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SUMAR

BULETINUL INSTITUTULUI POLITEHNIC DIN IAŞI
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CONSTRUCTIONS. ARCHITECTURE

C O N T E N T S

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Secţia
CONSTRUCŢII. ĂRHITECTURĂ

THE TORSIONAL ECCENTRICITY OF THE STRUCTURES IN MISCELLANEOUS DESIGN CODES

BY

MIHAELA ANECHITEI, DOINA ŞTEFAN and IOANA OLTEANU

Abstract. Structures rarely achieve structural symmetry. Even in symmetric structures, the asymmetric position of the structural components tends to produce an effective asymmetric structure. Such an asymmetry even if is small, can produce a torsional response coupled with translational response. In the torsional analysis, the provisions of the seismic norms are based on determining the static eccentricity. In this paper is presented a general way of computing the eccentricity in torsional design. The purpose of this article is to make a comparison of the general rules used to determine the eccentricity and their values. Are taken into account the norms from: Romania, Europe, Japan, Germany and the United States of America.

Key words: eccentricity; torsion.

1. Introduction

The reflection of the torsion effect in the design codes provisions, which often have critical accents and improvement suggestions, was the subject of several studies. The norms establish principles of compliance, computation methods and constructive rules for designing safe buildings considering the seismic hazard associated with the site. Any new structure designed and built in a seismic area must fulfill certain requirements which are correlated with the medium recurrence period of the seismic action. In order to prevent the damages and the collapse of the buildings each seismic design code defines a set of medium recurrence periods and a corresponding set of requirements. In case of damages the structure is considered in the elastic domain, while in case of collapse, postelastic strains are permitted.

The design processes for the torsion effects must consider several factors that are difficult to predict and assess [1]. Most of the current structural design provisions require to consider the torsional behavior using the design eccentricities, which take into account both natural and accidental sources of torsion. The natural eccentricity is generally defined as the distance between the center of mass (CM) and the center of rigidity (CR) for a considered floor, while accidental eccentricity generally accounts for factors such as the rotational component of ground motion about the vertical axis, the difference between computed and actual values of the mass and rigidity and an unfavourable distribution of live load mass.

These factors lead to the appearance of some accidental eccentricities and are not taken into account in the seismic analysis with no coupled torsion, determine that the seismic response in a coupled torsion structure to be amplified compared with the response obtained using the torsion moment.

2. Eccentricity Provisions in Different Codes

In Romanian code P100-2006 [2], in case of structures with rigid floors in their own plan, an additional accidental eccentricity is introduced through the effects generated by the uncertainties associate with the distribution of the mass level and/or the spatial variation of the ground seismic movement [2]. This is considered for each design direction and for each level and also is related to the center of mass. The accidental eccentricity is computed with the relationship

$$e_{1i} = \pm 0.05 L_i, \tag{1}$$

where: e_{1i} is the accidental eccentricity of mass for storey i from its nominal location, applied in the same direction at all levels; L_i – the floor dimension perpendicular to the direction of the seismic action.

In Eurocode 8 [3], if the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity is taken into account by a more exact method (modal analysis with response spectrum), the accidental torsional effects may be accounted by multiplying the loads effects for individual resisting elements with the δ factor given by

$$\delta = 1 + 0.6\frac{x}{L_e}, \tag{2}$$

where: x is the distance from the considered element to the center of mass of the building in plan, measured perpendicularly to the considered direction of the seismic action; L_e – the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.

In Eurocode 8, Annex A, is presented an alternative method for the one presented above, an approximately method for the torsion effects. Two planar models, each one for every direction, are used. Torsion effects for each of these two directions are determined separately. To take into account the dynamic effects of simultaneous vibration of translation and torsion and to include the uncertainties associated to the mass level distribution and spatial variation of the ground seismic movement, beside accidental eccentricity, the CM will be displaced from the initial position with a real eccentricity and an additional eccentricity.

The additional eccentricity is equal with the minimum values of the followings two relationships:

$$e_2 = 0.1(L+B)\sqrt{10\frac{e_0}{L}} \le 0.1(L+B), \tag{3}$$

and

$$e_2 = \frac{1}{2e_0}\left[l_s^2 - e_0^2 - r^2\sqrt{\left(l_s^2 + e_0^2 - r^2\right) + 4e_0^2 r^2}\right], \tag{4}$$

where: e_2 is the additional eccentricity; e_0 – the real eccentricity, calculated as the distance between the CM and the CR; r^2 – the ratio between the torsional rigidity and the lateral rigidity (square of torsional radius); l_s^2 – the square of the gyration radius,

$$l_s^2 = \frac{L^2 + B^2}{12}. \tag{5}$$

The torsion effects can be determined as the envelope effects resulting from the analysis of two loading situations with torsion moment, situations described in Fig. 1; where e_1 is the accidental eccentricity previously defined.

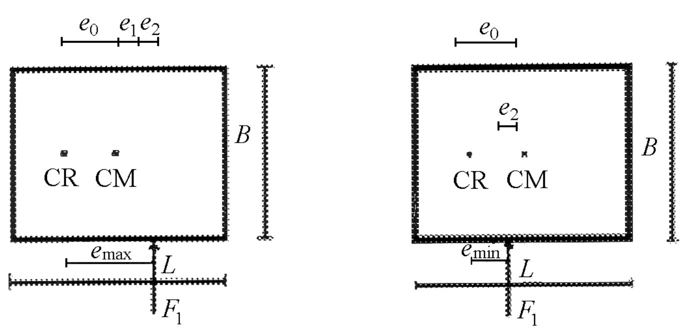

Fig. 1 – The establishing of the application of seismic force.

Some design codes require the torsion effects consideration by applying torsional moments on each level (multiplying the level seismic loads with a design eccentricity).

In the most analysed norms, the design eccentricity is obtained by superimposing the static eccentricity (defined as the distance between the CR and the CM), possibly multiplied by a coefficient (amplification coefficient for elements on the flexible side and a reduction coefficient for elements on the rigid side) and an accidental eccentricity, computed as a percentage of the building's dimension perpendicular to the earthquake direction [5]

$$e_d = \alpha e_s + \beta L_i, \tag{6}$$

$$e_d = \delta e_s - \beta L_i, \tag{7}$$

where: e_s is the real eccentricity; L_i – the floor dimension; α, β, δ – the coefficients specified in various design codes.

The values of the coefficients α, β, δ varies in codes like Uniform Building Code (UBC), Mexico City Building (MCBC), National Building Code of Canada (NBCC); these values are presented in Table 1.

Table 1

The Values of the Coefficients α, β, δ in Norms

Code	α	β	δ
UBC	1	1	0.05
MCBC	1.5	1	0.1
NBCC	1.5	0.5	0.1

For each element, the e_d value is used leading to a maximum computing force. The first term of these relationships is called the *dynamic eccentricity* and takes into account the coupled lateral – torsional response obtained from the symmetry absence. The second term considers the torsion effects due to factors which are not explicitly specified (such: the rotational component of the ground motion about the vertical axis, differences between calculated and actual values of the stiffness, unfavourable distribution of loads).

The specific eccentricities use different relationships in norms; they are presented in Table 2.

Some codes, like UBC [4], stipulate lateral force application at the $\pm\beta B_i$ distance from the CM, which would lead to $\alpha = \delta$. In this case the position of the CR on the various levels of the structure is not necessary to be determined. The codes that consider in the eccentricity relationship α or $\delta \neq 1$, as MCBC, the position of the center of rigidity seems to be necessary to be determined. Unlike buildings with a single level, buildings with more levels have problems with the level position of the CR, unless these belong to the special classes of the proportional buildings.

Table 2
The Eccentricities

Design codes	Primary design eccentricity	Secondary design eccentricity
USA	$e_0 + 0.05L_i$	$e_0 - 0.05L_i$
Bulgaria	$e_0 + 0.02L_i$	$e_0 - 0.02L_i$
Canada	$1.5e_0 + 0.1L_i$	$0.05e_0 - 0.1L_i$
Europe	$e_0 + 0.05L_i$	$e_0 - 0.05L_i$
Greece	$e_0 + 0.05L_i$	$e_0 - 0.05L_i$
Mexico	$1.5e_0 + 0.1L_i$	$e_0 - 0.1L_i$
Romania	$e_0 + 0.05L_i$	$e_0 - 0.05L_i$

In the German norm provisions [5], the eccentricity, e, of the application point of the resultant of the horizontal seismic loads related to the center of rigidity have the following relationships:

$$e_{\max} = e_0 + e_1 + e_2 , \qquad e_{\min} = e_0 - e_1 , \tag{8}$$

where: e_0 is the distance between the CR and the CM; e_1 – the additional eccentricity which takes into account the coupling of the translation vibrations of buildings with the torsion vibrations,

$$e_1 = 0.1(A + B)\sqrt{\frac{10e_0}{B}}, \tag{9}$$

with the following condition:

$$e_1 \le 0.1(A + B). \tag{10}$$

If this condition is not fulfilled, the eccentricity, e_1, is computed with the relationship

$$e_1 = \frac{1}{2e_0}\left[i^2 - e_0^2 - d^2 + \sqrt{\left(i^2 + e_0^2 - d^2\right) + 4e_0^2 d^2}\right], \tag{11}$$

where

$$i^2 = \frac{A^2 + B^2}{12}, \tag{12}$$

$$d^2 = \frac{K_t}{K_y}; \tag{13}$$

K_t is the torsion rigidity of the considered level; K_y – the translation rigidity of the considered level.

For rigid structures with vertical structural walls the square distance is computed with the relationship

$$d^2 = \frac{\sum_m I_m d_m + \sum_p I_p d_p}{\sum_m I_m}, \tag{14}$$

where: I_m is the inertia moment of the structural walls, m, parallel to the direction of the seismic action; I_p – the inertia moment of the structural walls p, perpendicular on the seismic action; d_m, d_p – the distances from the centers of gravity of the structural walls m and p to the considered center of rigidity; $e_2 = 0.05L_i$ – the accidental eccentricity that takes into account the inaccuracies of the building erection, some errors in the evaluation of the distance e_0 and asynchronous nature of the seismic movement at the level of the structure foundation which leads to torsional oscillations.

For $d^2 > 5(e_0^2 + i^2)$ the structure is stiffened for torsion and in this situation the eccentricity e_1 can be neglected.

For structures with symmetrical disposal of vertical structural elements $e_0 = e_1 = 0$.

In the Japanese recommendations [6] is provided that the stiffness eccentricity of each level, R_e, have to satisfy the condition

$$R_e = \frac{e}{r_e} < 0.15, \tag{15}$$

where: e is the distance between the CM and the CR; r_e – the elastic range, defined as the ratio of the square root of the torsional rigidity and lateral stiffness. The lateral rigidity, R_s, varies on each level, fulfilling the condition

$$R_s = \frac{r}{\overline{r}} > 0.6, \tag{16}$$

where: r is the lateral rigidity, defined as the ratio between the height of the floor and its displacement produced by the lateral seismic force, for moderate seismic movements; $\overline{r}$ – the medium lateral rigidity defined as the average of the lateral rigidities of the levels above the ground floor level.

The Mexican design code, MCBC-95 [7], requires the consideration of the torsion effects at each level through a torque level equal to the shear force multiplied by the most unfavourable of the following eccentricities:

$$1.5\, e_s + 0.1b \quad \text{or} \quad e_s - 0.1b. \tag{17}$$

The computation eccentricity must be at least equal with the maximum value of e_s of the lower floors.

3. Conclusions

Eurocode 8 uses a method of additional eccentricity in defining the computed primary eccentricity that controls the computation of the strength of the elements on the flexible side. All other norms realize a similar effect amplifying the structural eccentricity with a coefficient effect in order to take into account the dynamic effects of the torsion.

Unlike other norms, the value of the additional eccentricity presented in EC8 depends on the configuration of the building in plan, and on the ratio of the total torsion rigidity over the CR compared to its total lateral rigidity. In addition, all this norms base the strength amplification from the flexible side of the element on the structural eccentricity.

Unfavourable global behavior of buildings to strong earthquakes can be substantially influenced by improper arrangement of the masses on the height of the building and in plan. Thus, the poorly functions of the establishment in a building plan may lead to pronounced lop-sided location of the masses in the construction plan and therefore to substantial increase of the distance between the CM and the CR.

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EXCENTRICITATEA LA TORSIUNE A STRUCTURILOR ÎN DIVERSE NORME INTERNAȚIONALE

(Rezumat)

Aşa cum sunt construite, structurile ating rareori o simetrie structurală. Chiar în structurile simetrice dispoziția asimetrică a componentelor nestructurale tinde să producă o structură efectiv asimetrică. O astfel de asimetrie, chiar dacă este redusă, poate produce un răspuns torsional cuplat cu răspunsul de translație. Aplicarea prevederilor normativelor de proiectare antiseismică în analiza torsională se bazează pe determinarea excentricității statice.

Se prezintă modul general de determinare al excentricității privind calculul la torsiune. Articolul îşi propune să realizeze o comparație a normelor la nivel mondial cu privire la valorile şi la modul în care este determinată excentricitatea. Se iau în considerare normele din: România, Europa, Japonia, Germania şi Statele Unite.

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THE PRESERVATION OF ADOBE BUILDINGS

BY

DAN BABOR and DIANA PLIAN

Abstract. Whether built in the 17th or in the 20th centuries, adobe buildings share common problems of maintenance and deterioration. These types of buildings represent today in Romania a traditional house for country people. It also makes recommendations for preserving adobe buildings. By its composition, adobe construction is inclined to deteriorate; however, the buildings can be made durable and renewable when properly maintained.

Key words: clay; adobe; dried brick.

1. Introduction

Traditional adobe construction techniques in Romania have not varied widely for over 3 1/2 centuries (Fig. 2). Adobe building methods employed in the 16th century are still used today (Fig 1). Because adobe bricks are not fired

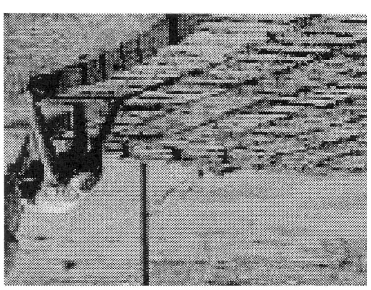

Fig. 1 – Sun-dried brick molding.

Fig. 2 – Adobe house.

in a kiln as are clay bricks, they do not permanently harden, but remain unstable they shrink and swell constantly with their changing water content. Their strength also fluctuates with their water content: the higher the water content, the lower the strength.

A mixture of mud and straw is pressed into a mold to form an adobe brick. After the adobe brick is removed from the mold, it must dry in the open air for a month or more before it can be used.

Adobe will not permanently bond with metal, wood, or stone because it exhibits much greater movement than these other materials, either separating, cracking, or twisting where they interface (Fig. 3). Yet, many of these more stable building materials such as fired brick, wood, and lime and cement mortars are nonetheless used in adobe construction.

Fig. 3 – Adobe cond with wood.

For example, stone may be used for a building's foundation, and wood may be used for its roof or its lintels and doorways. In the adobe building, these materials are generally held in place by their own weight or by the compressive weight of the wall above them. Adobe construction possibilities and variations in design have therefore been somewhat limited by the physical constraints of the material.

Preserving and rehabilitating a deteriorated adobe building is most successful when the techniques and methods used for restoration and repairs are as similar as possible to the techniques used in the original construction.

2. Adobe Construction Techniques

2.1 The Brick

The adobe brick is molded from sand and clay mixed with water to a plastic consistency. Commonly, straw or grass is included as a binder. Although they do not help to reinforce the bricks or to give them added long-term strength, straw and grass do help the bricks shrink more uniformly while they dry. More important for durability, however, is the inherent clay-to-sand ratio found in native soil. The prepared mud is placed in wooden forms, tamped, and leveled by hand. The bricks are then "turned-out" of the mold to dry on a level surface covered with straw or grass so that the bricks will not stick. After several days of drying, the adobe bricks are ready for air-curing. This consists of standing the bricks on end for a period of four weeks or longer (Fig. 4).

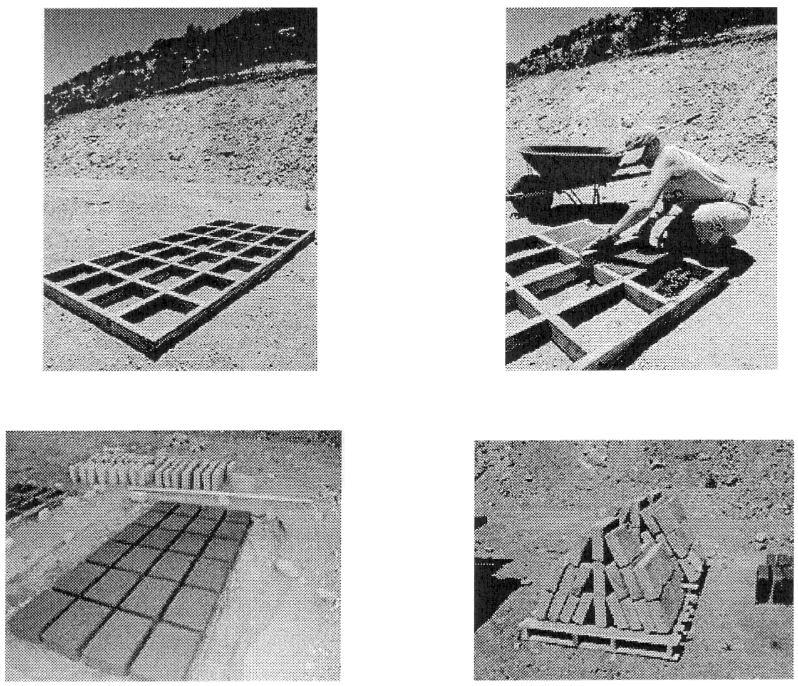

Fig. 4 – Adobe cycle.

2.2. Mortar

Historically, most adobe walls were composed of adobe bricks laid with mud mortar. Such mortar exhibited the same properties as the bricks: relatively weak and susceptible to the same rate of hygroscopic (moisture absorptive) swelling and shrinking, thermal expansion and contraction, and deterioration. Consequently, no other material has been as successful in bonding adobe bricks. Today, cement and lime mortars are commonly used with stabilized adobe bricks, but cement mortars are incompatible with unstabilized adobe because the two products have different thermal expansion and contraction rates. Cement mortars thereby accelerate the deterioration of adobe bricks since the mortars are stronger than the adobe.

2.3. Building Foundations

Early adobe building foundations varied because of the difference in local building practices and availability of materials. Many foundations were large and substantially constructed, but others were almost nonexistent. Most often, adobe building foundations were constructed of bricks, fieldstones (Fig. 5), or cavity walls (double) filled with rubble stone, tile fragments, or seashells. Adobe buildings were rarely constructed over basements or crawlspaces.

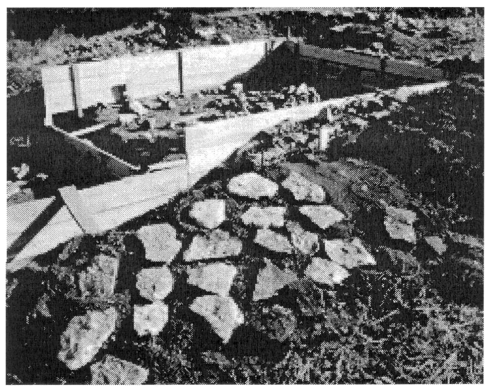

Fig. 5 – Stone foundation.

2.4. Walls

Since adobe construction was load-bearing with low structural strength, adobe walls tended to be massive. In some parts of the Romania, it was common to place a long wooden timber within the last courses of adobe bricks. This timber provided a long horizontal bearing plate for the roof thereby distributing the weight of the roof along the wall (Fig. 6).

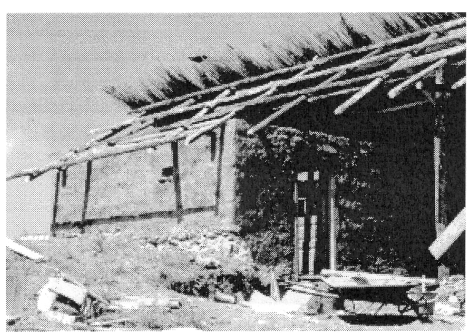

Fig. 6 – Adobe walls.

2.5. Roofs

Adobe roofs tended to be flat with low parapet walls. These roofs consisted of logs which supported wooden poles (Fig. 7), and which in turn supported wooden lathing or layers of twigs covered with packed adobe earth. The wood was fir, oak, lime, elm, or whatever was available.

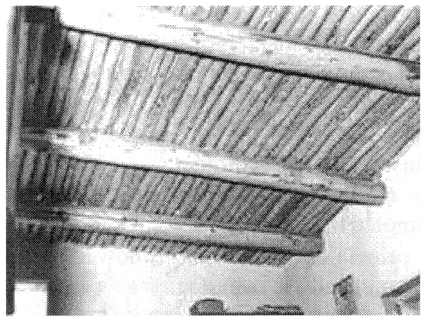

Fig. 7 – The wooden materials that comprise the traditional flat adobe roof create interesting and pleasing patterns on the ceilings of interior rooms.

2.6. Floors

Historically, flooring materials were placed directly on the ground with little or no subflooring preparation. Flooring materials in adobe buildings have varied from earth to adobe brick, fired brick, tile, or flagstone, to conventional wooden floors (Fig. 8).

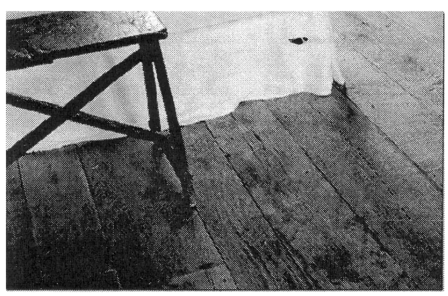

Fig. 8 – Ancient oak wood floor.

3. Adobe Deterioration

When preservation or rehabilitation is contemplated for an adobe building, it is generally because the walls or roof of the building have deteriorated in some fashion-walls may be cracked, eroded, pitted, bulging, or the roof may be sagging. When planning the stabilization and repair of an adobe building, it is necessary

a) to determine the nature of the deterioration;

b) to identify and correct the source of the problem causing the deterioration;

c) to develop rehabilitation and restoration plans that are sensitive to the integrity of the historic adobe building;

d) to develop a maintenance program once the rehabilitation or restoration is completed.

There are several principles that when followed generally it results a relatively stable and permanent adobe resource namely

1. Whenever possible, secure the services or advice of a professional architect or other preservationist proficient in adobe preservation and stabilization. Although this may be more costly than to "do-it-yourself," it will probably be less expensive in the long run. Working with a deteriorated adobe building is a complex and difficult process. Irreversible damage may be done by well-meaning but inexperienced "restorationists."

2. Never begin restoration or repairs until the problems that have been causing the deterioration of the adobe have been found, analysed, and solved. For instance, sagging or bulging walls may be the result of a problem called *rising damp* and/or excessive roof loads. Because adobe deterioration is almost always the end product of a combination of problems it takes a trained professional to analyse the deterioration, identify the source or sources of deterioration, and halt the deterioration before full restoration begins.

3. Repair or replace adobe building materials with the same types of materials used originally and use the same construction techniques. Usually the best and the safest procedure is to use traditional building materials. Repair or replace deteriorated adobe bricks with similar adobe bricks. Repair or replace rotted wooden lintels with similar wooden lintels. The problems created by introducing dissimilar replacement materials may cause problems far exceeding those which deteriorated the adobe in the first place.

4. Repairing and Maintaining the Adobe Building

Fig. 9 – A traditional mixture of mud and straw plaster should be applied to stabilize the exterior of this house.

Once the adobe deterioration and any resulting structural damage is repaired, the restoration of the adobe building can proceed. Careful attention should be given to replace, repair, and/or reproduce all damaged materials with traditional or original materials (Fig. 9).

4.1. Patching and Repairing Adobe Brick

In patching and replacing adobe brick every reasonable effort should be made to find clay with a texture and colour similar to the original fabric. When an individual adobe brick has partially disintegrated, it may be patched in place. The deteriorated material may be scraped out and replaced with appropriate adobe mud. Often fragments of the original adobe brick have been grounded up, mixed with water, and reused to patch the eroded area. However, some professionals advise against the reuse of material which has spalled off because it frequently contains a high concentration of salts.

If a substantial amount of the brick has been destroyed or spalled, commercially made adobe bricks and half-bricks can be obtained, or they may be made at the site or nearby. Generally they are composed of unstabilized adobe (that is, without any chemical additives). The deteriorated adobe bricks should be scraped out to insert the new bricks. If most of the brick is not deteriorated, then the deteriorated portion may be replaced with a half-brick. It may be necessary to cut back into undeteriorated portions of the brick to achieve a flush fit of the new or halfbricks. Spray (do not soak) the new brick and surrounding area lightly with water to facilitate a better bond. Too much moisture can cause swelling. Always use traditional adobe mud mortar.

When entire bricks or sections of the brick walls have to be replaced, caution should be taken when buying ready-made bricks. Many are now manufactured using stabilizing agents (Portland cement, lime, or emulsified asphalt) in their composition. While the inclusion of these agents in new adobe bricks is a technical advancement in their durability, they will prove incompatible with the fabric of the historic adobe building. Concrete blocks and cinderblocks are likewise tempting solutions to extensive adobe brick replacement; but, like commercially stabilized adobe bricks, they are not compatible with older and more unstable adobe bricks. However, concrete blocks have been used for interior partitions successfully.

4.2. Patching and Replacing Mortar

In repairing loose and deteriorated adobe mortar, care should also be taken to match the original material, colour, and texture. Most important, never replace adobe mud mortar with lime mortar or Portland cement mortar. It is a common error to assume that mortar hardness or strength is a measure of its suitability in adobe repair or reconstruction. Mortars composed of Portland cement or lime do not have the same thermal expansion rate as adobe brick. With the continual thermal expansion and contraction of adobe bricks, Portland cement or lime mortars will favour the bricks – the weaker material – to crack, crumble, and eventually disintegrate.

It is recognized, however, that some late historic adobe buildings have always had Portland cement or lime mortars in their initial construction. The removal and replacement of these mortars with mud mortar is not advised because their removal is usually destructive as regards the adobe bricks.

In repairing adobe cracks (Fig. 10), a procedure similar to repointing masonry joints may be used. It is necessary to rake out the cracks to a depth of 2 or 3 times the width of a mortar joint to obtain a good "key" (mechanical bond) of the mortar to the adobe bricks. The bricks should be sprayed lightly with water to increase the cohesive bond. A trowel or a large grout gun with new adobe mud mortar may then be used to fill the cracks.

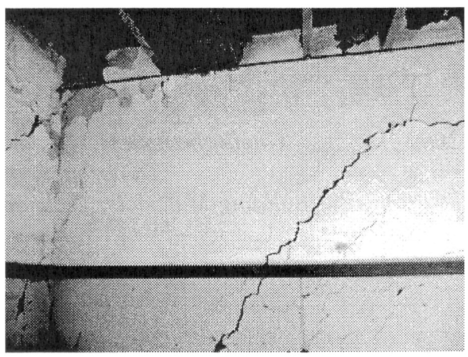

Fig. 10 – Adobe wall craks.

5. Maintenance

Cyclical maintenance has always been the key to successful adobe building survival. As soon as rehabilitation or restoration has been completed, some program of keeping maintenance should be initiated. Changes in the building should particularly be noted. The early stages of cracking, sagging, or bulging in adobe walls should be monitored regularly. All water damage should be noted and cured at its earliest possible stages. Plant, animal, and insect damage should be halted before it becomes substantial. The roof should be inspected periodically. Surface coatings must be inspected frequently and repaired or replaced as the need indicates.

Mechanical systems should be monitored for breakdown. For instance, leaking water pipes and condensation can be potentially more damaging to the adobe building than to a brick, stone, or frame structure. Observing adobe buildings for subtle changes and performing maintenance on a regular basis is a policy which cannot be over-emphasized. It is the nature of adobe buildings to deteriorate, but cyclical maintenance can produce a relatively stable adobe building.

6. Conclusions

In conclusion, to attempt the preservation of an adobe building is almost a contradiction. Adobe is a formed-earth material, a little stronger perhaps than the soil itself, but a material whose nature is to self deteriorate. The preservation of adobe buildings, then, is a broader and more complex problem than most people realize. The propensity of adobe to deteriorate is a natural, ongoing process.

While it would be desirable to stop that process in order to safeguard the building, no satisfactory method has yet been developed. Competent preservation and maintenance of adobe buildings in the Romania must: (i) accept the adobe material and its natural deterioration, (ii) understand the building as a system, and (iii) understand the forces of nature which seek to return the building to its original state.

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CONSERVAREA CLĂDIRILOR DIN ARGILĂ

(Rezumat)

Avînd în vedere fondul de construcţii din mediul rural din România, starea economică actuală a locuitorilor de la sate, construcţiile din argilă rămîn şi vor fi o alternativă viabilă mulţi ani înainte pentru aceşti locuitori. Astfel, conservarea şi întreţinerea acestor clădiri devine un deziderat imperios necesar pentru unii săteni.

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THE ROLE OF ESTIMATION ON CONSTRUCTION LIFE CYCLE

BY

EDUARD ANTOHIE

Abstract. Estimates serve a number of different functions in the construction industry. The reason for estimating is to provide the most realistic prediction possible of time and cost at any given stage in a project. Over the life of a project the estimator should be able to produce a series of estimates, from an early estimate at beginning, to the final account, in which the increasing accuracy of the estimate is reflected in the decreasing extent of risk and uncertainty. All estimates take the form of base estimates, plus allowances for uncertainties and specific contingencies as required. The role of the estimator or practitioner in cost engineering is vital to the success of a construction project. The estimators are responsible for predicting the most economic costs for construction in a way that is both clear and consistent. Although an estimator will have a feel for the prices in the marketplace, it is the responsibility of management to add an amount for general overheads, assess the risks and turn the estimate into tender.

Key words: cost estimate; estimator; cost engineering; cost construction life cycle; cost management.

1. Introduction

The estimated cost means an estimate of the final total cost of execution of a construction project. This definition requires two important issues, namely

a) the estimate is an approximate calculation;

b) estimate contains uncertainties.

Main purpose of estimating costs is to provide a size reference for cost control, to verify that the resources consumed during the execution of the project are kept in the costs assessed in feasibility phase of the project. Deviations from these issues can endanger the profitability of the project and a successful project can turn into a disaster. Accuracy of estimates of cost depends on existing information to reflect and their calculation. In Fig. 1 is shown a structure of total costs of a construction project in this regard.

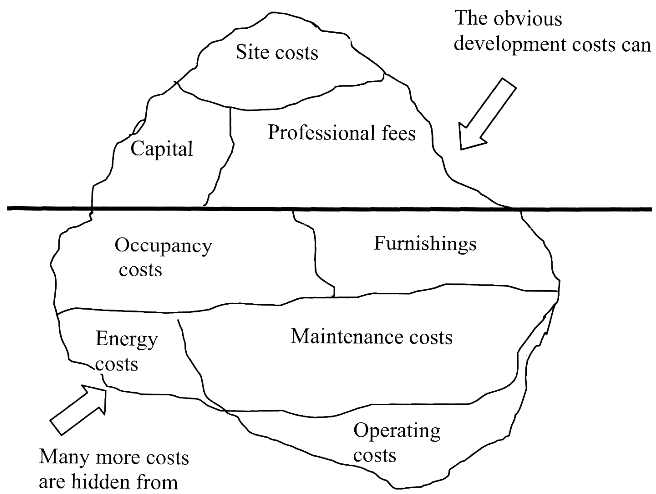

Fig. 1 – Components of life cycle costs.

There are several ways of classifying types of estimates of the costs of a construction project. Of these, the most significant are the followings:

a) *Estimates that depend on the degree of definition of the project*. Such estimates depend on the rate of breakdown of the architectural design and strength. Are defined as the level of information they have available the practitioner to perform engineering cost estimates.

b) *Estimates that depend on their use.* Such estimates are directly related to the utility for which they are made. Thus we have estimates used for the preparation of feasibility studies, estimates for preparing documentation authorizing the work, estimates for budget planning, estimates for the preparation of the contractors bid price, estimates for the aforementioned project, etc.

c) *The methodology to estimate.* To estimate costs using several methodologies, training and procedures that estimate the costs, which in turn can be probabilistic or deterministic in nature.

2. Types of Cost Estimates

During the life cycle of a project are made different estimates. These estimates are used for the various functions of project management. (s. Fig. 2).

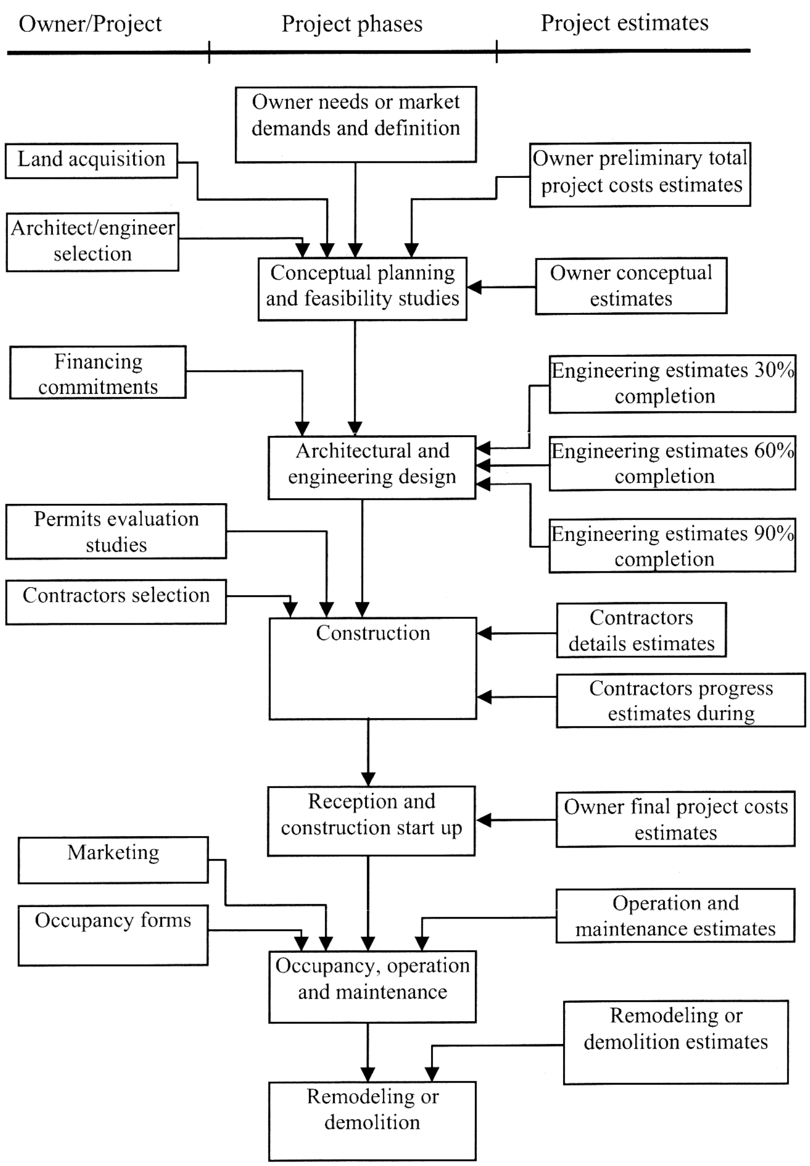

Fig. 2 – Costs estimates during construction life cycle.

Thus in an early stage of development of the construction project, the recipient needs an estimate of the likely cost of the project (investment) to assess the financial feasibility of its. This estimate is made available having a minimum volume of information as is required time when the investor has a vague idea of what wants to do. Once the design is started you can make estimates of the budgets of the various stages of project implementation. These budgets are grouped in a cost plan which is a summary of all project costs for construction. The size of this budget is checked periodically during the detail design, using the method of estimation of increasingly accurate with the design details of implementation. Also, in the initial budget period to undergo, changes and corrections are aiming to stick to the costs adopted in the content of the feasibility study. In case of correlation, are estimates for design alternatives. Finally, estimates are used to anticipate and analyse beneficiary bids. Estimates are also made after the start of actual implementation of the project, for cost control. Contractors made estimates of the maximum cost of their activities in such a way that with the amounts offered and contracted. During implementation estimated actual cost of works carried out, comparing them with limited costs and made the necessary corrections. Also during the execution, beneficiary estimate prices for the changes requested in the project and analyse their impact to the feasibility study and initial contract execution.

3. Role of Estimating in the Construction Industry

Estimates serve a number of different functions in the construction industry (s. Fig. 3). In the early stages of a construction program, the owner needs an estimate of the probable cost of construction. This conceptual estimate has to be prepared from a minimum amount of information because it is required at a time when the project is often little more than a vague idea in the mind of the owner. Once the design of the project is underway, budget amounts can be established for the various elements of the project. When the design is completed, a final pre-bid estimate can be compiled to anticipate the contractor's bid price for work. If this estimates is accurate, the obtained bid prices will be within the owner's budget for the project. Most contracts that transpire in the construction industry results from competing bids from contractors to suplly goods and services to meet certain specification for a stipulated sum of money. Estimates are also required after works starts on the project. In the cost control programs, estimating is required to facilitate the control of expenditure of founds on a project.

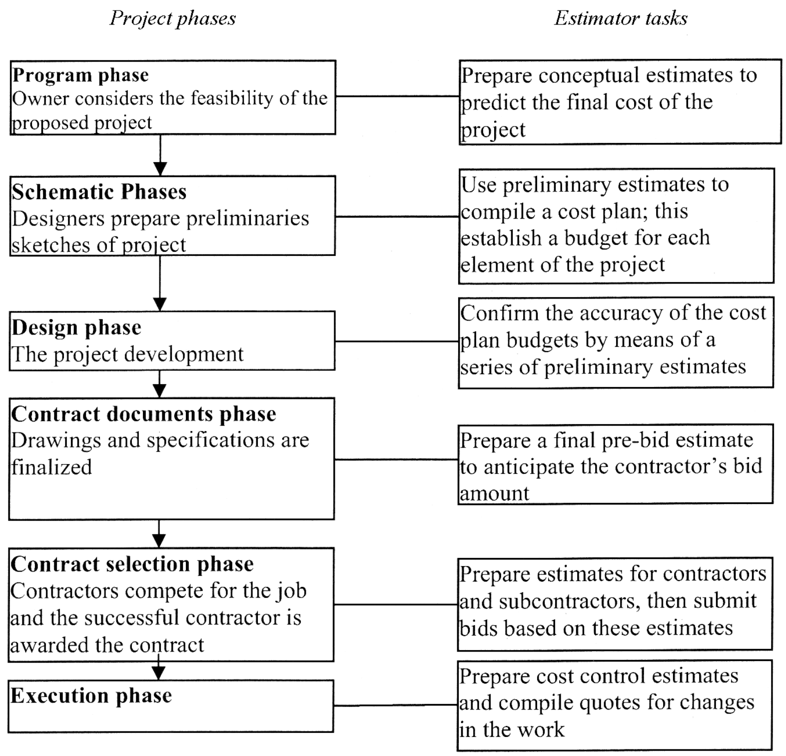

Fig. 3 – Role of the estimator of construction costs.

4. Conclusions

Estimating serves a number of purposes in the construction process including preparation of bids and cost control. The role of the estimator includes preparing conceptual estimates, preliminary estimates, pre-bid estimates, post-bid estimates, cost control, final project cost, cost on the operating construction period and cost of remodeling or demolition of the construction.

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ROLUL ESTIMĂRILOR PE PERIOADA DE VIAȚĂ A UNUI PROIECT DE CONSTRUCȚIE

(Rezumat)

Estimările sunt utilizate pentru funcţii diferite în industria construcţiilor. Motivul pentru estimare este cel de a oferi cea mai realistă predicție în termeni de timp şi cost în orice etapă a unui proiect. Pe durata de viață a unui proiect estimatorul ar trebui să fie în măsură să producă o serie de estimări, de la o estimare anticipată la început, la estimarea finală la terminarea proiectului, în care precizia tot mai mare de estimare este reflectată în scăderea riscului şi incertitudinilor. Rolul estimatorului sau practicianului în ingineria costurilor este vital pentru succesul unui proiect de construcție. Estimatorii sunt responsabili pentru estimarea costurilor într-un mod clar şi limpede. În ce priveşte realizarea costurilor şi încadrarea în cheltuielile cuprinse în studiul de fezabilitate, responsabilitatea este a managerilor de proiect. Se face o trecere în revistă a diferitelor tipuri de evaluări ce pot apare pe perioada de viaţă a unei construcţii, evidenţiind rolul estimatorului sau a practicianului în ingineria costurilor în succesul unui proiect de construcție.

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FORM AND SPACE ABSTRACTION IN ARCHITECTURAL REHABILITATION OF INDUSTRIAL BUILDINGS

BY

RADU ANDREI

Abstract. In classical art, form was considered to be reproduced in the most faithful way, representing a naturalistic reality. On the other hand, contemporary art strive to achieve its value by abstraction, and by intelectual and psychological approach. Corresponding to contextual art, the architectural rehabilitation of industrial buildings opens new paths towards form and space abstracting.

Key words: architectural rehabilitation; form; abstracting; industrial buildings.

1. Introduction

Our evolution as human beings, either of material or spiritual nature, is based on the experiences we have in relation to outside world. In this interaction, the first element setting the connection between the two aspects is our capacity to conceive, through feelings, the conditions of outside environment. Without this perception, man can not reason, can not make valuable judgments or can not interpret a certain message – either of rough – physiological or subtle nature.

2. The Balance between Clarity and Abstractly in Architecture

The matter around us is the environment through which the idea is transmitted. The phenomenon is alike in art. Without having a perception object, the aesthetical experience does not take place in abstract. The modern art developed towards an abstractization pushed in the extreme.

At the exhibition of Timothy Binkley in 1969 at the Amsterdam Art and Project Gallery, the artist informed the manager of the hired space to lock it and to post the announcement. “For the period of exhibition, the galleries will be closed”. However, we can not know if the author did it as an artist or from the perspective of a critic. We can draw the common sense conclusion that the abstraction in pure state can not aspire to the art concept, being only speculation – that do not need artistic talent.

One could be tempted to consider that, in art, the relation between matter and abstraction is of inversely proportionality through the opposition between *mundus sensibilis* and *mundus inteligibilis*, as Plato names the world of senses and the spiritual one of ideas. But is not like that. According to the Theory of Media, [1], the aesthetical elements depend in some way on the non-aesthetical elements; therefore they depend on the medium they were created in. But this medium is not a simple material, but a system of conventions delimiting the domain in which the physical elements and aesthetical qualities are related.

In other words, the value of the artwork consists in the “significant form”, [2]. If this element is not present in the work, the art becomes descriptive, says Clive Bell, and has only a historical value, not artistic. “The art conveys us from the world of human activity in a world of aesthetical exaltation. For one moment, we are disconnected from human concerns; our anticipation and memories are stopped. We are raised above the life whirl”.

Hence, the artwork means more than we perceive with our sense, it is a way by which we contact the essence of human being, beyond the reasoning and physiological aspects.

Hans Georg Gadamer consider that the artwork is not fair to be proposed to a consumer, in view of a pleasure deprived of obligations. The artist wants to provoke, to irritate and many of them start their work only as a kind of proposal, inviting the others to enter in a continuing, post-figurative activity.

We shall try to apply all these principles in theorizing the industrial architecture rehabilitation through analysing the space as a creative resource. Firstly, the space is a subtle quality of architecture that can be perceived only by physical elements which delimit it – walls, floors, volumes, etc. As the form has less significance and involves less interpretation liberty, the architecture is limiting towards the physical frame, becoming more of a machine and approaching to “the descriptive art”. It can have documentary historical value, but, in order its significance not to stop here, it should aspire to the status of authentic art. But the aesthetical experience is conditioned by perception, by the world of senses.

Cosmin C a c i u c, in an article on industrial decommissioning [3], brings Martin Heidegger into discussion who “reminded us in Being and in time that we, the humans, lost almost entirely the ability to feel. The five fundamental senses were anaesthetized by the effect of various technological extensions. The industrial site, opposed to the industrial area, arouses by "trailing" with all the sense awakened, similar in some way to wild animals that "feel" danger. This type of thinking is named by Martin Heidegger *Besinnung* ("meditative thinking").

The architect Cosmin Caciuc considers that "the type of computing knowledge is constituted of fragments of iceberg, taken out from a hypothetical Pangaea of the rationality, chaotically floating on an ocean of meditative knowledge. The revelation produced by the functional death of an industrial building or the decommissioning of a technological device, is equivalent to a spontaneous melting process of the iceberg and of the returning of its substance into the ocean that made it. Only the aggregation state separates the reason from irrational because, in fact, the "substance" of knowledge is the same: the phenomenological essence. The iceberg offers us the illusion of a *terra ferma* to step on and to move in knowledge. What matters is the submersed part of the ice or, better said, the potential of "reversed rationality", of the rational totally immersed in irrational".

Therefore, the rehabilitation of an industrial building consists in fact of shattering the illusion of progress and reintegration of space in nature. The building becomes by itself after the extinct of its utilitarian function and of its initial logic, as a natural form, a relief form delimiting the building space.

The industry and techniques, as Martin H e i d e g g e r estimates [4], are in an opposite position with Nature, being a relation of domination. The industry is the means by which the man requests resources from the natural environment, without giving back anything. The circuit of the elements in nature, by which it regenerates itself, is interrupted and an imbalance is created. In this way, a vicious circle is created through which the man is losing the link with nature, the force that created him, but because he is a component part of this system, he loses the link with himself.

This discontinuity, as a consequence of the dominating relation, can be read also at the spatial and compositional level of the industrial architecture. The spaces are closed towards inside. The building should respond to the needs of technological flow, its utility being the dominating quality and that determining the space organization.

But when it is desirable to carry out a readapting of these spaces for social functions, the first thing to be considered is surpassing this pure physical stage. The space should be redefined in order to offer an attestation of the remaking of continuity between man and nature, between man and himself. The space should become a work of art. And "the accomplished work of art is always a successful attempt to reunite of what is disjointed presented" – in our case man/nature and industrial space.

Although the matter and the abstract are not in a relation of inversed proportionality, the excess matter leads to exaggeration of the descriptive side of an object, therefore in order to get a detachment effect from physical side, from spiritualization (characteristic and necessary to man by its nature), the form should gain on the "significance" side.

An "opening" should be made inviting to a "continuing, post-figurative activity" as Hans Georg Gadamer suggested.

In the many cases of industrial space rehabilitation, the architects, in order to get this approach to human side needing aesthetical experiences, look for dematerializing the architecture, to recreate or reestablish the connection between man–nature/his nature.

a) A project that reveals a kind of paradigm of mentality changes recorded in the 20th century art presentation, is that of the architects Herzog & de Meuron of Tate Gallery of Modern Art, Bankside, London (Fig. 1).

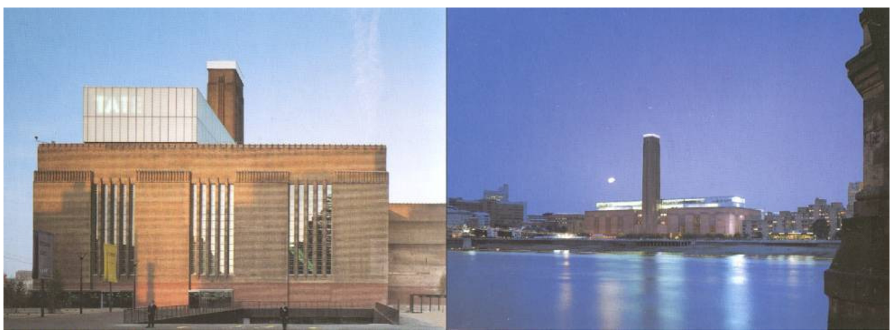

Fig. 1 – Tate Gallery of Modern Art, Bankside, London.

A former electric power station, the building is, bit its features, by its position and by the specific of its activity, an enclave hard to integrate in the urban texture. With a steel and alloy structure, with numerous details, with a brick façade, the conversion should preserve the data of the original and in the same time to adapt to the requirements of a museum of such complexity: different lighting conditions, access facilitation to visitors in order to form their own route, rooms of various dimensions depending on the types of works to be exhibited, rooms for video art and mixed media, etc. The central hall (of the turbine), the most important space of the building, was transformed in a place to exhibit sculpture or for performance type actions, etc.; in the central area, three rooms were grouped, provided with many entries and exits, for free access of visitors; in the northern part, new walls and roofs were created in order to provide natural lighting.

All these started from the premise that the architecture should serve as an instrument, as a stimulus in displaying and receiving the work of art (for as much in the case of contemporary art works where any transfer of significances is equally easy and risky) and not as a restriction.

In this case, the architects choose for dematerialization of the construction not through physical fragmentation of the masonry massive crust, but with the help of an optical illusion: the extension at the roof level of a light colour passes through the whole length of the construction and is integrally lighted by night, and by its reflection in the River Thames. The light framing thus the weight of the former power station, creates the illusion of reducing the masonry compact mass.

b) In Fig. 2 is represented the Contemporary Temporary Museum, Los Angeles, USA (Architect Frank O. Gehry).

Fig. 2 – Contemporary Temporary Museum, Los Angeles, USA.

The suggested ensemble for conversion was made of a storage room, a former garage and a car service station.

The most visible architectural intervention is the pergola carried out of metallic structure that Gehry considered vital for getting out of anonymity of the building and rediscovering an identity. This element is the one contributing to the dematerialization in the building perception. The pergola is the one that offer the illusion of an uncertain limit of construction; it is a transparent intermediary space preceding the massiveness of the industrial building. The intervention extends the indoor space towards outdoor, achieving a continuity of the route.

The indoor space offers a complete flexibility contrasting the apparent outdoor rigidity, the only partitions being suspended by the roof structure and being able to be changed depending on the needs.

The Museum was closed in 1992 because of economical reasons and reopened in 1995 – being transformed – by the architect – in destinations for education.

A new volume was added in order to shelter the reading room and the coffeehouse.

c) In Fig. 3 sugar factory conversion – Eridania Paganini Auditorium (Architect Renzo Piano) – is represented.

The conversion of the sugar factory into an auditorium was possible due to an ideal series of factors: the main body of factory, accommodating the main functions of the auditorium (lounge, hall and stage) observes the basic criteria of acoustic proportions and the surrounding park simplifies the potential problems of sound insulation.

The solution proposes the elimination of the transverse side walls of the factory and their replacement with two walls – glass curtain with a total opening of over 90 m, discovering a panoramic perspective towards the park.

Fig. 3 – Sugar factory conversion – Eridania Paganini Auditorium.

The formal, as well as the functional composition is organized along the new light axis, which unites the interior spaces with the exterior.

The transparent background of the stage offers a vivid visual sight and creates a light contrast but also between the massiveness of old building crust and the transparency and the continuous changing of the landscape depending on the day time or season.

In Fig. 4 conversion of factory from Ennetbaden – Switzerland was represented. The building that was the object of this conversion belongs to an industrial complex from the 20th century located on the river banks.

The building has 7 partitions, is 35 m long, 17 m wide and 5 m high. The apartments are simple but a special attention was paid to the details. The indoor space is one of sculptural nature in which new objects were placed as in a museum. In this way, the space materiality is diminished by the apparent mobility of elements that break it up.

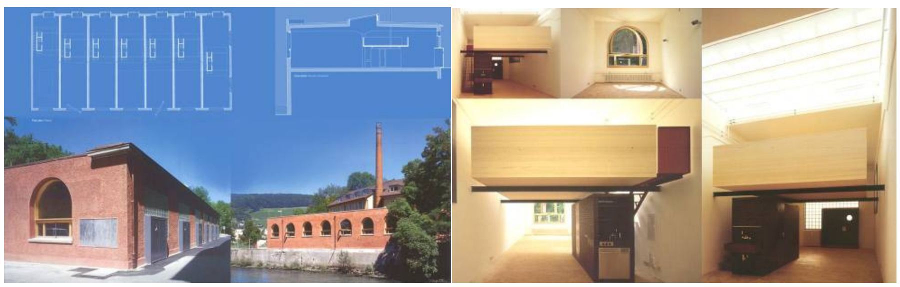

Fig. 4 – Conversion of factory from Ennetbaden – Switzerland

The first object is the split level – detached from the side walls and placed on two H metallic beams – painted in blue. The split level has a solid rampant of the same material with the floor that creates the image of a compact and massive box. The second object is the sanitary unit – a module including a small kitchen and a sanitary group. This differentiates from the rest of the setting by the colour and texture of dark bricks, creating, like in the case of the split level, an illusion of object mobility. In this way, the indoor space becomes like a free gallery where the furniture objects are placed.

The third element is the staircase – located between the sanitary unit and the wall – that by its transparency underlines the detachment of the new intervention. The staircase is not tectonically connected to existing walls, leaving the tangential light given by the roof skylight to flow along the walls.

The architect breaks the symmetry of the entrance façade in order to contrast with the opposing façade from the water, creating a solid/empty play perceptible also from inside and outside. This new language brings another rhythm and enriches the aesthetical language of the façade.

d) The gasometer is represented in Fig. 5. The building, a former furnace in Ruhr area, is not one of the most unusual places of exhibition in Europe. The huge metal room offers the experience of an echo with 7…8 periods, and the dome covering the space offers a light play revealing the space in its entire splendor, creating a kaleidoscopic, immaterial image, contrasting with the massiveness and the rejection of the external form.

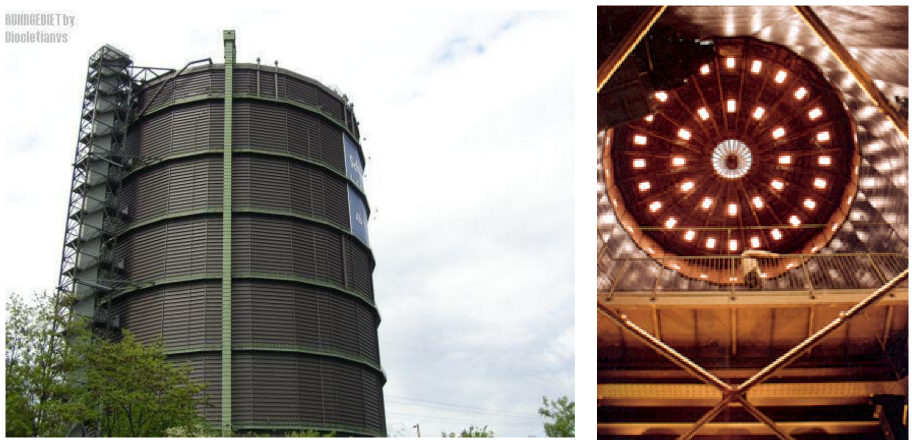

Fig. 5 – Gasometer – Ruhr area.

4. Conclusions

The abstracting of form and space can play a therapeutic role in the rehabilitation of architecture and society, because only through dematerialization of matter, in the sense of “spiritualization”, one can find the fertile ground for evolution, without losing the contact with solid history. This is a challenge for the creative and artistic side of architect, who is called to “open” the rigid box and enable multiple interpretation of the object.

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ABSTRACTIZAREA FORMEI ŞI SPAȚIULUI ÎN REABILITAREA CLĂDIRILOR INDUSTRIALE

(Rezumat)

În arta clasică, forma trebuia reprodusă cât mai fidel, într-un mod naturalist. Arta contemporană, însă, îşi caută valoarea prin abstractizare, printr-o abordare intelectuală şi psihologică. Împărtăşind din punct de vedere compozițional aceleaşi principii cu arta contextuală, reabilitarea arhitecturală deschide noi căi de abstractizare a formei şi spațiului.

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EXPERIMENTAL INVESTIGATION IN DEVELOPING LOW COST CONCRETE FROM PAPER INDUSTRY WASTE

BY

R. SIRINIVASAN, *K. SATHIYA and M. PALANISAMY

Abstract. Over 300 million tones of industrial wastes are being produced *per annum* by chemical and agricultural process in India. These materials pose problems of disposal and health hazards. The wastes like phosphogypsum, fluorogypsum and red mud contain obnoxious impurities which adversely affect the strength and other properties of building materials based on them. Out of several wastes being produced at present, the use of phosphogypsum, fluorogypsum, lime sludge, hypo sludge, red mud, and mine tailing is of paramount significance to protect the environment.

Paper making generally produces a large amount of solid waste. Paper fibers can be recycled only a limited number of times before they become too short or weak to make high quality paper. It means that the broken, low-quality paper fibers are separated out to become waste sludge. All the inks, dyes, coatings, pigments, staples and "stickies" (tape, plastic films, etc.) are also washed off the recycled fibers to join the waste solids. The shiny finish on glossy magazine-type paper is produced using a fine kaolin clay coating, which also becomes solid waste during recycling. This paper mill sludge consumes a large percentage of local landfill space for each and every year. Worse yet, some of the wastes are land spread on cropland as a disposal technique, raising concerns about trace contaminants building up in soil or running off into area lakes and streams. Some companies burn their sludge in incinerators, contributing to our serious air pollution problems. To reduce disposal and pollution problems emanating from these industrial wastes, it is most essential to develop profitable building materials from them. Keeping this in view, investigations were undertaken to produce low cast concrete by blending various ratios of cement with hypo sludge.

This project is concerned with experimental investigation on strength of concrete and optimum percentage of the partial replacement by replacing cement *via* 10%, 20%, 30%, 40%, 50%, 60% and 70% of Hypo Sludge.

Key words: hypo sludge; pozzolanic property; supplementary cementitious materials.

1. Introduction

1.1 General

Energy plays a crucial role in growth of developing countries like India. In the context of low availability of non-renewable energy resources coupled with the requirements of large quantities of energy for Building Materials like cement, the importance of using industrial waste cannot be underestimated.

During manufacturing of 1 tones of Ordinary Portland Cement (OPC) we need about 1…1⅓ t of earth resources like limestone, etc. Further during manufacturing of 1 t of Ordinary Portland Cement an equal amount of carbon-di-oxide are released into the atmosphere. The carbon-di-oxide emissions act as a silent Killer in the environment under various forms. In this Backdrop, the search for cheaper substitute to OPC is a needful one.

1.2. Solid Waste from Paper Industry

a) *Hypo Sludge Properties*

Where, this hypo sludge contains, low calcium and maximum calcium chloride and minimum amount of silica. Hypo sludge behaves like cement because of silica and magnesium properties. This silica and magnesium improve the setting of the concrete.

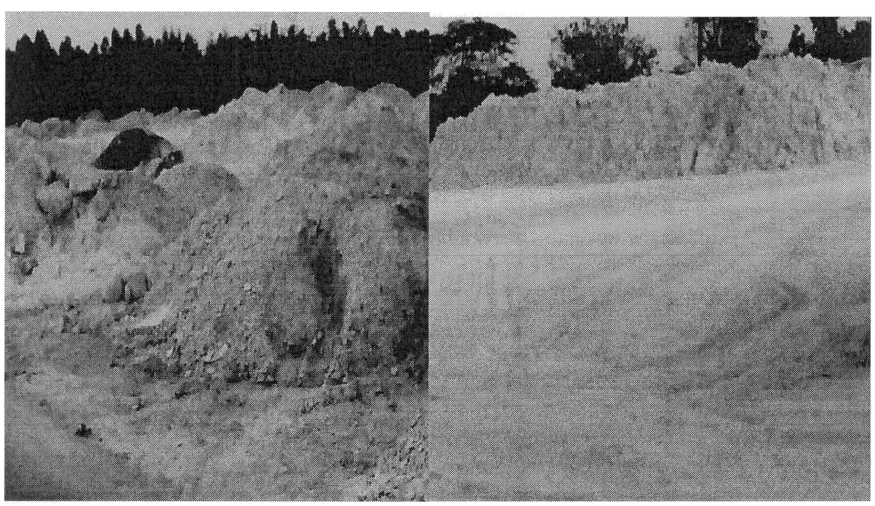

Fig.1 – Raw Hypo sludge disposal from TNPL

b) *Need for Hypo Sludge Utilization*

While producing paper the various wastes are comes out from the various processes in paper industries. From the preliminary waste named as hypo sludge, due to its low calcium is taken out for our project to replace the cement utilization in concrete. Due to the cement production green house gases are emitted in the atmosphere. For producing 4 million t of cement, 1 million t green house gases are emitted. Also, to reduce the environmental degradation, this sludge has been avoided in mass level disposal in land. To eliminate the ozone layer depletion, production of cement becomes reduced. For this, the hypo sludge is used as partial replacement in the concrete as high performance concrete. By utilizing this waste the strength will be increased and also cost reduction in the concrete is achieved.

1.3. Objectives

To investigate the utilization of Hypo Sludge as Supplementary Cementitious Materials (SCM) and influence of these hypo sludge on the Strength on concretes made with different Cement replacement levels.

1.4. Scope

a) To provide a most economical concrete.
b) It should be easily adopted in field.
c) Using the wastes in useful manner.
d) To reduce the cost of the construction.
e) To promote the low cost housing to the E.W.S. group people.
f) To find the optimum strength of the partial replacement of concrete.
g) Minimize the maximum demand for cement.
h) Minimize the maximum degradation in environment due to cement and safeguard the ozone layer from green house gases.
i) To study the crack development in hardened concrete.

1.5. Methodology

a) Tested the material properties as per Indian standards code (IS 383 – 1996) procedures.
b) Mix design for concrete proportion has been developed as per IS 10262 – 1982.
c) Casted and cured the concrete specimens as per Indian standards procedures.
d) The characteristic strength of hardened concrete specimen was tested as per IS 456 – 2000.
e) Finding the optimum strength of optimum replacement of hypo sludge as cement.

f) Compare the results of conventional concrete and partial replacement concrete.

2. Literature Review

Felix F.U d o e y o, Hilary I n y a n g, David T.Y o u n g and Edmund E. O p a r a d u are the authors of a recent paper in this field of articles [1]. The enormous amount of wastes produced during wood processing operations in many countries provides challenging opportunities for the use of wood waste as a construction material. In this research, wood waste (saw dust and wood shaving) ash (WWA) of pretreated timber of 0, 5, 10, 15, 20, 25and 30% by weight of cement was added as a supplement to a concrete of a mix proportion 1:2:4:0.56 (cement: sand: coarse aggregate: water cement ratio), and the strengths and the water absorption of the matrix were evaluated. Also, the metal leachability of WWA was analysed. The compressive and the flexural strengths of WWA concrete for the ages investigated ranged from 12.83 to 28.66 N/mm^2, and 3.652 to 5.57 N/mm^2, respectively, with the lowest values obtained at 30% additive level of ash. When compared with strength of plain concrete (control), the compressive and the flexural strengths of WWA concrete were between 62 and 91% and 65 and 95%, respectively, of the former. The trend of the water absorption of WWA concrete was a reversal of those of the strengths, that is, the highest water absorption values were recorded for the concrete specimens with the highest additive level of ash. A batch leaching test also performed at an ash-leachant volumetric ratio of 20 produced leachate containing chromium, arsenic, iron, copper and zinc with the following concentrations:410; 6,720; 150; 280 and 1,690 μg/L, respectively, when leached at a pH = 4, and 400; 10; 670; 0; 100; 1,470 μg/L, respectively, when leached at a pH = 5. These concentration levels exceed the EPA fresh water acute criteria limits.

Shi C o n g K o u, Chi S u n P o o n and Dixon C h a n published the results of a research [2] concerning the use of high percentages of recycled aggregates in concrete which would usually worsen the concrete properties. This paper tries to address the deficiency of the use of recycled aggregates by systematically presenting results on the influence of incorporating Class F fly ash on concrete properties. In this study, two series of concrete mixtures were prepared with water-to-binder (W/B) ratios of 0.45 and 0.55; the recycled aggregate was used as 0, 20, 50, and 100% by weight replacements of natural aggregate. In addition, fly ash was used as 0, 25, and 35% by weight replacement of the cement. The obtained results showed that the compressive strengths, tensile strength and static modulus of elasticity values of concrete at all ages decreased as the recycled aggregate and the fly ash contents increased. Further, an increase in the recycled aggregate content decreased the resistance to chloride ion penetration and increased the drying shrinkage and creep of concrete. Nevertheless, the use of fly ash as a substitute for cement improved the resistance to chloride ion penetration and decreasing the drying shrinkage and creep of the recycled aggregate concrete. The results showed also that one of the practical ways to utilize a high percentage of recycled aggregate in structural concrete is by incorporating 25…35% of fly ash as some of the drawbacks induced by the recycled aggregates in concrete could be minimized.

3. Materials Used

3.1.Cement

The most common cement used is an ordinary Portland cement. The Type 1 is preferred according to IS269-1976, which is used for general concrete structures. Out of the total production, ordinary Portland cement accounts for about 80…90%. Many tests were conducted on cement; some of them are consistency tests, setting tests, soundness tests, etc.

3.2. Aggregate

Aggregates are the important constituents in concrete. They give body to the concrete, reduce shrinkage and effect economy. One of the most important factors for producing workable concrete is good gradation of aggregates. Good grading implies that a sample fractions of aggregates in required proportion such that the sample contains minimum voids. Samples of the well graded aggregate containing minimum voids require minimum paste to fill up the voids in the aggregates. Minimum paste will mean less quantity of cement and less water, which will further mean increased economy, higher strength, lower shrinkage and greater durability.

Aggregate comprises about 55% of the volume of mortar and about 85% volume of mass concrete. Mortar contains a size of 4.75 mm and concrete contains aggregate up to a maximum size of 150 mm.

a) *Coarse Aggregate*

The fractions from 80 mm to 4.75 mm are termed as coarse aggregate.

b) *Fine aggregate*

Those fractions from 4.75 mm to 150 micron are termed as fine aggregate.

3.3. Water

Water is an important ingredient of concrete as it actually participates in the chemical reaction with cement. Since it helps to from the strength giving cement gel, the quantity and quality of water is required to be looked into very carefully.

3.4. Hypo Sludge

The Tables 1,…,4 and Fig. 2 show the hypo sludge chemical properties and comparison between cement and hypo sludge.

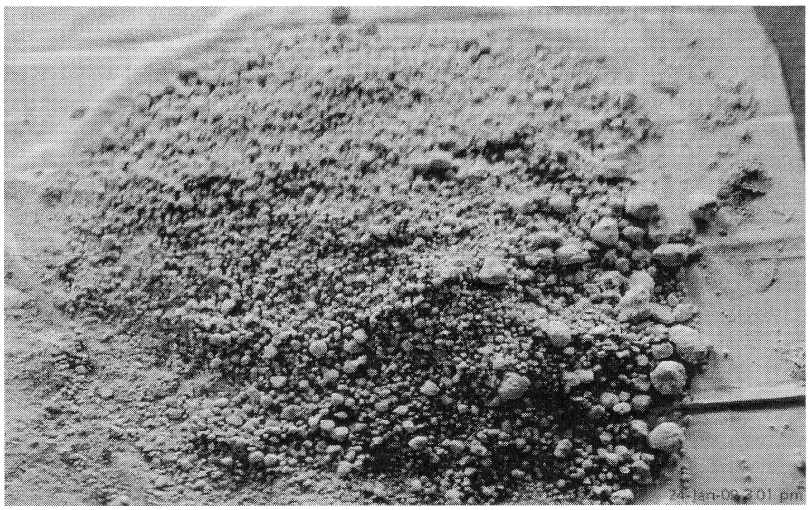

Fig. 2 – Factory outlet hypo sludge.

Table 1
Properties of Raw Hypo Sludge

Sl.No	Constituent	Present in Hypo Sludge, [%]
1.	Moisture	56.8
2.	Magnesium oxide (MgO)	3.3
3.	Calcium oxide (CaO)	46.2
4.	Loss on ignescent	27.00
5.	Acid insoluble	11.1
6.	Silica (SiO_2)	9.0
7.	R_2O_3	3.6

Table 2
Properties of Hypo Sludge as Cement Ingredient

Sl.No	Constituent	Present in Hypo Sludge, [%]
1.	Magnesium oxide (MgO)	3.3
2.	Calcium oxide (CaO)	46.2
3.	Loss on ignescent	27.00
4.	Acid insoluble	11.1
5.	Silica (SiO_2)	9.0
6.	R_2O_3	3.6

Table 3
Comparison of Cement and Hypo Sludge

Sl. No	Constituent	Cement, [%]	Hypo Sludge, [%]
1.	Lime(CaO)	62	46.2
2.	Silica(SiO_2)	22	9
3.	Alumina	5	3.6
4.	Magnesium	1	3.33
5.	Calcium sulphate	4	4.05

Table 4
Setting Time for Cement and Hypo Sludge

Sl. No	Ingredients	Initial, [min]	Final, [min]
1.	Cement + 0% hypo sludge	30	600
2.	Cement +10% hypo sludge	31	598
3.	Cement +20% hypo sludge	33	597
4.	Cement +30% hypo sludge	34	595
5.	Cement +40% hypo sludge	36	593
6.	Cement +50% hypo sludge	37	592
7.	Cement +60% hypo sludge	38	591
8.	Cement +70% hypo sludge	40	590

4. Mix Design

A mix M_{25} grade was designed as per Indian Standard method and the same was used to prepare the test samples. The design mix proportion is done in Table 5.

Table 5
Design Mix Proportion

	Water	Cement	Fine aggregate	Coarse aggregate
By weight, [kg]	191.6	547.42	456.96	1255.475
By volume	0.35	1	0.834	2.29

4.1 Mix Proportions

Conventional Concrete	1: 0.834 : 2.29
10% replacement	0.9: 0.834 : 2.29
20% replacement	0.80:0.834:2.29
30% replacement	0.70:0.834:2.29
40% replacement	0.60:0.834:2.29
50% replacement	0.50: 0.834: 2.29

60% replacement	0.40: 0.834: 2.29
70% replacement	0.30: 0.834: 2.29

5. Details of the Experimental Study

5.1 Compressive Strength Test

150 mm × 150 mm × 150 mm concrete cubes were casting using M25 grade concrete. Specimens with ordinary Portland cement (OPC) and OPC replaced with hypo sludge at 10%, 20%, 30%, 40%, 50%, 60% and 70% levels were cast. During casting the cubes were mechanically vibrated by using a table vibrator. After 24 h the specimens were removed from the mould and subjected to water curing for 14 and 28 days. After curing, the specimens were tested for compressive strength using a calibrated compression testing machine of 2,000 kN capacity.

5.2. Split Tensile Strength Test

Split tensile strength of concrete is usually found by testing plain concrete cylinders. Cylinders of size 150 mm × 300 mm were casting using M25 grade concrete. Specimen with OPC and OPC replaced by hypo sludge at 10%, 20%, 30%, 40%, 50%, 60% and 70% replacement levels were cast. During moulding, the cylinders were mechanically vibrated using a table vibrator. After 24 h the specimens were removed from the mould and subjected to water curing for 28 days. After curing, the specimens were tested for compressive strength using a calibrated compression testing machine of 2,000 kN capacity.

6. Results and Discussions

The obtained results are given in Tables 6,…,8 and Figs. 3,…,5.

Table 6
Compressive Strength of Cubes at 14 Days

Partial replacement %	Number of specimen	Initial crack load, [kN]	Ultimate load, [kN]	Ultimate compressive strength, [N/mm^2]
0	3	193.000	400.725	17.81
10	3	23.850	577.575	25.67
20	3	328.650	764.100	33.96
30	3	360.550	798.750	35.5
40	3	215.950	499.500	22.2
50	3	173.250	348.750	15.5
60	3	128.650	279.000	12.4
70	3	92.550	193.500	8.6

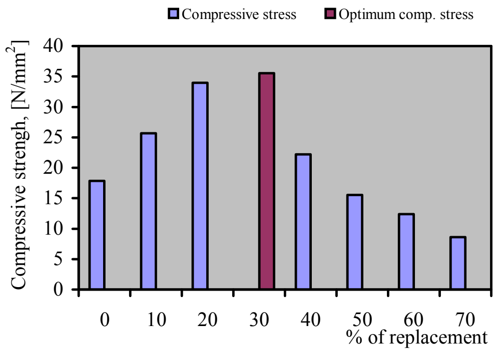

Fig. 3 – Compressive strength of concrete specimen at 14 days.

Table 7
Compressive Strength on Cubes at 28 Days

Partial replacement %	Number of specimen	Initial crack load, [kN]	Ultimate load, [Kn]	Ultimate compressive strength, [N/mm^2]
0	3	697.100	839.925	37.33
10	3	810.300	908.325	40.37
20	3	948.250	1,253.025	55.69
30	3	925.950	1,262.475	56.11
40	3	720.00	898.875	39.95
50	3	308.350	412.537	18.335
60	3	175.650	357.075	15.87
70	3	115.850	291.150	12.94

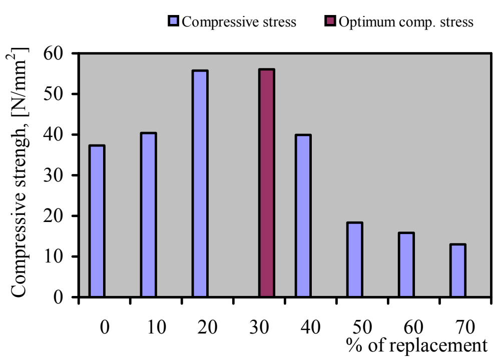

Fig. 4 – Compressive strength of concrete specimen at 28 days.

Table 8
Split Tensile Strength of Cylinder at 28 Days

Partial replacement %	Number of specimen	Ultimate load, kN	Split tensile strength, [N/mm^2]
0	3	130.061	1.84
10	3	110.269	1.56
20	3	104.615	1.48
30	3	100.373	1.42
40	3	98.253	1.39
50	3	97.546	1.38
60	3	101.080	1.43
70	3	102.494	1.45

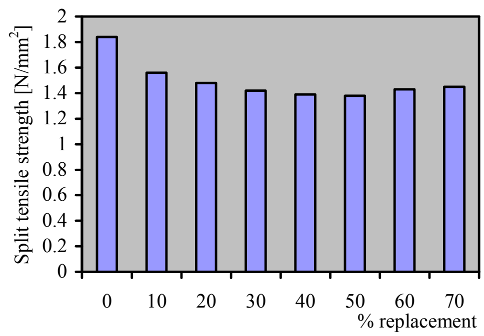

Fig. 5 – Split tensile strength of concrete specimen at 28 days.

6.1. Economic Feasability

Cost analysis is carried out for the optimum proportion of percentage of hypo sludge in concrete. This project was carried out in our college campus. The cost is compared to the conventional concrete.

a) *Cost of Materials*

Cost of cement per bag = 5.37 \$
Cost of sand per m^3 = 18.64 \$
Cost of hypo sludge per kg = 0.01\$
Cost of coarse of aggregate per m^3 = 12.07 \$
(All the rates are included with lead charges).

Table 9

Cost of Material of Normal Cncrete/ m³

Description	Quantity, [kg/m³]	Cost, [$]	Cost of material, [$]
Cement	547.42	0.11/kg	58.83
Hypo sludge	–	0.01/kg	–
Sand	456.96	18.64/m³	5.32
Coarse aggregate	1,255.475	12.07/m³	6.89
Total cost			71.04

Table 10

Cost of Material of 10% Partially Replaced Concrete/m³

Description	Quantity, [kg/m³]	Cost, [$]	Cost of material, [$]
Cement	492.678	0.11/kg	52.95
Hypo sludge	54.742	0.01/kg	0.59
Sand	456.96	18.64/m³	5.32
Coarse aggregate	1,255.475	12.07/m³	6.89
Total cost			65.75

Table 11

Cost of Material of 20% Partially Replaced Concrete/m³

Description	Quantity, [kg/m³]	Cost, [$]	Cost of material, [$]
Cement	437.936	0.11/kg	47.07
Hypo sludge	109.484	0.01/kg	1.18
Sand	456.96	18.64/m³	5.32
Coarse aggregate	1,255.475	12.07/m³	6.89
Total cost			60.45

Table 12

Cost of Material of 30% Partially Replaced Concrete/m³

Description	Quantity, [kg/m³]	Cost, [$]	Cost of material, [$]
Cement	383.194	0.11/kg	41.18
Hypo sludge	164.226	0.01/kg	1.76
Sand	456.96	18.64/m³	5.32
Coarse aggregate	1,255.475	12.07/m³	6.89
Total cost			55.16

Table 13

Cost of Material of 40% Partially Replaced Concrete/m³

Description	Quantity, [kg/m³]	Cost, [$]	Cost of material, [$]
Cement	328.452	0.11/kg	35.30
Hypo sludge	218.968	0.01/kg	2.35
Sand	456.96	18.64/m³	5.32
Coarse aggregate	1255.475	12.07/m³	6.89
Total cost			49.86

Table 14
Cost of Material of 50% Partially Replaced Concrete/m^3

Description	Quantity, [kg/m^3]	Cost, [$]	Cost of material, [$]
Cement	273.71	0.11/kg	29.42
Hypo sludge	273.71	0.01/kg	2.94
Sand	456.96	18.64/m^3	5.32
Coarse aggregate	1,255.475	12.07/m^3	6.89
Total cost			44.57

Table 15
Cost of Material of 60% Partially Replaced Concrete/m^3

Description	Quantity, [kg/m^3]	Cost, [$]	Cost of material, [$]
Cement	218.968	0.11/kg	23.53
Hypo sludge	328.452	0.01/kg	3.53
Sand	456.96	18.64/m^3	5.32
Coarse aggregate	1,255.475	12.07/m^3	6.89
Total cost			39.27

Table 16
Cost of Material of 70% Partially Replaced Concrete/m^3

Description	Quantity, [kg/m^3]	Cost, [$]	Cost of material, [$]
Cement	164.226	0.11/kg	17.65
Hypo sludge	383.194	0.01/kg	4.12
Sand	456.96	18.64/m^3	5.32
Coarse aggregate	1,255.475	12.07/m^3	6.89
Total cost			33.98

The compared values of cost show gradual decrement in total cost of per cubic meter concrete. The above tables show cost values up to 30% replacement and the *difference in cost* from normal concrete to partially replaced concrete was **15.88 $**.

7. Conclusions

Based on limited experimental investigation concerning the compressive and split tensile strength of concrete, the following observations are made regarding the resistance of partially replaced hypo sludge:

a) Compressive strength of the concrete should be increased when the percentage of replacement is increased up to 40% and replacement increased compressive strength become reduced.

b) The split tensile strength should be decreased when the percentage of the replacement is increased.

c) From this level, replacement of cement with this waste of hypo-sludge material provides maximum compressive strength at 30% replacement.

d) We find the glory to E.W.S. group people by get the 28 days curing test. When government implement the projects for temporary shelters for who those affected by tsunami, E.Q., etc., this material can be used for economical feasibility.

e) Cost of cement should become low from this project.

f) Environment effects from wastes and maximum amount of cement manufacturing is reduced through this project,

g) A better measure by a *New Construction Material* is formed out through this project.

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INVESTIGAŢII EXPERIMENTALE PRIVIND PRODUCEREA DE BETOANE IEFTINE FOLOSIND DEŞEURI DIN INDUSTRIA HÂRTIEI

(Rezumat)

Anual, în India sunt produse peste 300 de milioane de tone de deşeuri şi reziduuri industriale datorate industriei chimice şi chiar agriculturii. Aceste deşeuri

prezintă un adevărat risc la adresa sănătății. Deşeuri precum fosfo-gipsul, fluoro-gipsul şi „noroiul roşu” conțin impurități ce pot avea efecte nedorite asupra rezistenței materialelor de construcții în a căror componență sunt folosite. Folosirea deşeurilor susmenționate, la care se pot adăuga şi deşeurile provenite din industria producătoare de hârtie, ca materiale înlocuitoare în construcții, este de un real interes atunci când este vorba de protejarea mediului.

În urma procesului de producere a hârtiei rezultă numeroase deşeuri solide. Spre exemplu, fibrele de hârtie pot fi reciclate doar de un număr limitat de ori înainte de a deveni improprii (prea scurte sau cu rezistență scăzută) producerii de hârtie de calitate. Acestea sunt date deoparte şi devin astfel deşeuri. La fel şi cerneala, vopseaua, straturile protectoare, capsele precum şi benzile adezive sunt îndepărtate din hârtia reciclată şi devin deşeuri. Hârtia lucioasă provenită din cataloage şi reviste este acoperită cu un strat subțire de caolin. Acestea din urmă, prin îndepărtare, devin deşeuri.

Deşeurile rezultate în urma procesării hârtiei reciclate trebuie depozitate în halde de steril ce ocupă suprafețe importante de teren, fiind expuse intemperiilor. Mai mult, anumite deşeuri sunt împrăştiate în zonele cultivate, sub formă de îngrăşământ, creând motive de îngrijorare în ceea ce priveşte contaminarea solurilor şi a pânzei de apă freatică. Unii producători folosesc incineratoare pentru a scăpa de aceste deşeuri, ceea ce duce la o mai mare poluare a atmosferei.

Pentru a reduce efectul acestor deşeuri asupra mediului, este de dorit folosirea lor în obținerea de materiale de construcții. Astfel, s-au făcut investigații privind producerea de betoane în care s-au folosit deşeuri provenite din industria hârtiei pentru a înlocui o anumită parte din ciment.

În lucrarea de față se prezintă rezultatele investigațiilor experimentale în ceea ce priveşte rezistența la compresiune a betoanelor în care s-a înlocuit cimentul în proporție de 10% până la 70% cu deşeuri provenite din industria hârtiei. Cercetările s-au axat şi pe dozajul optim de ciment înlocuit pentru a crea un beton de cât mai înaltă rezistență.

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BEHAVIOR'S EVOLUTION OF MICRO-CONCRETES IN COMBINED SULPHATE AND MAGNESIUM AGGRESSIVE ENVIRONMENTS, AT THREE-YEARS OLD

BY

MIRCEA RUJANU

Abstract. In the present paper is analysed concrete behavior from strength structure elements, subjected to combined sulphatic and magnesian aggressive actions. In order to clearly relieve cement influence regarding corrosion behavior and to be as close as possible of concrete structural characteristics, tests are realized on standard mortars, which from compositional point of view are micro-concretes.

There were used different W/C ratios in order to obtain different structural characteristics, knowing that in the first period after casting, when concrete porosity is sensibly higher, corrosion process is more emphatic, with subsequent negative effects.

It was taken into account the fact that sulphatic corrosion effect appears after a long duration of aggressive agent action; in the present paper a three year duration for aggressive attack is proposed.

In the same time it was considered the fact that usual cement for structural elements is composite cement type CEM II/A-S 32,5, studying combined sulphatic and magnesian corrosion evolution on a large type of micro-concrete compositions with different compactness and aggressive agent concentrations.

Key words: micro-concrete; aggressive environment; cement; corrosion.

1. Introduction

This study takes into account the fact that aggressive factors action is combined, not independently, the combination proposed being frequently used in practice.

It is proposed the analysis of the concrete behavior in the resistance structure of buildings subjected to combined sulphate and magnesium aggressive actions. To point out better the quality of the used cement, and to get closer to structural characteristics of the concrete from work, the trials will be realized on micro-concretes.

The concrete obtained must have strong specific resistances to the aggressive action because at least in the first period after its application, when the porousness of the concrete is a bit higher, the soak of concrete structure with aggressive solution can lead to emphasize process of corrosion in the initial period with unfavourable effects, in perspective.

It was taked into account the effect of the sulphate corrosion which appears after a longer period of action of the aggressive agent, and in the present paper we suggest a three years period for the aggressive attack.

Having in view that the usual cement for the structure elements has the composition cement of CEM II/A-S 32,5 (Pa35) type, the evolution of sulphate and magnesium corrosion with several compositions of micro-concretes and with different densities and concentrations of the aggressive agent were studied.

2. General Theoretical Aspects of Sulphate and Magnesium Type Corrosion

This type of corrosion is constituted like superposition of effects from the two following recognized types of corrosion:

a) *The sulphate type corrosion* is characterized by the expansion of the cement rock due to the formation of crystallized products with a higher number of crystallized water molecules.

This type of corrosion is also produced by the sodium sulphate (Na_2SO_4) which is a soluble sulphate. They react with calcium hydroxide in the cement rock and form the calcium sulphate. The calcium sulphate combines with the calcium hydroaluminium forming a complex salt which crystallizes the etringit with 31 crystallized water molecules. This tendency to form crystals makes it possible in a first phase to observe an improvement of the structure characteristics and in time it produces internal tensions which at the end lead to the destruction of the cement rock structure.

The sulphate type corrosion supposes the formation of less soluble compounds with a high molecule volume which will be deposited in the pores, the capillaries and the micro fissures of the concrete leading to internal tensions and the destruction of the concrete later on.

In the case of this type of corrosion the concrete structure plays an important role by its porousness and permeability by its contact surface between the aggressive agent and the cement rock, the volume of open and closed pores and the uniformity of their distribution.

Taking into account the complex nature of the sulphate aggressive action on concretes, the problem of the sulphate compound, which is the main cause of the destructive effect, is under discussion.

If a synthesis were to be made as far as information in this field is concerned, the following conclusions could be drawn:

(i) the nature of the corrosive processes can be modified by increasing the concentration of sulphate ions under certain limits;

(ii) at low concentrations of sulphate ions the expansion is produced by the formation and crystallization of calcium sulphate and at high concentrations of the sulphate ions the expansion is mainly produced by the crystallization of the gypsum.

b) *The magnesium type corrosion* belongs to the II-nd type of corrosion. In this case the magnesium salts operate directly on $Ca(OH)_2$ and the basic reaction which takes place in the case of the corrosion produced by the magnesium sulphate, which is investigated in this experiment, is

$$MgSO_4+Ca(OH) \rightarrow CaSO_4+Mg(OH)_2 .$$

A characteristic of this interaction is the fact that one of the reaction products, $Mg(OH)_2$, is little soluble and it is deposited in the pores, the capillaries and the micro-fissures of the cement rock. For lower concentration solutions spongy deposits are formed which can be easily penetrated by aggressive waters while at high concentrations (5…6%) compact deposits can be formed and contribute to the improvement of the behavior at aggressive actions of the magnesium sulphate solution, a fact that was proved by a number of previous researches.

3. Experimental Organization

The combination of the two types of corrosion can't be simply due to a number of effects because the mechanism of reaction products forming, in the imposed conditions, are much more complex and the concrete structure plays an important role from the point of view of his behavior in time.

For organizing the experiment, the present standards referring to the achievement of concretes in aggressive environments were taken into account and they also specify the limits for composition dosage (minimum cement dosage, maximum A/C ratio) and they also recommend the cement type according to the exposure class of the concrete. Taking into account the restrictions imposed by the standards, three recipes were obtained using poligranulate quartz sand as an aggregate and cement of CEM II/A-S 32,5 type for different values of the A/C ratio obtaining the structural characteristics indicated in Table 1.

Table 1
Compositional Characteristics

Recipe index	W/C ratio	C, [%]	P, [%]
R1	0.64	21.0	79.0
R2	0.53	17.5	82.5
R3	0.35	10.8	89.2

Prismatic test tubes of 40 × 40 × 160 mm, three prism for each recipe, were immersed in water and were kept there as witness tests until the testing time.

Three prisms of each recipe were immersed after a 28-day maturation in two parts calcium sulphate solution and one part magnesium sulphate with the following concentrations: 500; 1,000; 5,000 mg/dm^3 SO_4^{2-} and Mg^{2+} ions.

4. Experimental Results

According to the composition characteristics of the three recipes after subjecting the tests to the corrosion process, the obtained results were reproduced in Table 2.

Table 2
Structural and Mechanical Characteristics at 3 Years Old

Recipe index	Concentration mg/dm^3 SO_4^{2-} + Mg^{2+}	P_t, [%]	C, [%]	Resistance to compression N/mm^2
R1	$MgSO_4 + Na_2SO_4$, 500	21.0	79.0	40.2
R2		17.5	82.5	58.6
R3		10.8	89.2	85.6
R1	$MgSO_4 + Na_2SO_4$, 1,000	21.0	79.0	39.7
R2		17.5	82.5	59.4
R3		10.8	89.2	88.2
R1	$MgSO_4 + Na_2SO_4$, 5,000	21.0	79.0	43.4
R2		17.5	82.5	51.9
R3		10.8	89.2	72.5
R1	Drinking water	21.0	79.0	36.1
R2		17.5	82.5	50.3
R3		10.8	89.2	84.8

A graph would allow a clearer estimation of the micro-concrete behaviour subjected to the influence of sulphate and magnesium aggressively (Fig. 1).

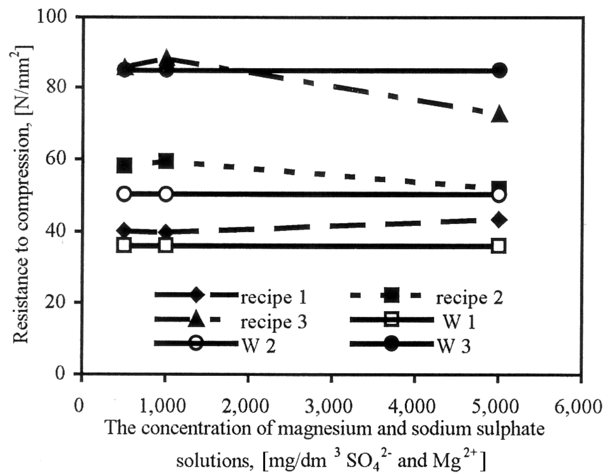

Fig. 1 – The variation of resistances at three years old in combined sulphate and magnesian corrosion.

4. Conclusions

The analysis of the obtained experimental values leads to the following conclusions:

1. Can be observed a slow decalcification to edges and corners of prisms, without a material exfoliation or tears.

2. Recipe 1 presents constantly resistance growths, lower than the others recipes, but for all concentrations in condition of a structural characteristics which shown a reduced compactness; it may consider to have the best behavior in the conditions of present experiment, because the resistances values, no matter concentrations, are presenting an approached growth and relative high values and this can be real basis for a better behavior in future;

3. Recipe 2 is giving the highest growths for concentration less than 1,000 mg/dm^3 SO_4^{2-} and Mg^{2+} ions, but the resistance growth at 5,000 mg/dm^3 is closed to the value of witness test, all those in conditions of structural characteristics which shown us a supreme compactness than recipe 1.

4. Recipe 3 has an unfavourable behavior, with less growth for reduced concentrations and a suggestive reduction of the resistance to compression for a 5,000 mg/dm^3 concentration.

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EVOLUȚIA COMPORTĂRII MICROBETOANELOR ÎN MEDII AGRESIVE COMBINATE SULFATICE ŞI MEGNEZIENE, LA VÂRSTA DE 3 ANI

(Rezumat)

În lucrarea de față se pleacă de la analiza comportării betoanelor din elementele unor structuri de rezistență ale construcțiilor supuse acțiunilor agresive combinate sulfatice şi magneziene. Pentru a pune mai bine în evidență influența cimentului privind comportarea la coroziune şi a fi cât mai aproape de caracteristicile structurale ale betonului, încercările s-au realizat pe mortare standard, care din punct de vedere compozițional sunt de fapt microbetoane.

S-au utilizat rapoarte A/C diferite pentru obținerea unor caracteristici structurale diferite, ştiind că cel puțin în prima perioadă de după punerea în operă, când porozitatea betonului este uşor mai mare, procesul de coroziune este mai accentuat, cu efecte defavorabile ulterioare.

S-a avut în vedere faptul că efectul coroziunii sulfatice apare după o durată mai mare de acționare a agentului agresiv, iar în lucrarea de față se propune o durată de trei ani pentru atacul agresiv.

De asemenea s-a avut în vedere faptul că cimentul uzual pentru elemente structurale este cimentul compozit tip CEM II/A-S 32,5, studiind evoluția coroziunii combinate de tip sulfatic şi magnezian pe mai multe compoziții de microbeton cu compactități diferite şi cu concentrații diferite ale agentului agresiv.

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CITY PLANNING EVOLUTION – URBAN DEVELOPMENT DIRECTIONS IN THE TRANSITION PERIOD

BY

MIRCEA GRIGOROVSCHI

Abstract. Urban evolution post 1989 has a series of specific characteristics mainly on a spatial–territorial plane.

Determination of the main developing factors and urban evolution directions (dimensions, rhythm, expansion level, centrifugal and axial character, concentric, centripetal, functional evolution, tendencies and social implications, etc.) represents a necessity and obligation for action from professionals in urban and landscaping fields.

This necessity even arises from the perspective of the need for realizing strategies, planning, documentation and urban studies, which must intervene correctively in the evolution of areas with structural problems and to guide urban evolution towards the main goal, namely the growth in residential quality of life in human settlements.

Key words: city; urban space; evolution; urban development; development directions; urban trends; social purpose; resources; durable development.

1. Introduction

The city can be defined as

a) an area usually comprised by several residential areas, industrial and business areas, completed with administrative functions, concerning a larger geographic zone (Wikipedia.ro);

b) complex human settlement of variable size and industrial equipment, usually having an administrative function, industrial, commercial, politic and cultural DEX;

c) a form of human community characterized by a relatively large number of people, diverse economic activity with a predominance in the secondary and tertiary sectors, and having a high degree of social and institutional organization (Gabriel Pescariu).

2. Urban Settlements

Urban settlements represent a place in which most work resources are focused on non-agricultural activities, with a diversified facility level, exerting a constant and meaningful social-economic impact, concerning the surrounding area (Law 351/2001, section h PATN – network localities).

Essentially any human settlement represents a ground settlement, on a spatial layout of society. In this way any human settlement bears the footprint of defining characteristics given by the social-economic, political and cultural climate. The city represents a social space defined as a spatial setting in which groups live, has a social structure and this groups are conditioned by ecological and cultural factors (Ch.De Lawe).

Taking into account these determinations of the spatially conditioned social settlement, it comes as a second nature for the cities to adapt to the changes in our social structure. The changes in the collective mind, in social behavior, economic, cultural of social groups of urban actors, have determined, as a consequence, fundamental changes in spatial structure of the city and they defined a series of directions in urban development.

Defining factors of urban structural diversification, of their evolution in time and space, are mainly about structural, functional and cultural diversity. Although the urban development objectives submitted in spatial studies and strategies like lowering extensive development, existing anthropic urban structure restructuring, recycling of urban areas, restructuring of the existing spatial technological configurations in durable development directions, the contemporary evolution in Romania fully contradicts this at least on some levels and directions.

3. Urban Evolution Trends

Analysing the past 20 years, we can see two distinct periods in urban evolution. The first is the transition period between 1990 and 2008, accompanied by a whole range of social phenomena, economic and cultural, developed on an undefined spatial support, chaotic at times, and responding nearly all of the time to efficiency rather than a durable and comfort based dimension of urban living.

The second period started in 2008, and stands characterize by the start of economical depression doubled by a political crisis. Also, the difference between the two periods stands only in the rhythm of evolving phenomena. If in the transition period the rhythm of urban development needed a strong spatial dynamic, without strategic backup, coherent programs and projects on short, medium and long term basis, the crisis period continues, and deepens this phenomena, but nearly favourable, in the sense that offers the involved actors the necessary time for creating and implementing spatial development strategies – urban and territorial (not the understanding this necessities). In this setting of two periods, nearly identical to each other in structure, but developing at different speeds and acceleration (actually acceleration and deceleration) we can distinguish a series of directions of urban evolution.

A first direction is about the urban expansion, the explosion of urban functions in the territory. The development of this urban functions in a great variety from habitations, commercial, services, storage, to production, did not wait for redefining internal functions of the city, of spaces and existing urban landscaping, disorganized, in an accelerated declining process, but they looked for spatial support outside the building realm.

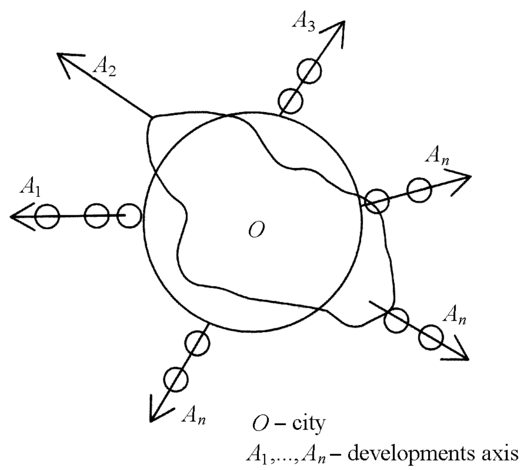


Fig. 1 – Representation of urban axial expansion.

Sometimes this expansion, centrifugal development was done at great social expense and not always located with the necessity of the urban organism in mind. One of the predominant characteristics of this spatial–territorial expansion of urban functions is the form and trends of development. A first level is represented by the evolution and occupation of development directions, generating evolutionary poles, with tendencies towards influencing and justifying future developments (Fig. 1). If we take a closer look at the city of Iaşi, the main development axis is represented by the direction of national road 28 (DN28) Iaşi–Târgul Frumos and followed closely by the axis Iaşi–Vaslui, Iaşi–Ungheni, Iaşi–Breazu, Iaşi–Ciurea.

The second level, which was developed nearly at the same time, is the centrifugal evolution in the area, of some urban functions, especially housing, namely the concentric occupation of peri-urban zones (Fig. 2).

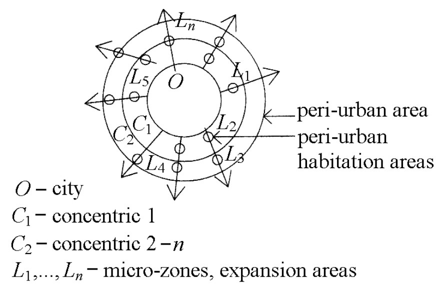


Fig. 2 – Concentric urban planning occupation.

These concentrically developed functions are generally habitation in groups, micro-areas and housing areas. Example – in Iaşi building complexes like Copou–M. Sadoveanu, I.C. Brătianu, housings in Bucium and Miroslava areas. One characteristic for both types of evolution represents the development with the lack of main urban utilities which give dimension for a durable development: water, sewage system, gas, appropriate means of communication, compatible functions (commerce, education, culture, religion, etc.).

Another characteristic is the explosion of these urban functions (although they mainly damage the city of Iaşi) on administrative areas, belonging to other entities (ex. Miroslava, Bârnova, Aroneanu, Leţcani, etc.) (Fig. 3).

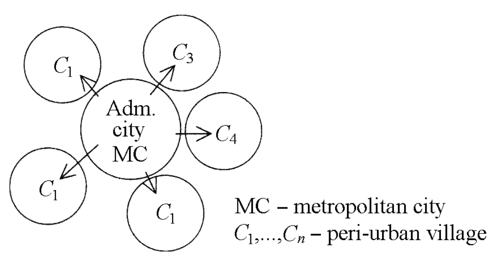


Fig. 3 – Spatial expansion representation.

In this context, the spatial expanding of the city was made including functions (non administrative-judicial) territories in peri-urban areas, belonging to neighboring villages, areas occupied with urban functions but lacking urban facilities (utilities, transport, culture, education, etc.).

This phenomenon is accompanied by a diverse range of other phenomena related to movement and population dynamics, generating transport modules varied in size and time frame. Thus we keep an eye on

a) workers commute;

b) transportation to commercial areas school and pre-school commute;

c) weekends transportation to cultural, fun and leisure activities;

d) transportation to intercity and international transport nodes: train stations, bus stations and airports;

e) transportation for different administrative and judicial needs, etc.

I would like to emphasize a constant and important urban phenomenon in inhabited areas. The tendency (natural when 80% of people in Iaşi, used to live in collective apartment buildings) of housing system development per lot, coupled or (seldom) rows (Fig. 4). This phenomenon is a great spatial consumer, to the detriment of natural landscape and terrain used in agriculture or other purposes.

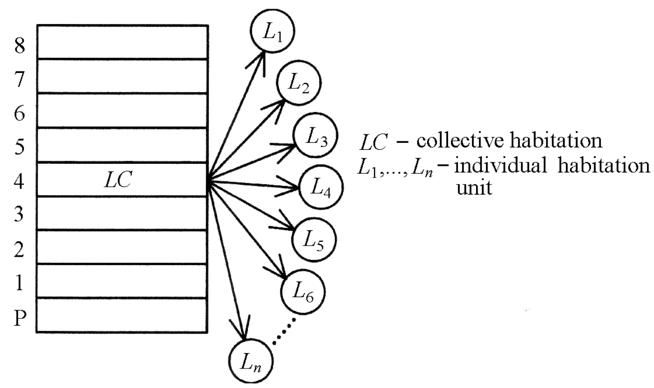


Fig. 4 – Evolution of urban living trend.

This first urban development phenomenon is simultaneously reduplicated by a complementary phenomenon, equally accelerated and present in our cities.

Until now I have discussed about centrifugal development; now we define the phenomenon of centripetal development. This phenomenon is characterized by the following aspects:

a) functional modifications;

b) functional adaptations and restructuring;

c) functional implementation: thickening;

d) overlapping (overbunking);

e) expansions.

This phenomenon takes place on the entire urban city area but especially in the downtown area, in areas with central features, in areas with great development potential. In Iaşi this phenomenon is summarized by transforming habitation spaces in offices and services all around the first floor of apartment buildings, positioning of commercial modules on the main axis of circulation, the occupation in great percentages of free areas, demolition of constructions with lower heights and erecting medium to high buildings, reassigning first floor spaces in apartment buildings (commerce, services, banks, etc.), outcasting cultural functions (cinema, bookstores, etc.) in peripheral areas.

A peculiar instance represents placing supermarkets and hypermarkets in downtown areas. Iaşi was one of the "beneficiaries" of such placing, in relatively central areas of these functions. The consequence wasn't late to arrive. Placing these functions downtown succeeded to destroy a part of the small boutiques and neighborhood shops commercial network, with their charm, but especially it destroyed the nature of these commercial avenues as social contact areas. The social life, the conscience of belonging to a community, the conscience of the existence of a personal urban area with its traditional features, of the legacy and conservation of habits and specific behavior aspects, was replaced by the globalization, standardization and the maximum efficiency phenomena. These phenomena are accompanied by the centrifugal development, by a cohort of adjacent sub-phenomena to the type of concentration, the dilution of urban functions, of social contact. If the central areas heavily concentrate the town's life, there are areas (some with tradition) transformed in peripheral or extremely peripheral (without mentioning the peri-urban areas) which if they won't become the object of some social, economical, spiritual restructuration programs, on a spatial–territorial support, are condemned to involution, to a quality of life degradation, in between their limits. Other consequent phenomena are defined by the quality *vs*. quantity difference (ex. Offices – in the first phase, any space was arranged seconded for this type of function, later special structures are buildly for offices), functional increase and decrease (commercial function increase – hypermarket, and habitation dilution in the central zone).

The horizontal-vertical binomial is another characteristic of contemporary urban development.

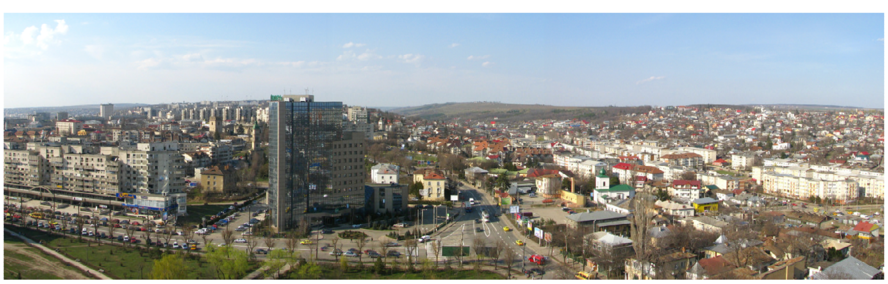


Fig. 5 – Aereal view – central zone, Europa Hotel.

Thereby some functions – especially commerce, services, storage and economic have gained horizontal spatial and volumetric expressions, others imposed (by force) a vertical development. The issue with high rise buildings in

Iaşi has a distinct place, determined by the spatial configuration of the city, placed on two elevation levels, namely

a) cornice – the traditional geometric birthing place of the city, of urban significance, of perception;

b) the under cornice areas, with great development potential for putting forward the cornice by altimetry and silhouette.

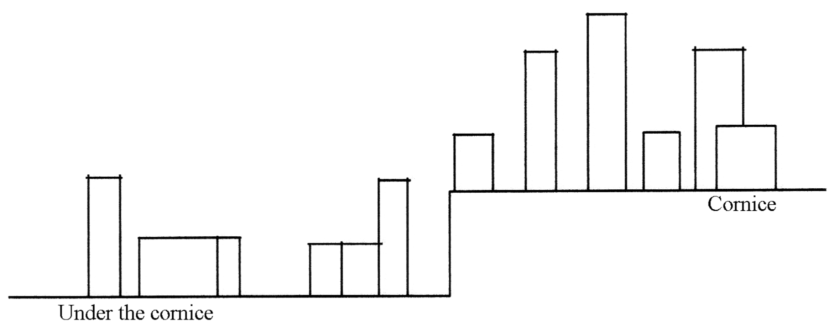


Fig. 6 – High buiding phenomenon in Iaşi.

The phenomenon of tall buildings is on the rise, in Iaşi, the magnitude of this projects is random, dictated by the necessity of efficient capitalization or extremely efficient in using the potential on terrains owned by urban actors (especially private sector, developers, realtor agencies, investors, business men, international corporations, etc.) (Fig. 6).

If the cornice emplacements in analysed areas by means of strategies, studies, developing programs, or in access areas of the city (Nicolina, Copou, Păcurari, etc.) which constitute sub-highlights of urban spaces is understandable, not understood and unacceptable is the placement of high rise buildings in areas that undermine and damage the cities silhouette (Fig. 7).

Other directions of urban evolution which appeared in the two stages of urban space evolution, could be highlighting the extinction of certain functions in the detriment of others, newer functions that didn't exist before (wholesale stores, hypermarkets, etc.)

The entire spectrum of urban phenomena had a series of consequences or has gained momentum based on real trends. In this respect we can mention

a) The structuring of mental and behavioral adaptation faced with the new spatial, territorial, social, economic and cultural conditions.

b) Structural phenomena and destructuring of existing urban textures.

c) The change in urban, cultural, and traditional landscapes, which give way in the face of globalization, a phenomenon generating new spatial and aesthetic landscaping, in the continuity of tradition, inherited, imagined, lived, reflected and projected in the existing public consciousness.

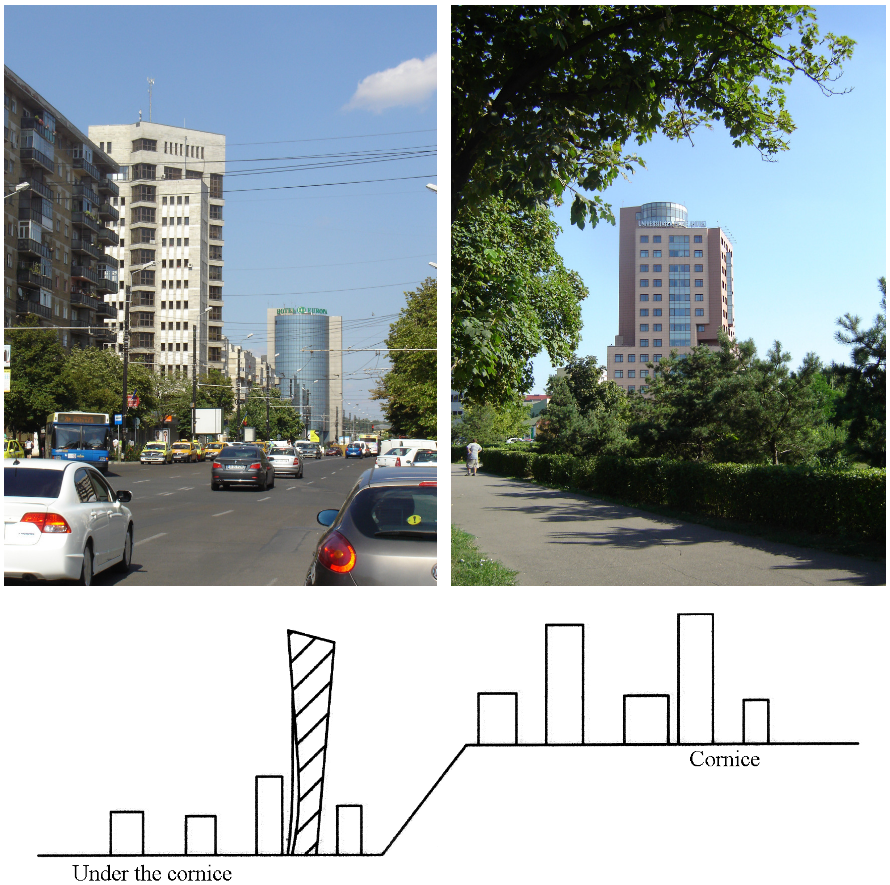


Fig. 7 – High buiding phenomenon in Iaşi.

Infact this level of globalization requires as a tendency two levels and namely

a) the level of adequacy and overlapping adequacies, additional natural over the traditional urban structure perceived by the subject (social groups, individuals) as an essential , known, friendly and specific matrix;

b) the level of irreparable damage, alienation of spatial succession in the network of public and traditional private spaces, of great importance in the existing cultural landscape (for instance emplacement of hypermarket functions with a traditional urban texture).

This phenomenon of globalization in the spatial and territorial evolution of the city doesn’t take into account the rate of consciousness adaptation, buildings, urban implementations of the new development trends.

There are great discrepancies of speed in adapting to new urban phenomena related to

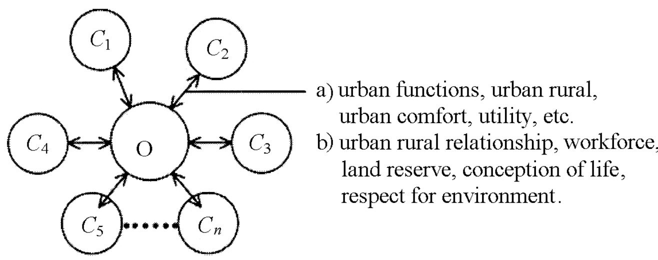
a) Education.
b) Level of culture.
c) Professional training.
d) Age.
e) Assimilation capability.
f) Adaptability.

It can be said that the adaptation speed of individual and socio-cultural consciousness is in rhythm disadvantaged *vs*. the phenomenological reality of function dynamics, models, methods, procedures and materials used in urban development, functional development, and spatial–volumetric development.

Another development direction of great importance for the urban phenomenon is given by the use of national resources. Among the top resources is space, territory, the necessary terrain of urban settlement expansion. Terrain occupation is no longer a problem of human developing, but more so of durable development in the respect for future generations, to the environment in which we live in, and our children.

Fig. 8 – Aereal view – Podu Roş.

A great aspect of city developing direction is the metropolization phenomenon (Fig. 9). This phenomenon represents urban development based on external administrative entities, thus cooperation and cohabitation in that system implies a biunivocal relationship.



O – city.
C – suburban settlement.
⟷ – interaction type a) and b).

Fig. 9 – Metropolisation phenomenon.

The main resource of urban space, besides terrain, is the labor force, and the benefits in that relationship being represented by (urban services)

a) utility;
b) transport;
c) education;
d) culture;
e) healthcare.

Besides all of the above, another dimension of great value, is the management and maintenance of green spaces as an integral part of urban structure.

The metropolization phenomenon – metropolitan areas – must consider a management adapted to urban–rural cohabitation, oriented towards feasible, realistic and beneficial objectives for both parties involved, namely

a) peri-urban rural restructuring in regard to resources and natural and human capital;

b) limitation of urban expansion to functions and utilities, urban services by preserving rural individuality and characteristics;

c) avoiding social and economic isolation of peripheral rural areas;

d) eco-systematic approach to management practice in the metropolitan area;

e) the concept of diversity as a vital operational principle.

These objectives are feasible taking into account a series of two-way action that will lead to

a) increasing functional integration;

b) metropolitan system integration with the regional, national, European system;

c) increasing the competitiveness of the system.

4. Conclusions

Finally, contemporary urban development requires an urban structure adjustment and its reorganization based on the analysis of relationships between urban system elements, realized by

a) The evolution in time of local urban relationship caused by the process of adaptation to urban space.

b) The processes of adapting existing urban relations to the needs requirements appeared during development.

c) Introducing the new urban context of new relationships required by contemporary evolution that are not contradictory but complementary in relation to existing relationships and socio-cultural and economic context of existing and budding.

Inevitably we are witnessing a process of fundamental transformation of urban structures, a process that requires knowledge, understanding, self-regulating and constant control.

"To change means knowing well the existing fund" (Octav Doicescu).

Changes, transformations, adaptations to urban body evolution and contemporary society competition doesn't have to do with sacrifice essential for efficiency and comfort of living. Urban development and the evolution of urban functions have to consider at least two aspects namely: a) the delivery of the social comfort and efficiency (the ratio between social community effort for investment and change and the sum of social qualities) and b) achieving economic efficiency through economic development (quantitative and qualitative) in relation with social and global investment effort.

The last to be followed in defining, knowledge and action on trends of urban evolution must be quality of life for city residents through planning, restructuring, implementation, monitoring and control of spatial and territorial effective development.

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EVOLUȚIA URBANĂ – DIRECȚII DE DEZVOLTARE URBANĂ ÎN PERIOADA DE TRANZIȚIE

(Rezumat)

Evoluția urbană post 1989 are o serie de caracteristici specifice mai ales în plan spațial, teritorial.

Determinarea principalilor factori de dezvoltare şi a direcțiilor de evoluție urbană (dimensiuni, ritmuri, nivel de expansiune, caracter centrifug axial, concentric, centripet, evoluție funcțională, tendințe şi implicații sociale etc.) reprezintă o necesitate şi o obligație de acțiune a profesioniştilor din domeniul urbanismului şi a amenajării teritoriului.

Această necesitate apare şi din perspectiva necesității realizării strategiilor planurilor documentațiilor şi studiilor de urbanism care să intervină corectiv în evoluția zonelor cu probleme structurale şi să orienteze evoluția urbană către scopul principal al acesteia şi anume creşterea calității vieții locuitorilor din aşezările umane.

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QUELQUES CONSIDÉRATIONS SUR L'ÉVOLUTION DES NORMES DE CALCUL DES POTEAUX AVEC LA SECTION MIXTE ACIER–BÉTON

PAR

N. CHIRA, CRISTINA CÂMPIAN et TRAIAN ONEŢ

Resumé. Depuis près d'un siècle, le système de construction basé sur des portiques en acier ou mixtes acier–béton est devenu l'un des types les plus utilisés dans le domaine du génie civil. Plusieurs générations d'ingénieurs se sont préoccupées du développement des méthodes de calcul et des technologies de fabrication relatives à ces structures. En vue d'un dimensionnement optimal des structures, les ingénieurs sont tenus de trouver un compromis entre les exigences structurales de résistance, rigidité et ductilité d'une part, et les objectifs d'utilisation et de fonction relevant d'exigences architecturales d'autre part. Cette article réalise une comparaison entre différents méthodes de dimensionnement des poteaux mixtes acier–béton, en tenant compte des plusieurs paramètres.

Mots clef: poteaux; section mixte acier–béton; codes.

1. Introduction

Dans le domaine des constructions de Génie Civil, les avantages d'un matériau relèvent de divers facteurs comme sa disponibilité, sa résistance structurale et sa mise en œuvre. Le choix du matériau dépend des critères esthétiques, ainsi que des coûts liés au type de construction. Il est très surprenant de constater qu'aucun matériau connu à l'état naturel ne possède pas toutes ces propriétés au niveau souhaitable. Le problème de l'ingénieur consiste donc à trouver une solution optimale dans l'utilisation de différents matériaux et méthodes de construction, avec l'objectif de réaliser une structure de coût minimal tout en satisfaisant aux exigences structurales de celle-ci. De nos jours, la majorité des constructions est réalisée en utilisant un des matériaux: béton ou acier.

La définition d'un poteau mixte est basée sur la contribution plus ou moins importante de l'acier à la résistance de l'élément, mais le mode de fonctionnement d'un poteau mixte ne peut se définir uniquement considérant le principe du béton armé qui impose qu'aucun glissement apparaisse à l'interface acier– béton ; en raison de l'élancement souvent plus important et la flexibilité plus grande d'un poteau mixte, son mode de fonctionnement s'apparente également à celui des poteaux métalliques, affecté par des imperfections géométriques et structurelles.

2. Paramètres qui caractérisent le comportement des poteaux

Il y a trois paramètres importants qui caractérisent le comportement d'un poteau, tant celui réalisé d'un seul matériau que celui mixte

a) sa rigidité initiale;
b) sa capacité de résistance ultime;
c) sa ductilité.

Dans ce qui suit on va s'efforcer de situer les approches des normalisations modernes vis-à-vis de ces paramètres.

2.1. La norme japonaise AIJ_{LSD}-90 [1]

Au Japon, les constructions utilisant des éléments à section mixte acier–béton sont fréquentes pour des bâtiments avec une grande ou une moyenne hauteurs. Pour les projets on utilise une méthode qui est basée sur la résistance du poteau à la charge sismique approprié, résistance obtenue en additionnant les capacités portantes des composants de la section transversale, soit acier et béton.

On considére une distribution de type plastique-parfait des contraintes, avec un facteur de réduction de la résistance du béton. Ce facteur a des valeurs différentes pour les sections remplies de béton et pour celles enrobées de béton.

La méthode est utilisable pour la situation où on a compression avec moment fléchissant simple ou bi-axial. D'habitude, les poteaux des portiques sont souvent massifs, vu que le projet considère des valeurs importantes des forces latérales. On utilise des coefficients d'amplification des moments pour prendre en compte la déformation des poteaux lorsque ceux-ci sont souples. De plus, la norme impose des valeurs maximales des excentricités de la charge axiale.

2.2. La norme britannique BS 5400 Part. 5:1989 [2]

La méthode de calcul développée dans cette norme est plutôt empirique, déduite d'une méthode déjà appliquée aux poteaux en acier. On utilise des coefficients partiels de calcul pour les sollicitations ainsi que pour les matériaux.

On considère également une distribution plastique des contraintes dans la section mixte, conduisant à une courbe d'interaction *M–N*. Pour prendre en compte dans les calculs les effets de longue durée dans le béton, des caractéristiques géométriques réduites sont proposées pour la partie en béton de la section, considérée non-fissurée. Comme dans la norme européenne Eurocode 4 [6], on tient compte, dans les poteaux remplis de béton, de l'augmentation de la résistance du béton en raison de son confinement.

La méthode est applicable seulement aux sections bi-symétriques dans un domaine limité, couvert par les courbes européennes de flambement. Les poteaux sont calculés en utilisant une formule de type parabolique pour la courbe $M(N)$, utilisable pour différents rapports des moments aux extrémités des poteaux et des valeurs de la charge axiale, pour une souplesse et un facteur de contribution du béton à la capacité totale de la section, donnés.

2.3. La norme américaine de American Concrete Institute ACI 318 [3]

Dans ce code on utilise la méthode des états limites ultimes et des facteurs de réduction des charges et des résistances. La capacité portante d'un élément mixte est calculée comme pour un élément en béton armé, en considérant que la ruine est atteinte pour une déformation limite de 0,3‰ dans les fibres du béton. Les effets de la souplesse sont analysés par l'augmentation de la valeur du moment en utilisant la charge critique d'Euler réduite (ramenée à l'effort normal plastique). Une rigidité réduite équivalente est considérée pour la partie en béton. En outre, on fixe des valeurs minimales des excentricités pour satisfaire les tolérances constructives.

2.4. La norme américaine AISC – LRFD de l' American Institute of Steel Construction [4]

La méthode utilisée dans ce code est celle du facteur de charge (LFRD), qui implique que les charges soient multipliées par un facteur de charge, L.F. (plus grand que 1,0) et les résistances nominales des éléments par un facteur de réduction, Φ (moindres que 1,0).

La méthode s'apparente à celle proposée par l'ACI, avec en fait peu de différences. La ruine est supposée atteinte dans le béton pour une déformation maximale de 0,03%. On utilise une courbe $M(N)$ de la forme montrée dans la Fig. 1, où M_n est la valeur de la capacité portante en flexion de l'élément (en absence de la force axiale), N_n – la valeur de la capacité portante en compression.

Une relation pour le facteur de reduction, Φ, est appliquée, afin d'obtenir la courbe de dimensionnement $M_d(N_d)$, avec des valeurs plus réduites de Φ pour les section dans lesquelles un confinement du béton apparaît .

On peut observer que cette méthode est assez conservative, car le facteur de réduction utilisé est le même que dans le cas des poteaux en béton armé; également, la même excentricité minimale est fixée pour les poteaux mixtes et pour les poteaux en béton armé.

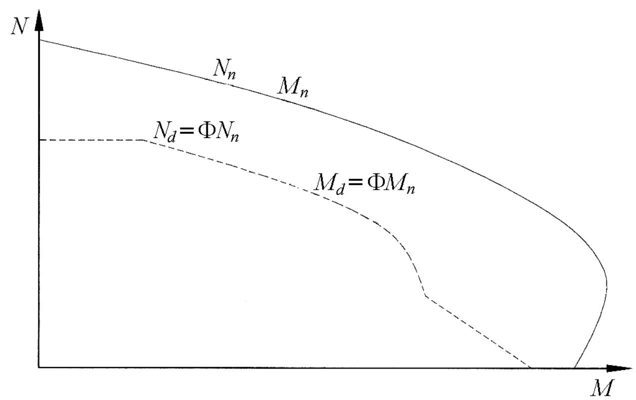

Fig. 1 – Forme de la courbe d'interaction, *M–N*, pour un poteau complètement enrobé en béton.

2.5. Le code Canadien CSA S161 [5]

Dans le code Canadien CSA S161 la capacité portante du poteau mixte est obtenue en additionnant la capacité portante pour la partie en béton et la capacité portante de la partie en acier, l'ensamble étant multiplié par un facteur de reduction, Φ.

On considère que pour une sollicitation au moment fléchissant et à la compression, le moment est repris intégralement par le composant en acier, qui est dimensionné en conséquence et avec l'effort axial dont on déduit la partie prise par le béton.

Un commentaire s'impose; on néglige la contribution du béton à la rigidité en flexion, *EI*, de l'élément, en se privant ainsi des possibilités offertes par le béton.

2.6. L'Eurocode 4 [7]

En Europe plusieurs méthodes ont été développées pour le calcul des poteaux mixtes. L'Université de Liège (CRIF) a effectué des recherches et a publié une méthode de calcul des poteaux mixtes enrobés en béton.

L'Université de Bochum a également participé à de nombreux recherches et le Comité international pour le développement et l'étude de la construction tubulaire (CIDECT) a proposé un document spécifique traitant des poteaux remplis de béton. Tous ces développements sont en fait assez proches de ceux proposes par l'Eurocode 4.

L'Eurocode 4 présente deux méthodes de dimensionnement des poteaux mixtes.

La première est une *Méthode Générale* qui impose de prendre explicitement en compte les effets de second ordre et les imperfections. Cette méthode peut être appliquée en principe à des sections de poteaux qui ne sont pas symétriques ainsi qu'à des poteaux de section variable sur leur hauteur. Elle nécessite l'emploi des procédés de calcul numériques et ne peut être envisagée que si l'on dispose de logiciels appropriés.

La seconde est une *Méthode Simplifiée* utilisant les imperfections géométriques appropriées et calibrées pour un calcul élastique équivalent au 2$^{\text{ème}}$ ordre géométrique local (ou éventuellement global pour le cas des structures souples). Cette méthode est en pratique limitée au calcul des poteaux mixtes présentant une section doublement symétrique et uniforme sur la hauteur. Les deux méthodes sont fondées sur les hypothèses classiques suivantes:

a) Il y a une interaction complète entre la section en acier et celle en béton, jusqu'à la ruine.

b) Les imperfections géométriques et structurales sont prises en compte dans le calcul.

c) Les sections droites restent planes lors de la déformation du poteau.

L'application de la seconde méthode comporte les limitations suivantes :

a) La section du poteau est constante et présente une double symétrie sur toute la hauteur du poteau.

b) La contribution relative de la section en acier à la résistance de calcul de la section complète, à savoir $\delta = \left(A_a f_y / \gamma_a\right) / N_{pl,Rd}$, est compris entre 0,2 et 0,9, où: δ est le rapport de la contribution du profilé en acier à la capacité portante de la section; A_a – l'aire de la section droite d'acier; f_y – la résistance élastique de l'acier du profile; γ_a – la résistance élastique de l'acier du profile; $N_{pl\,,\,Rd}$ – la résistance plastique des sections transversales en compression.

c) L'élancement réduit, $\overline{\lambda}$, du poteau mixte ne dépasse pas la valeur 2.

Pour les sections totalement enrobées, l'aire des armatures doit être égale au moins à 0,3% de l'aire en béton et les armatures présentent des épaisseurs d'enrobage satisfaisant des limites données.

La Fig. 2 présente, en bref, la comparaison des courbes d'interaction $M(N)$, suivant les normes précisées ci-dessous, pour une section mixte donnée.

Dans l'Eurocode 8 [8], une part importante des notions utiles au dimensionnement parasismique des structures en acier s'applique également aux structures mixtes acier–béton (les chapitres **6** et **7** sont d'ailleurs rédigés de manière relativement homologue).

Il convient aussi de préciser que dans l' EN 1998-1 des règles et dispositions constructives bien claires sont données. Par exemple, l'article 7.6.4. donne pour les poteaux mixtes en I ou H, partiellement ou entièrement enrobés en béton, une région critique de longueur L_{cr} spécifiée, pour répondre à la demande éventuelle de rotation plastique et compenser la perte de résistance due à l'éclatement du béton d'enrobage. Ces longueurs critiques sont définies tout comme pour les poteaux en béton armé (cf. 5.4.3.2.2). Aussi des valeurs de l'espacement minimal, s, des armatures et leur diamètre minimal, d_{bw} , sont précisés dans le paragraphe 7.6.4. pour les armatures de confinement qui doivent être placées sur la longueur des zones critiques.

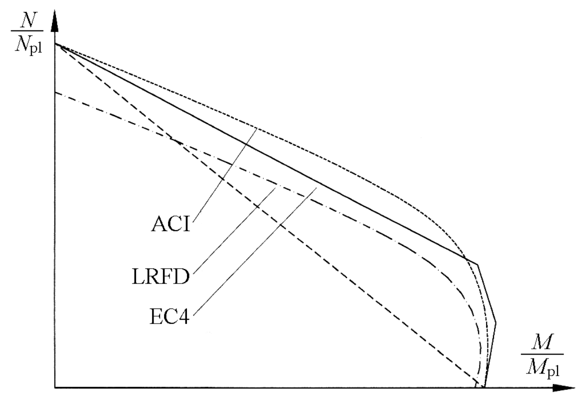

Fig. 2 – Comparaison entre les courbes d'interaction *M–N*.

Dans le cas des structures mixtes en portique, soumises à des actions sismiques répétées et alternées, il est difficile de proposer une localisation des zones fissurées. L'EC8 a considéré préférable de procéder avec des valeurs homogénéisées pour la rigidité flexionnelle (sur la longueur des éléments) (cf. 7.7.2(4))

(1) $$(EI)_{\mathrm{eq}} = 0,9(E_a I_a + E_s I_s + rE_{cm} I_c).$$

Sur la base de travaux normatifs récents lors de la rédaction de l'EN 1994-1-1, E_aI_a est la rigidité en flexion du profilé en acier, $E_{cm}I_c$ – la rigidité en flexion de la partie en béton en considérant que celui-ci n'est pas fissuré et E_sI_s – la rigidité en flexion de l'armature; r est un facteur de réduction qui dépend du type de poteau. On propose r = 0,5 pour les poteaux mixtes à profil creux remplis de béton armé, r = 0,4 pour les poteaux mixtes en profils I ou H partiellement enrobés de béton, et r = 0,3 pour les poteaux mixtes en profils I ou H totalement enrobés (conformément à la clause 6.7.3.1 (2) de l'EN 1994-1-1).

Enfin, dans certains cas (ossatures en acier relativement souples, voire ossatures mixtes), l'analyse élastique globale pour le calcul des effets doit être effectuée au 2$^{\text{ème}}$ ordre géométrique. Dans le cas où cette analyse au 2$^{\text{ème}}$ ordre est obligatoire au niveau global, voire au niveau local des poteaux en introduisant une imperfection géométrique adéquate (c'est le cas des poteaux mixtes, l'EN 1994-1-1 ne fournissant pas de formule de résistance élasto-plastique au flambement comme l'EN 1993-1-1), la vérification se ramène à une vérification de la résistance dans la section la plus critique (où l'amplification maximale des efforts due aux effets du 2ème ordre géométrique est maximale).

Pour la vérification de la stabilité au flambement, basée sur la combinaison la plus défavorable (N_{Ed} , M_{Ed}), les efforts sont majorés au sens du dimensionnement en capacité.

Par exemple, pour un poteau mixte en flexion uniaxiale on utilise la courbe d'interaction plastique (cf. 6.7.3.2 de l'EN 1994-1-1). Pour le niveau d'effort normal, N_{Ed} , le moment de calcul doit satisfaire la condition (Fig. 3)

$$M_{Ed} \leq 0.9\mu M_{pl,Rd}. \tag{2}$$

En flexion biaxiale on effectue des vérifications similaires dans chaque plan de flexion, plus une condition d'interaction entre les deux flexions (cf. 6.7.3.7 de l'EN 1994-1-1).

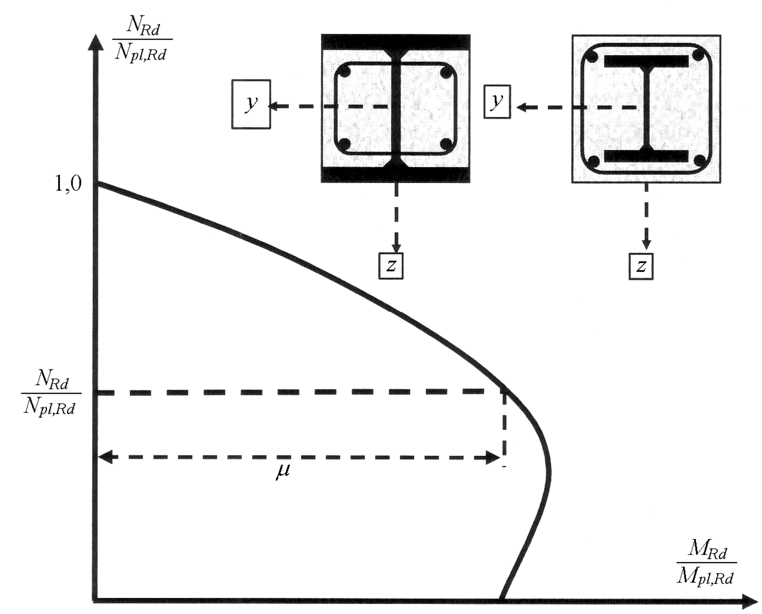

Fig. 3 – Courbe d'interaction plastique $M(N)$.

On impose des valeurs limite du rapport $N_{Ed}/N_{pl,Rd}$ et de $\overline{\lambda}$ et en fonction des valeurs limite du facteur de comportement, q, qui caractérisent la capacité dissipative de l'ensemble d'une structure. $\overline{\lambda}$ désigne l'élancement réduit dans le plan de flambement le plus défavorable (calculé avec la longueur d'épure du poteau)

$$\overline{\lambda} = \sqrt{\frac{N_{pl,Rd}}{N_{\mathrm{cr}}}}. \tag{3}$$

Aussi, pour les poteaux mixtes en I ou H, partiellement ou entièrement enrobés en béton, et pour les poteaux mixtes avec profil creux, des valeurs données sont utilisées pour les limites d'élancement des parois vis-à-vis du voilement local (cf. Tableau 7.3 de l'EC8), en fonction de la valeur de q et de la classe de section.

3. Quelques commentaires sur ces normes

3.1. L'origine de la méthode de calcul

En général, soit les ingénieurs specialists en acier ont considéré pour les constructions mixtes un meilleur calcul des constructions en acier, soit les ingénieurs spécialists en béton ont souhaité une résistance meilleure.

Dans le premier cas ils ont utilisé les courbes européennes de flambement pour l'acier. Dans le second cas, ils ont adopté les méthodes de calcul du béton.

Les méthodes ayant à la base l'acier ont été utilisées en AISC et BS. Le code ACI utilise une méthode de calcul en partant du béton armé. Le EC4 et le DIN (maintenaït totalement remplacé par EC4) utilise une combinaison des deux méthodes. Distinctivement, le code japonais utilise la méthode de surposition des résistances individuelles des parties de la section des matériaux différents.

3.2. Capacité portante et facteurs de charge

Les standards utilisent différents facteurs pour les charges ou pour les combinaisons des charges.

Les standards américains (ACI et AISC) utilisent des facteurs de reduction, Φ (ayant des valeurs de 0,7 et 0,85), par rapport à l'Europe où on utilise des coefficients partiels pour les matériaux. On utilise différentes valeurs pour les matériaux utilisées, d'habitude de 1,5 à 1 ou 1,1.

Le code japonais utilise la méthode de résistance admissible, avec la recommandation de vérifier toujours la capacité portante ultime pour la situation d'un séisme sévère possible.

3.3. Rigidité équivalente

Pour considérer la sveltesse il est nécessaire la calcul d'une rigidité équivalente (EI).

Différentes codes utilisent différentes approximations pour aboutir à la valeur de la rigidité du béton.

Le Tableau 1 présente les valeurs recommandées par les codes ci-dessous pour le module d'élasticité du béton.

Tableau 1

Standard	E_c	Observations
BS 5400	$450F_{cu}$	Valeur petite, on peut l'utiliser pour des sections pas fissurées
EC4	$600F_{cyl}$	On tient compte de la contraction du béton
ACI	$57,000(F_{cyl})^{1/2}$	Valeur grande si on utilise une section fissurée
AISC	$w(F_{cyl})^{1/2}$	Valeur grande si on utilise une section fissurée

3.4. Les courbes d'interaction

Le Tableau 2 présente une comparaison générale des différents codes. On constate de grandes différences pour aboutir a la capacité portante.

Tableau 2

Standard	Observations	
	Modalité de calcul	Résultat
AIJ	Distribution plastique (PD)	F_{cyl} réduite
BS 5400	Distribution plastique	Surface approximative
EC4, DIN 8806	Distribution plastique complète	Surface simplifiée de la courbe
ACI 318	Équilibre et compatibilité des tensions	La ruine par l'atteinte de 3% dans une fibre en béton
AISC - LRFD	Distribution plastique	Surface approximative

4. Conclusions

On peut évaluer la capacité portante comme une somme des capacités portantes des éléments composants. L'existence des différences peut être expliquée par, d'une part, les différences apparues dans la philosophie du calcul, et d'autre part, les quantifications numériques. La première couvre des considérations fondamentales, comme la distribution des tensions, tandis que la deuxième est la conséquence de l'utilisation des bases des données expérimentales spécifiques pour aboutir à des expressions analytiques actuelles.

Comme méthode générale de calcul on accepte le calcul semi probabilistique aux états limites.

Même si tous ces codes utilisent la même philosophie et les mêmes résultats expérimentaux, dans l'évaluation des propriétés finales d'une section, pour une charge donnée, ou la capacité portante pour une section prédéfinie, peuvent apparaitre certaines divergences. Celles-ci sont dues aux différences apparues dans la considération des facteurs de sécurité, des propriétés des matériaux, de la considération des charge à longue durée, etc.

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CÂTEVA CONSIDERAȚII ASUPRA EVOLUȚIEI NORMELOR DE CALCUL ALE STÂLPILOR AVÂND SECȚIUNEA MIXTĂ OȚEL–BETON

(Rezumat)

Normele de calcul aplicate în diferite ţări respectă recomandările normativelor pentru structurile din oțel şi cele pentru construcțiile din beton armat şi beton precomprimat în măsura compatibilității lor cu natura comportării specifice a unui element compus. O atenție deosebită este acordată aplicării principiilor generale de securitate luând în considerare structura compusă oțel–beton, cu posibilități specifice de atingere a stărilor limita.

În codurile de proiectare analizate in lucrare există diferite procedee de calcul referitoare la stâlpi. Un stâlp compus poate fi tratat cu anumite metode ca un stâlp metalic întărit cu beton, pe când alte metode îl pot considera ca un stâlp de beton cu o armatură specială. În încheiere se prezintă o comparație critică a principalilor parametri care influențează comportarea stâlpilor.