ATMOSPHERIC PHYSICS*

This domain comprises: petrogenetic, metalgenetic, paleontologenetic and mineral systems and models; the structure, the dynamics and the evolutions of the lithosphere (continents, seas and oceans), climatology, paleoclimatology and geochronology.

*BOARDER DOMAINS*

This field is narrower and makes reference to the modeling of physical, chemical, biological and geological processes, nanocomposition, core of Earth physics, environmental and cosmic space physics as well as knowledge-based economy.

## 2. Areas of research funded by the European Union through Program 7

The program of Technologic Research and Development 7, abbreviated PC7, is UE's main instrument for funding research in Europe and runs from 2007 until 2013.

PC7 supports research in priority areas in order to make UE a world leader in these sectors and answer the need for competitive employment in Europe.

PC7 consists of four main blocks of activity forming four specific programs, plus a fifth special program of nuclear research:

1) Collaborative cooperation-reasearch
- health;
- food, agriculture and biotechnology;
- informatics and communication technologies;
- nanosciences, nanotechnologies, materials and new production technologies;
- energy
- environment (including climate changes)
- transport (including aeronautics);
- socioeconomic sciences and humanities;
- security;
- space;

2) Ideas –European Research Council
- actions of frontier research; people –human potential, Marie Curie actions;
- initial training for researchers –Marie Curie networks;
- long term research and career development –individual scholarships;
- partnerships between academies and the industry;
- international dimension –incoming and outgoing scholarships, international scheme of cooperation, integration scholarships;
- exellence awards;

3) Capacities –Research capacities
- research infrastructures;
- research for the benefit of IMMs;
- knowledge areas;
- research potential;
- science in society;
- support for the coherent development of research politics;
- specific international cooperation activities;

4) Nuclear research and training
- fusion energy –ITER;
- nuclear fusion and radiation protection;

5) Joint research center
- direct actions in Euratom;
- non-nuclear actions;

The UE budget for PC7 is 50.5 billion Euros while the Euratom budget for five years is 2.7 billion Euros. In total, this represents an increase of 41% compared to PC7 and the prices in 2004, and a 63% growth compared to the prices in 2007. Budget division by activity is shown in Table1, the distribution on cooperation programs in Table2.

Table 1: The budget allocated to activities within FP7 (million euros)

| Cooperation | Ideas | People | Capacity | Euratom | JRC |
|---|---|---|---|---|---|
| **32365** | **7460** | **4728** | **4217** | **2751** | **1751** |

Table 2: Cooperation Program Budget (million euros)

| Health | Food, Agriculture and Biotechnology | Environment (including climate change) | Nano production | Energy | Transport (including Aeronautics) | Space | Security | Socio-economical sciences and Humanities | Information and Communication technologies |
|---|---|---|---|---|---|---|---|---|---|
| **6050** | **1935** | **1800** | **3500** | **2300** | **4180** | **1430** | **1350** | **610** | **9110** |

## 3. The objectives of research programs

### *3.1 HEALTH*

The objective of the research programs in the field of health is the health improvement of the European citizens, as well as the increase and intensifying of the competitiveness and innovative capacity of the industry, and of businesses in the field of health at European level. The global health issues will be analyzed, as for example forthcoming epidemics. The European collaboration with the developing countries will allow these countries to develop their own research capacities.
The European research program in the field of health will focus on:
- the transferring of the results of research, such as the transfer of fundamental discoveries to clinical applications;
- the development and validation of:
- new methods of treatment for promoting health and prevention, including the promoting of instruments for the diagnosis of normal aging;
- diagnosing instruments and medical technologies;
- durable and efficient systems of medical care.

The clinical research will engage with a number of diseases, such as cancer, cardiovascular, infectious, mental and neurological diseases, as well as Alzheimer's and Parkinson's diseases. International researches across multiple research centers, involving the requested number of patients, will make possible the creation of drugs and treatments in a shorter period of time.
The biomedical research at European level will contribute to an increase in the competitiveness of the European pharmaceutical industries and the system of medical care. Thus, it is imperative for EU to create an environment to lead to innovation in the public and private sectors.

### *3.2 ALIMENTATION, AGRICULTURE AND BIOTECHNOLOGIES*

The advancement of knowledge in durable management and the production and utilization of biological resources (microbiological, plants and animals) will ensure the basis for increasingly safer eco-efficient competitive products, as well as services for agriculture, fishing, human and animal alimentation, health and the wood industry. It is anticipated that there will be important contributions to the implementation of the present and perspective policies and reglementations in the public health sector, as well as in the animal and plant health and the consumer protection sectors.

The scientific research, industry and society will solve the social, economic and environmental issues and lead to the durable management of the biological resources. They will also make use of the progresses in microbial, plant and animal biotechnology in order to develop newer, healthier, eco-efficient and competitive products and services. Rural and coastal development will be aimed at, as well as the local economies, at the same time having in view the conservation of the patrimony and cultural diversity.

Research on the safety of the aliments and alimentary chains will be sustained, as well as on alimentary diseases, the preferences of the consumers in matters of food, and the impact on health of the aliments and nutrition.

The creation of a European bioeconomy will lead to efficiently utilizing the innovations and technologic transfer for all industries and economic sectors that produce, administer and explore the biological resources, as well as the afferent services from the supplying and consumer industries. These activities are aligned to the European strategy concerning the sciences of life and biotechnology, and are expected to promote the competitiveness of European agriculture and biotechnology, of companies specialized in the agroalimentary field, the highly technologic IMMs in particular, alongside with the improvement of social welfare and prosperity.

### *3.3 INFORMATION AND COMUNICATION TECHNOLOGY (ICT)*

ICT plays an important role in boosting knowledge, creativity and competitiveness in all industrial and service sectors, providing numerous opportunities for the European citizens and consumers.

ICT applications are diverse, including healthcare, transport systems, innovative interactive systems for recreation and learning. ICT innovations can help improve disease prevention and medical security; they can facilitate active participation of patients and can offer personalized care. Problems associated with the aging of the population can also be solved.

Under PC7, the research activities in the ICT field will cover strategic priorities in developed European industrial and technologic fields, such as: communication networks, applied computerization, nano-electronics and technologies such as in the audiovisual field.

The sectors that make intensive use of ICT include: industrial production, car engines, space sector, pharmaceuticals, medical equipments, food, financial services, media and retail. Benefits reported by firms, as a result of increased use of ICT, include: faster production development, lower costs and expenses, faster and more secure transactions, better relations with customers and suppliers, improved level of service, support for customers and increased collaboration opportunities.

### *3.4 NANOSCIENCES, NANOTECHNOLOGIES, MATERIALS AND NEW PRODUCTION TECHNOLOGIS*

Nanotechnologies bring new solutions and performance improvements in the production sector but also in health, medicine and agriculture.

The creation of new production processes may mean a reduction of polluting emissions and a more responsible use of natural resources. At the same time, product innovation seeks quality, novelty, less polluting vehicles, in order to cover the necessities of the population, improve the quality of life, wellbeing, reduce risks, and improve health. The promotion of sustainable consumption patterns will also lead to changes in the behavior of citizens, due to new risks and ethical issues. The ethical issues are related to human integrity and dignity (“chips” that monitor or control human behavior), as well as health and environmental-related risks.

a) Nanosciences and nanotechnologies –the objective is to create materials and systems with predefined properties and behavior, based on improved knowledge and rich experience, at nano scale. This will lead to a new generation of products and services for a large range of applications, at the same time minimizing any potential harmful environmental or health-related impact.

b) Materials – research will focus on creating new multifunctional surfaces and materials with adjusted properties and predictable performances for new products, processes and reparation.

c) New production – the basis for innovation in this field will be new knowledge and its application in sustainable production and consumption models. As such, suitable conditions for continuous innovation in industrial activities and production systems are provided, including design, constructions, equipment and services, for the development of generic production "goods" (technologies, organization and production facilities) while also taking into consideration safety and environmental protection.

d) technology integration for industrial applications – the integration of knowledge and technology from the three domains of research concerning nanomaterials, nanosciences and new production is essential for rushing the transformation of the European industry and economy. Research will focus on new applications and solutions as RTD solicits, requirements identified through the diverse European Technology Platforms.

Increased industrial competitiveness and high quality products will protect jobs and therefore promote social and economical cohesion. The new Technologic Platforms will highlight social aspects through their pan-European strategies. The overall aim will be to maximize the tax revenue in Europe. New regulations and standards have always been an adjacent product of the progress of industrial technology and these "platforms" will modernize and consolidate it in various areas of human activity.

### *3.5 ENERGY*

Energy systems are dealing with major challenges: global demand for energy, the finite character of oil and gas resources, the necessity to reduce the emission of greenhouse gases. Research in this field will seek to effectively reduce the devastating consequences of climate change, instability in oil prices and geopolitical instability in the oil-supplying regions.

Citizens will benefit from the energy research through more affordable prices, more efficient energy use and through the reduction of the climate-change causes.

Researchers will help transform the current energy sistem into one that is more sustainable and less dependent on import fuels. The final result will be a diversified mix of energy sources, particularly renewable-energy ones, energy carriers and non-polluting sources. Energetic efficiency, which includes rationing the energy use and the energy stocks, will be intensified in order to be able to face problems related to the security of supply and climate change.

Activities in the energy sector include:

- hydrogen and combustion cells;
- production of energy from renewable sources;
- production of renewabe fuels;
- renewable energy for heating and cooling;
- $CO_2$ capture and storage technologies for the production of energy without noxious emissions (no emissions);
- clean technologies for coil;
- smart energy systems;
- energetic efficiency and energy economy;
- knowledge for the elaboration process of energetic policies;

The European Industry has developed world leadership in terms of technology to generate and properly use energy. It is the pioneer of renewable energy technologies such as: solar energy, bioenergy and wind power. EU is also a global competitor in terms of energy regeneration and distribution technologies, and has a strong research capacity in carbon caption and isolation. To maintain this position, European industries must continue their efforts through international cooperation.

### *3.6 ENVIRONMENT AND CLIMATE CHANGE*

The increasing natural and human pressures on the environment and its resources require a coordinated pan-European and international approach.

We need a better understanding of the problems related to climate change and we have to identify environmental technologies in order to improve natural and artificial resource management. These activities will answer energetic policy needs related to long-term impact of EU policies; the continuation of the Kyoto protocol actions and post-Kyoto protocol actions referring to climate change.

Sustainable environment and resource management requires interdisciplinar research integrated so as to improve knowledge on the interactions between climate, biosphere, ecosystems and human activities, allowing the development of new technologies, instruments and environmental services.

The „Environment" program will be introduced in the following activities and fields:

*Climate change, pollutions and risk*

- pressures on the environment and climate;
- environment and health;
- natural hazards;

*Sustainable resource management*

Conservation and sustainable management of natural and artificial resources and of the biodiversity.

- management of the marine environment;

*Environmental technologies*

- environmental technologies for the observation, simulation, prevention, attenuation, adaptation, remediation and restoration of the natural and artificial environmental factors;
- protect, preserve and enhance the cultural heritage;
- evaluate, verify and test technologies;

*Earth observation and evaluation means*

Earth and ocean observation systems, methods of environmental monitoring and sustainable development;

- forecasting methods and evaluation instruments for sustainable development;

Strengthening the position of the EU's environmental technologies on the international markets will contribute to a sustained consumption and production, bringing sustainable growth through business opportunities and high competitiveness, while protecting the cultural and natural heritage.

### *3.7 TRANSPORT*

Transport is one of the great advantages that Europe has –the air transport sector contributes with 2.6 % at the UE's PIB, with 3.1 million jobs while the surface transportation sector produces 11% of the UE's PIB having 16 million employees. But transport is also responsible for 25% of the total amount of $CO_2$ emitted in the EU.

During PC7, at least 4 billion € will be allocated to finance EU's research to develop safer, greener and „smart" air-transport systems, for the benefit of all citizens. Research in transport will have a direct impact on other important areas, such as: trade, competition, employment, environment, energy, security and intern market.

It's imperative to efficiently approach the various political, technologic, socio-economical challenges on issues such as: the "safe and clean vehicle" of the future, interoperability and intermodality, particularly on rail and naval transport. Also, the development of support technologies for the Galileo system and its applications will be crucial in implementing European policies.

Activities approched during PC7 are:

- aeronautics and air transport –reduction of greenhouse emissions, engine and alternative fuel research, air traffic management, air traffic security, environmentally efficient aviation;

- sustainable surface transport –railways, roads, naval transport (development of clean and efficient engines and power-trains, reducing the impact of transport on climate change, intermodal regional and national transport, clean and safe vehicles, infrastructure construction and maintenance, integrative architecture)
- support for the European global satellite navigation system –GALILEO and EGNOS (navigation and synchronization systems, efficient use of satellite navigation)

Investments in transport research are necessary for the European transport industries to have a technologic advantage allowing them to become globally competitive. In addition, transport research in PC7 will produce, within the IMMs, a greater scale of innovation with better access to pan-European research programs and related benefits.

### *3.8 SOCIO-ECONOMIC SCIENCES AND HUMANITIES*

The long research tradition in this field as well as the different social, economic and cultural approach offer a unique opportunity to conduct this type of research in the EU.
During PC7, EU research in socio-economical sciences and in Humanities promise to study and offer answers to questions regarding demographic change and the quality of life, education and employment possibilities in the current economic trends, global interdependence and knowledge transfer, the wellbeing of democracies and political participation, cultural diversity and value.
The examined topics represent a priority at European level and are directed by community policies. In fact, research in the EU has specific benefits because it can develop information that is necessary for the enhancement of the knowledge level regarding complex issues.
The research subjects that will be analyzed within PC7 will be selected from the following areas:

- growth, employment and competitiveness in knowledge-society (innovation strategies, market competitiveness and employment, education and training throughout life, economic and productive structures);
- a combination of economic, social and environmental objectives from a European perspective (socio-economic models within Europe and the whole world, economic and social cohesion between regions, the social and economic dimensions of work strategies);
- major trends in society and their implications (demographic change, reconciliation between family and work, health and quality of life, youth policies, social exclusion and discrimination);
- Europe in the world (trade, migration, poverty, crime rate, conflict resolution);
- the citizen in the European Union (political participation, citizenship and rights, democracy and responsibility, media, cultural diversity and heritage, religions, attitudes and values);
- socio-economic and scientific indicators (the use and value of indicators in policy making at micro and macro level);
- foresight activities (the future implications of the global level of knowledge, migration, aging, risk and emerging areas of research and science);

Under PC7, the industry and the IMMs will be actively encouraged to participate in all fields, particularly in those conducted by the Cooperation program. The topics analized by the socio-economic sciences and by the Humanities offer them the opportunity to work, on one hand, as participants in the creation of information within teams and, on the other hand, as receptors of information, by putting it into application.

### *3.9 SPACE*

In the last 20 years, Europe has become a technology pioneer through applications such as Earth Observation and Galileo. Europe has invested in space exploration through cost-efficient missions and initiatives in collaboration with the European Spatial Agency, ensuring a strategic place in the field.

Research financed by EU will help develop the European Spatial Strategy. The European Spatial Strategy will sustain in return the strategies of the community in fields such as agriculture, environment, fish and fishing, transport, communications, either by instruments to observe space or by space-based solutions.

Science based on space research is an important force that leads to new technologic developments that have impact upon our daily life.

Research activities during PC7 will be selected from the following areas:

- research from space in the service of the European society (by developing satellite observance systems and GMES services for the administration of the environment, security, agriculture, forestry and meteorology, civil protection and risk management);
- space exploration (finding support for initiatives coming from collaborations between ESA and national space agencies, coordinated efforts to develop spatial telescopes);
- R&D of space technology for a stronger spatial infrastructure (research to support long term needs like spatial transportation, biomedicine, life sciences and physical sciences in space);

Space is a growing strategic sector and its applications support the economic activity and the governmental services. European companies, including IMMs as a majority, are the main providers of satellite manufacturing, launching systems, satellite operations and services on the global commercial market. A competitive industry requires new research and new technologies and the support offered through PC7 promises to create these opportunities.

### *3.10 SECURITY*

European security is a precondition for prosperity and freedom. The necessity for a comprehensive security strategy that encompasses both civil security and defense measures must be taken into consideration.

During PC7, the research financed by the EU will approach subjects regarding civil security (antiterrorism and crisis management) and will contribute to a range of communitarian strategies, such as transport, mobility, civil protection, energy, environment, health. By collaborating and coordinating the efforts on a Europe-wide scale, EU can better understand and respond to risks in a changing world. Security-related research is expected to generate new knowledge and promote the application of new technologies in the field of civil security.

Security within PC7 approaches the following areas:

- security of citizens (technologic solutions for civil protection, biosecurity, protection against crime and terrorism);
- security of infrastructures and utilities (examine and ensure infrastructures in areas such as ICT, transport, energy, financial and administrative services);
- intelligent surveillance and border security (technologies, tools, instruments and methods for protecting Europe’s inland and costal border control);
- security restoration in case of crisis (technologies and communication, coordination on the behalf of civil acts, humanitarian and rescue tasks);
- security systems integration, interconnectivity and interoperability (information gathering for civil security, privacy protection and transaction traceability);
- security and society (acceptance of security solutions; socio-economic, political, and ethical aspects of security, ethics and values, social environment and perceptions of security);
- security research structure and coordination (coordination between the European and the international security research efforts);

Security research will reinforce the competitiveness of the European security industry, stimulating the cooperation of providers and users of solutions for civil security. It will also attract the best intellectual and technologic values in Europe, by the active implication of the IMMs.

### *3.11 IDEAS*

The „Ideas" program, implemented by the European Research Council (ERC), will boost Europe's competitive level by attracting and keeping the most talented scientists, supporting risk-taking and high-impact research and by promoting world-level scientific expertise in new fields.
Countries with leading research are best positioned to provide better life quality for their citizens. While maintaining their economic position and their advance in the global competition they continue to attract scientists.
During PC7, the „Ideas" program will finance EU's frontier research. The concept behind Ideas is for first-class researchers to be put to identify new opportunities and directions at the frontiers of knowledge. They will gather feedback from society, will find their way towards industries and markets and will have greater social implications in the future.
There are two types of ERC scholarships, both operating upwards, without predetermined priorities, in all the fields of research.

- ERC scholarships for independent beginner researchers (ERC scholarships for beginners). The objective is to provide support for the independent careers of outstanding researchers. They are either located or moving to countries from or associated with the EU and are in progress of establishing their first research team or research program, regardless of nationality.
- ERC scholarships for advanced researchers. The objective is to support excellent frontier-research projects with researchers from the EU and the associated countries, regardless of nationality.

Frontier research is a key driver of wellbeing and social progress because it offers new opportunities for scientific and technnologic advancement. It is also important for the production of new knowledge, leading future applications and markets.
Projects will be financed on the basis of proposals made by researchers, both from the private and the public sector, on topics of their choice, and will be evaluated on the sole criterium of excellence.

### *3.12 PEOPLE. TRAINING AND DEVELOPING THE CAREER OF RESEARCHERS*

Highly trained and qualified researchers are needed in order to improve the welfare of citizens and foster economic growth.
To be competitive on a global scale many people working in research are needed. The foundation for an open labor market has to be provided for them. Therefore, Europe must be transformed into an attractive continent that supports innovation, knowledge, the creation of knowledge which will encourage scientists to stay.
During PC7, a series EU-financed actions will support sustained training, research and mobility for high-qualified scientists within Europe and worldwide. The proliferation of excellence centers in EU is encouraged as well as their contribution to new research and technology areas.
Based on the successful actions of Marie Curie, the "People" program will improve the human potential in European research and development, covering all the stages of a researcher's professional life, from initial training to life-time study and career development.
During PC7, the following actions are planned:

- Initial training of researchers through Marie Curie networks will improve their research skills and will help them join already-established research groups. In parallel, complemetary training will enhance their career perspectives in the public and private sectors.
- Life-long training and career development through individual scholarships and co-financing from international, national and regional level. All these provide the opportunity to acquire new skills, enhance mobility and reintegrate in research as experienced researchers.

- International scholarships in order to increase research talents outside Europe and host mutual-benefit research collaborations with scientists from outside Europe. The activity will also include measures to stop „braindrain” and create networks of European researchers that work abroad.
- Specific activities of forming an authetic european job market for researchers: removing mobility obstacles and enhancing their career posibilities. Public institutions will be offered stimulents to promote the mobility, quality and the profile of their reserchers as well awards in order to increase public awareness of Marie Curie actions and objectives.

Industry and SME participation is envisaged for all Marie Curie actions. The implication of the industry will be strongly supported through actions directed towards the initial training of researchers.

In parallel, another action will aim to build long-term collaborations between academies, industries and SMEs. The objective is to stimulate mobility between sectors and to enhance knowledge-sharing through mixed research partnerships. Recruitment of experienced researches in this kind of partnerships will be consolidated through temporary transfer of personnel between sectors and event organization.

### *3.13 RESEARCH INFRASTRUCTURES*

Research infrastructures play an increasingly important role in knowledge, technology and their exploitation. Thousands of scientists and university students, research institutes and industries from Europe and abroad benefit from research infrastructures such as: radiation sources, databases, observatories for environmental sciences, image systems, clean rooms for the creation of new materials or nanoelectronics, electronic infrastructures of information and communication, telescopes. These facilities, resources and services have the power to bring together people and investments and contribute to national, regional and European economic development. That is why they are important for research, education and innovation.

The industry uses the facilities for research infrastructures in collaboration with researchers. Construction and maintenance create important supply an demand effects. Such innovation capacities can be seen in the mobility of researchers from the public sector to the private one, in the new technologies applied in the creation of world-wide research installations, in derived products and/or in the new companies. Without a doubt, research infrastructures stimulate industrial impacts and play an important role in the creation of interfaces between science and industry.

Research infrastructures also have socio-economic impacts, for example, where there are pan-European research infrastructures, there often are “technologic nodes” called technologic parks as well. Such strategic centers of knowledge transfer can provide both the possibility for better contracts in interdisciplinary research and greater attraction of high-tech firms. As a result, different regions often compete to attract new installations, and this may represent an opportunity to increase public-private interaction in financing research activities.

### *3.14 REGIONS OF KNOWLEDGE*

Regions are being increasingly recognized as important factors in research and development in EU. Local resources play an active role in the scientific effort and innovation in favor of the society.

Actions undertaken in this field will enable European regions to strengthen their capacity to investigate and conduct research activities. This can be beneficial for regions from a local point of view but it can also be a way to maximize their potential for successful involvement in European research programs.

Region-level research policy and activities often rely on the development of “groups” uniting public and private factors. The Pilot actions of the program “Regions of knowledge” have demonstrated the dynamic of this evolution and the necessity to support and encourage such regional structures. Encouraging transnational networks between regions and groups that have research as the main focus will help maximize the regional potential, creating a dynamic environment that can attract and retain the best scientists. These groups will bring together universities, research centers, companies or regional

authorities, councils and development agencies.
Industry, in general, and SME's, in particular, are essential partners in EU's successful research programs. By assisting regions in increasing their investment, research and development capacity, the competitiveness and the knowledge absorption capacity will be consolidated, as well as the correlation with the regional policy of the Community in major national and regional programs, particularly with regard to convergence and EU's remote regions.
The activity of the Regions of knowledge will encourage regional and cross-border cooperation in research, regardless whether the concerned regions are under the convergence or the regional competitiveness objective.

### *3.15 THE RESEARCH POTENTIAL OF THE CONVERGENCE ZONES*

Europe needs to exploit its research potential, especially in less advanced areas, located far from the European center for research and industrial development. An inclusion strategy could bring benefits for the social factor, for the research community and for the industry, both locally and at the European Area of Research.
Taking advantage of the knowledge and experience existing in other parts of Europe, this action seeks to upgrade the research potential where necessary, providing support in the form of investments, personnel, networks or advice. This effort is directed towards scientists and institutions from the public and private sector.
The research comunity from convergence and remote areas will be supported through:

- exchange of research staff at transnational level, in both ways, between organizations selected from the convergence areas and one or more partner organizations; support for selected existing or emerging excellence centers, the recruitment of experienced scientists from other european contries;
- acquisition and development of research equipements; the creation of a material environment ofering the possibility to exploit the intellectual potential in existing or emerging excellence centers selected from the convergence areas;
- organization of seminars and conferences to facilitate knowledge transfer; activities of promotion and initiatives aimed to disseminate and transfer the results of research in other contries and on other international markets;
- "Evaluation facilities" through which any research center from convergence areas can obtain an international independent expert evaluation of the general quality level of research and research infrastructures;

To fully achieve a European Research Area in the enlarged Union, all regions must participate and be supported if necessary. This strategy brings direct benefits to SMEs and to the industrial organizations from the convergence zones.
Actions under this heading will indentify needs and opportunities for strengthening the research capacity of existing and emerging convergence centers from convergence areas, that can be fulfilled by structural and cohesion funds. The promotion of regional trade, research and development, in collaboration with the industry, is also necessary.

### *3.16 SCIENCE IN SOCIETY*

"Science in society" aims to bridge the gap betwen science professionals and those with no scientific studies and to promote the idea of scientific culture for the large public. Therefore, some of the initiatives aim to attract young people towards science and to consolidate scientific study at all levels.
While science and technology have an increasing influence on our daily life, they may seem to be absent from the daily concers of a large part of the public and politicians. The polemic aspects regarding emerging technologies should be addressed by the society on the basis of a well informed debate that will lead to decisions and clear choices. Therefore, another crucial factor is to encourage social dialogue related to research policy; stimulation of civil organizations to get them to become more involved in

research; to debate and promote shared values, equal opportunities and social dialogue.
The initiative undertaken during "Science in society" will provide support for issues such as the strengthening and improvement of the European scientific system. This includes "self-regulation" and the development of a policy on the role of universities in research development and their engagement in the challenges of globalization.
Special attention will be given to the communication between the scientific world and politicians, mass media and the public, and this will partially be achieved through supporting collaboration between scientists and mass media professionals.
Further efforts will be made in order for research to be done in an ethical manner and in accordance with the fundamental rights. Initiatives for a better governance of the European research and innovation system will be taken.
By stimulating young people to pursue scientific studies, industry personnel needs can be better supported on a long term. Women's progress in scientific careers will be promoted together with a greater acknowledgement of their professional and scientific talents.
The ethical frameworks for research activities together with a culture open to debate in terms of research and its place in society will be reinforced in order to enhance the credibility of citizens in the activities of industrial research.

### *3.17 INTERNATIONAL COOPERATION*
More than 100 countries from around the world are involved in EU's Research Programs. These activities will continue in PC7's „Cooperation" Program which covers international cooperation actions in 10 thematic and trans-thematic domains. They will be implemented in coordination with the „Cooperation, People and Capacities" Programs of PC7.
International request and development will contribute to the production of global public goods and will help reduce differences between the countries of the world. There already is a considerable amount of scientific knowledge improving the life standard of the people living in developing countries and that of Europeans. Where possible, the Framework Program will also help to reach the development objectives.
The participation of researchers and research institutions from outside the EU in partnerships or cooperation must respect the confidentiality restrictions in the thematic sectors.
These actions are associated either with the bilateral cooperation or with multilateral dialogues between the EU and these countries and will serve as privileged instruments for the application of the cooperation between the EU and them. They can be:

- actions to consolidate the research capacity of candidate and neighboring countries;
- cooperation activities for developing and accessioning countries emphasizing their particular needs in different sectors, like: health, agriculture, fishing, environment, and their application in financial conditions adapted to their capacity;

International cooperation under PC7 will further integrate into the international community and will help research and technology to advance in those countries that are building their own knowledge capacity. They will, on one hand, enrich the European research with knowledge generated in the world; on the other hand, they will increase technology and science knowledge through the competence of the societies and companies from the developing countries.

### *3.18 EURATOM*

The framework program for nuclear research and instruction activities will comprise Communitarian research, technologic development, international cooperation, dissemination of technical information as well as training activities.
Two specific programs are planned:

- fusion energy research as a reliable and sustainable energy source, reliable in terms of environment and viable from an economic point of view; Activities will include the completion of an International Experimental Reactor (RTEI=ITER) (as an international research infrastructure), research and development of RTEI's functionality, technologic activities to prepare the Nuclear Fusion Demonstration Reactor (DEMO), preparing an International Iradiation Center for Fusion Materials (IFMIF-CIIMF);

- the second program will cover the activities of the Joint Research Center (JRC) in nuclear energy, including radioactive waste management and environmental impact, nuclear safety and nuclear security[2].

## 4. CONCLUSIONS

It is imperative for everybody to acknowledge the fact that scientific research is not a commercial activity. Many official documents give the impression that the main justification for investments in scientific research would be the need for new products: as productivity increases, fewer and fewer employees can do the same job, so how can you offer jobs to the others if not by creating new industries?

On one hand, the association is correct: there can't be new products and industries without research.

On the other hand, we are dealing with a serious confusion: the purpose and utility of the research system is not the production of new marketable products, but the creation and improvement of scientific methods of prediction of reality. Some side effects of the scientific progress are indeed economic, but out of them only a small fraction is represented by the creation of new industries.

The misunderstanding of the social utility of research, through the commercial side effect, gives rise to harmful strategy errors on long term.

European civilization is largely defined by the emergence of a form of organization called "learned society" or "scientific society". This is usually an association of certain persons that are interested about a domain. Within such an organization, many of the problems of communication are substantially reduced or disappear. First of all, the members share the same language and comparable knowledge in the field. Secondly, novices entering the society assign older members a certain authority and they communicate a lot with the other members. Among peers, sincerity gives more authority than keeping the old opinions at any price.

The models and verifications that convince the members of the society are generally put down in a journal (magazine) of their own. Registration (publication) in this journal implies a first acceptance of the plausibility of the work through a process called peer review. This process eliminates many of the errors of the new models proposed through discussions inside the society rather than trough public disputes, which would be more confusing and expensive.

For the public, the acceptance of a model by the peers in the society is a guiding indicator regarding its potential predictive value.

The size, funding and legal status of scientific societies are largely governed by the communication costs, communication (model announcement, announcement of research results, dispute development, peer review and publication) as their main reason for existence. The means of communication have changed with time: meetings, proceedings, letters, publications in journals, newsletters, books, observation reports, scientific anagrams, secret storage of documents, patents, emails, each corresponding to a specific literary genre. Archiving and communication infrastructure include commercial editions, libraries, organizations maintaining electronic archives and databases, physical or electronic mail, patent offices.

A restricted category of applications of scientific models are predictions on whether a device will function, or a substance, material or living creature will be obtained through a certain process and will have certain properties. Checking this kind of predictions is possible by constructing the device or by trying to synthesize the substance and test the qualities.

The main use of such devices, substances or beings for research purposes generally is that of making possible or accelerating the taking of new directions. This market nourishes the most innovative and exploratory segment of the industry. Most products do not reach the markets but almost all come from here.

Compared to this segment of the industry, fundamental research has an irreplaceable role in evaluating

[2] www.ec.europa.eu/PC7

whether a new product has a certain quality or utility, which would otherwise be impossible to objectively assess.

Some of the products sold on the laboratory research market are those proven to be useful after predictions (such as instruments and reactives), which eventually find their place on the of consultancy services market (meaning of prediction).

Sometimes, if there is a trader and an investor interested, large scale production can be tried for a product based on a new method, thus transforming a scientific progress into a commercial product and eventually into a new industry even if most of the products for which large scale production has been attempted have failed to reach a profitable level for various reasons.

This commercialization is often done when the solicitation for a new or improved product leaves from the trader (who sees a potential need on a market), but usually, even though they seem new to the public, we are dealing with operating principles known for years and already popularized in commercial environments, where one finally finds an outlet. Between the prototype and the successfully marketed product, numerous redesigning processes requested by the marketing needs usually intervene, as the trader discovers the real requirements of the market, so from the original prototype rarely does anything other than the principle remain.

The research corresponds to the need of the society to adapt, the research system constituting the perceptual branch of the adaptation mechanism. Adaptation involves changing society. Some changes can be economically described or can even be described in terms of industrial development: industries disappear (for example, carriage transport), new industries appear (for example, electronics industry), and other reform. On the short term, this Process seems more or less enjoyable for certain members of the society, but others oppose it and try to delay it, more or less successfully. But most changes can't be economically described, and even less industrially. For example it is hard to say what economic impact the theory of atmospheric warming will have. The energy industry will reform. For some, new opportunities will arise. Others will have to close their businesses, perhaps with a loss, perhaps through the intervention of governments.

Only the defining facets of the research system are accessible to economic analysis. For example, it is obvious that the financing for the research system takes three specific destinations: the salaries of the employees in the system, communication costs (including travels and documentation), investment costs and consumables. Wages get spent on goods and services. Communication costs are actually the acquisitions of the editorial, tourism and communication industries.

The immediate impact of research investments on industrial development is indirect, through purchases of equipment, installations, services and specific consumables. Funds given for this purpose to research institutes end up to the producers of these devices, installations, software and new consumables. Without this research-laboratory market most of the new products would not pass from the prototype stage to the large- market-ready product stage: the healthy objective for an industry is to satisfy a demand, a new industry presupposes a demand for new products and the ability of costumers to evaluate them, and this new kind of demand is normally created by the research system.

The analysis of these interferences with the economy shouldn't however be extended over the entire system of research, whose main objectives and activity are of a different nature.

The wrong perception of the scientific system as being aimed to attract financing, economic impact, or social and financial involvement in such a concept can be harmful.

## 5. References


[1] Corlan, A. D., Cercetarea ştiinţifică nu este o activitate comercială, 2005, www.ad-astra.ro/journal

[2] Florescu, M. S., *Managementul cercetării ştiinţifice*, Bucharest, ASE, 2006

[3] Gheorghe, I. Gh., *Metodologia cercetării ştiinţifice, dezvoltării şi inovării*, Bucharest, Cefin, 2008

[4] Krausz, S., *Metodologia cercetării ştiinţifice*, Petroşani, Universitas, 2007

[5] Plumb, I., Vişan, S., Botez, L. F., Florescu, M. S., Angelescu, A., *Managementul cercetării şi*

*inovării*, Bucharest, Second Edition, ASE, , 2007
[6] Răboacă, Gh.,Comşa, M., Ciucur, D., *Metodologia cercetării ştiinţifice economice*, Bucharest, Editura Fundaţiei România de Mâine, 2007
[7] Sandu, S., *Inovare, competenţă tehnologică şi creştere economică*, Bucharest, Expert, 2002
[8] Institutul Naţional de Statistică, (2009), *Cercetare-dezvoltare în România, Colecţie de date statistice (2003-2008)*
[9] Hotărârea Guvernului nr. 475/2007, publicată în Monitorul Oficial nr. 371 din data de 31.05.2007 privind Planul Naţional de Cercetare-Dezvoltare şi Inovare II pentru perioada 2007-2013
[10] www.ancs.ro/Programe interne
[11] www.ec.europa.eu/PC7