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IRREGULAR OLD STORIED MASONRY BUILDINGS. RISK EVALUATION AND STRENGHTENING SOLUTIONS

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SUMMARY

In the first part of the paper, one presents the general aspects regarding the general features of old storied masonry buildings, having irregular conformations, especially. Afterwards, a short review of the analysis procedures used in the risk assessment of these "non-classic" structures marks their efficiency degree. Finally, it presents an analytical FEM study on a representative model. A very irregular quasi-dual structure was been supposed at two successive simplifications: a. the erase of its the major tower-story; b. the split (divide) of the initial building in two different independent structures. The comparisons between result data lead to the quantification of the idealization errors.

In the second part of the paper, there present two applications with irregular old buildings. First, historic data and structure information (conformation, supported earthquakes and damages) are given. Second, there present the used earthquake assessment analyses with their main conclusions and recommended strengthening interventions. The analyses tacked into account the findings of the FEM study and the proposed interventions do not violate the Venice Chart provisions for historical monument restorations. The two representative old buildings presented are "Old Custom House" (build in 1872 year, four levels, historical monument) from Botosani city and "Nicolina International Railway Station" (build in 1954-1955 years, 4 levels, architectural monument) from Iasi city.

The final chapter contains the general conclusions. These synthesis data proceed from the previous chapters and from other scientific studies and references.

INTRODUCTION

Often, the old masonry buildings have irregular conformations and their major cracks eliminate the continuity of the materials. Another general feature is the absence of the horizontal diaphragms, (the wood floors working as weak links between walls). The non-symmetric plan conformation, the partial retreating of a story, local weakening or strengthening are other features of the old buildings. Of course, the damage assessment procedures for the newer structures are improperly in this case.

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The objectives of the paper are the following: a. to present adequate procedures for the seismic risk evaluation of the specified old irregular structures; b. to illustrate some strengthening solutions, able to respect the requirements of the Venice Chart (for historical and architectural monuments).

STUDY OF A GENERAL IRREGULAR STRUCTURE MODEL

Criteria for the selection of models for general irregular old structures

In analysis practice, many structural engineers apply various simplifications. Sometimes, these assumptions can lead to the big errors. It must be establish some rules for the usual seismic risk assessment of the old irregular structures. One may simplify the analysis procedure, but only with the final reliable results. Thus, it was necessary to find the limitation of the model approximations. Studies in this direction may lead to the desired rules for simplified analyses.

The examination of the old historic or architectural buildings shown for many cases the significant the absence of the symmetric plan conformation. Other irregularities of these structures result from their vertical configuration. Some of these “anti-engineering” particularities are partial retreating of top stories (often having non-symmetric plan position), presence of (heavy) towers, local absence of floors, variation of the “base” level number (caused by the high slope of terrain) etc.

The main characteristic of the irregular old structures seams to be the pregnant non-symmetric distribution of the stiffness at earthquake actions. The second one could be the high differences between the pressures under foundations. Of course, the question of the important non-symmetric distribution of the stiffness becomes a true difficulty for structural engineers in the seismic risk assessment.

Therefore, the representative model for old structures must be one with significant horizontal and vertical stiffness irregularities. This is the first criteria for the selection of representative models. If the floors were been build from wood or iron beams with masonry vaults, the position of the longitudinal axes of girders will be the second criteria can be considered.

Selected model and various idealizations for seismic assessment analysis

The author of this paper conceived a model structure represented in the Fig. 1. This building has the next features:

- The plan is strongly asymmetric.
- The main vertical strength members at seismic actions are the structural (weak) masonry walls.
- The columns and girders (in a network 400 cm x 400 cm) are from RC.
- The RC floors were been considered flexible (in their plan), because their reduced thickness (10 cm only).
- The complex object can be separate into two corps: one (the biggest) with two stories, another (the smallest in plan, but higher) with three stories.
- Each of the two corps has an anti-symmetric position of vertical members.

The idealizations were been based on the FEM (Finite Element Analysis) in the study. The reason of this selection derived from by the truism “The FEM procedure is the most closed to the exact solution”. The columns and girders were been represented by FRAME elements. The masonry walls and the RC plats were been represented by SHELL elements.

For masonry SHELL elements, the excess of the non-positive stresses S11 ($|S11/t| > R_{c,}$) leads to the crush of the compressed zone, the excess of the stresses S12 ($|S12/t| > R_{sh}$) leads to the slip cracking and the excess of the stresses SMAX ($|SMAX/t| > R_{dt}$) leads to the diagonal cracking ultimate mechanism. Notations: R_c – ultimate compression resistance of the masonry; R_{sh} – ultimate shear resistance of the masonry horizontal mortar joint; R_{dt} – ultimate diagonal tension resistance of the masonry; t – wall thickness.

The following models were been considered:

- Entire structure (without simplifications), named “ALL STRUCTURE” (ALL), Fig. 2
- Structure without the third level of the high zone, named “WITHOUT TOWER” (WT), Fig. 3
- The two parts decoupled in “SHORT PART” and “HIGH PART” (DEC), Fig. 4.

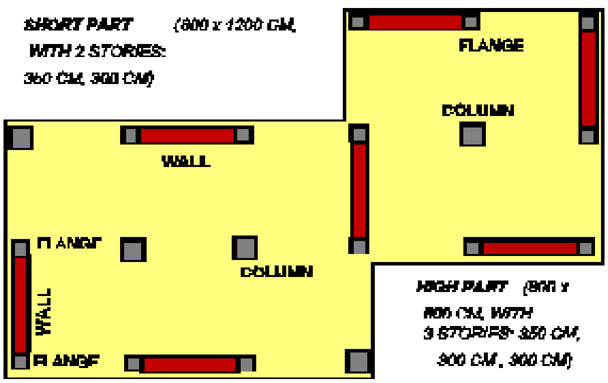


Figure 1. Floor plan of the "ALL” structure

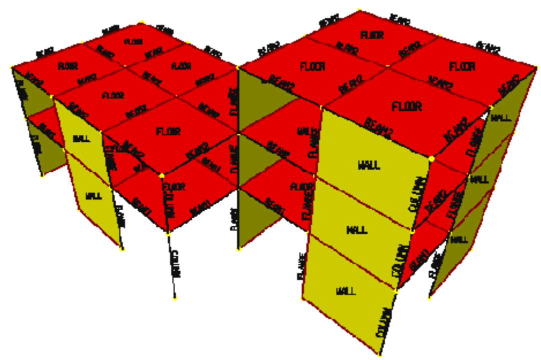


Figure 2. Idealization of the "ALL” structure

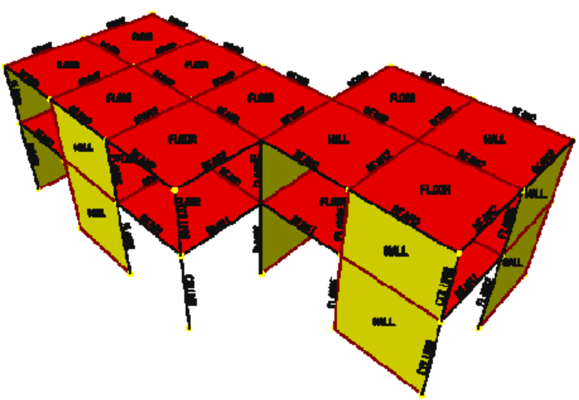


Figure 3. Idealization of the "WT" structure

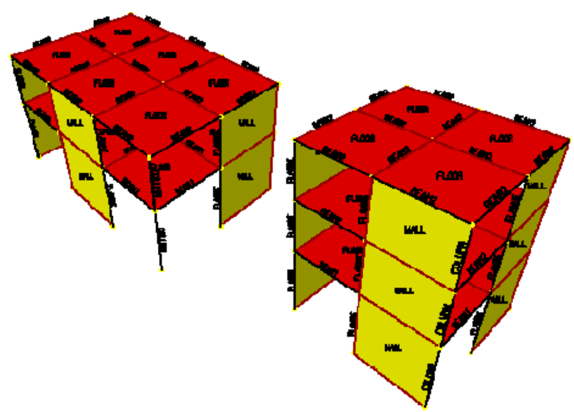


Figure 4. Idealization of the "DEC" structures

The earthquake spectra method (using values resulted from the Romanian seismic code [1]) applied in the earthquake analysis, for the compatibility with the results, ones obtained using the usual methods:

$$E_0 = S_d / g\ G \tag{1}$$

$$c = S_d / g = \alpha\ k_s\ \beta\ \psi\ \varepsilon \tag{2}$$

where: E_0 – earthquake shear base force; S_d – design acceleration spectra; G – total weight of the structure; α – factor of the importance; k_s – seismic coefficient of the zone (= PGA / g); β – dynamic factor; ψ – ductility factor (=1/K; K – behavior factor of the structural type); ε – equivalence factor with a single degree oscillator).

Results of FEM analyses

Here, one selected only the main results, regarding the influence of the each simplification over the seismic response. One reminds these studied simplifications:

- Cutting the "tower" (the 3rd level over high substructure), resulting WT model
- Decoupling initial structure (ALL), resulting DEC structures. The "new" models of the independent structures are SHORT PART and HIGH PART.

The retained results are:

- Decrease percent of the earthquake base shear
- Value changes of the modal participation factors.

Effects caused by tower cutting

The main processed result data regarding the elimination of the third partial level of the initial structure are presented in the Fig. 5 and 6.

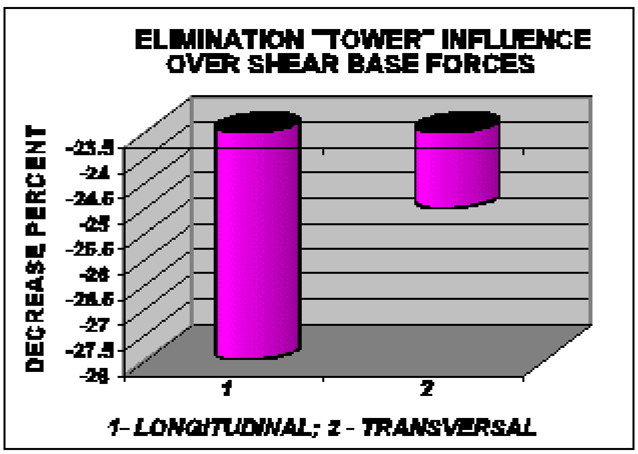


Figure 5. Base shear reduction by WT

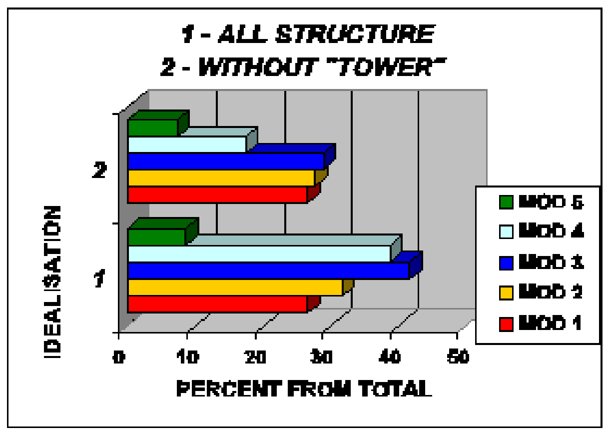
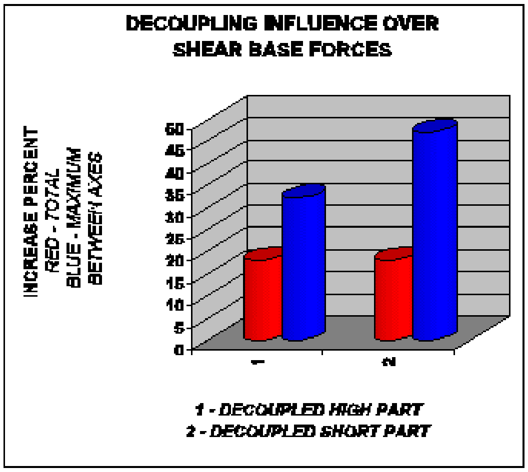


Figure 6. Modal participation factor changes by WT

The first effect was the major reduction of the earthquake base shear: with 25.4 % for transversal direction and with 28 % for longitudinal direction. This means the possibility of the neglecting of the very important parts of the total efforts in structural members, if one will cut the heavy tower in the analysis model.

On the other hand, the heavy and rigid tower cutting will produce the strongly redistributions of the modal oscillations modes. These alterations come from the torsion components. In this study, one been resulted the higher variations of the modal participation factors, especially for the transversal direction of the earthquake actions. The most affected were the oscillation modes 2, 3 and 4. These effect, supposed over the shear base reduction, can do it to the inadmissible errors in the earthquake capacity evaluation of the irregular structures.

Effects caused by the decoupling structure in independent structures
The main processed results regarding the total separation of the initial (ALL) structure into two independent ones (SHORT PART and HIGH PART) are given in the Fig. 7 and 8.



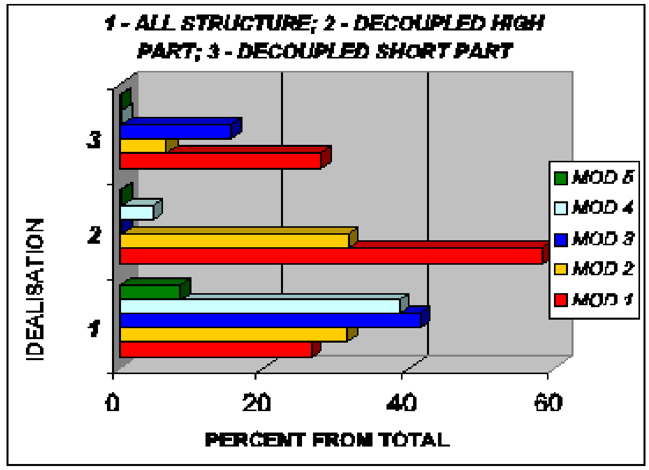


Figure 7. Base shear increases by DEC

Figure 8. Modal participation factors changes by DEC

The first major effect of the decoupling is the considerable increase of the earthquake base shear: till 200 % for HIGH PART substructure and until 300 % for SHORT PART substructure. The errors are too higher for the acceptation the decoupling simplification for irregular structures.

The second major effect consists in the strong redistributions of the modal oscillations modes. These alterations come from the reduction of the torsion components. In the studied variants, the higher modifications were been at the longitudinal seismic actions. The higher increase of the first modal participation factor was for the HIGH PART substructure, while for the SHORT PART substructure one remarked an important reduction of the participation factors in the modes 2, 3 and 4. These complex effect, supposed over the strongly shear base amplification, also can do it to the inadmissible errors in the earthquake capacity evaluation of the irregular structures.

Another procedures for seismic assessment of old masonry structures
Engineers apply the simplified procedures in the current cases. These methods were been named the "practical procedures" for masonry structure analysis. A short review of the most common methods is necessary, in this context. It must establish their limitations and the needed corrections to be in accord with the "exact" solutions. The presentation begin with the complex methods and ends with the most simple one. Of course, one does not present all variants, here.

Frame assimilation methods
Often, the walls structures were been assimilated with the frame ones. The pilasters are the columns and lintels (or masonry arches) became equivalent beams. The floors are considered as horizontal diaphragms, even these members are build from masonry (domes or vaults) or from wood. Usual, earthquake analyses are been leaded on the planar frames.

The main shortcoming is the equivalence of the planar or three-dimensional members (as masonry pillars) with columns, because their stress distribution and break mechanism may be different. In addition, the "diaphragm" hypothesis of the wood walls or masonry vaults doesn't respected.

In these conditions, the decomposition in plan substructures of the irregular old buildings will lead to the results affected of the large errors.

Masonry pillars assimilated with the quasi-rigid member methods
The design codes (as Romanian ones [1] and [2]) accept the decomposition of the masonry structures in vertical linear or planar members (columns, walls). Each of these “k” resistant members may be destroyed at the minimum value of the associated shear forces with ultimate sectional efforts or stresses (M and N for eccentric compression = ec, Q for slip in horizontal mortar joints = slip, SMAX stress from diagonal tension = dt):

$$Q_{avaiable,k} = \text{Min}\ [Q_{ec,k}; Q_{slip,k}; Q_{dt,k}] \quad (3)$$

For the entire structure, one be summed the individual values tacking into account the effects of the general torsion and of the flexibility of the floors, using some correction factors.

Some specialists developed variants in which the vertical members can be effective three-dimensional blocks, [3], [4] and [5]. Similar relationships with the general formula (3) are used in these variants. In addition, there are necessary the mechanical equilibrium of the solid bodies:

$$M_s \geq M_o \quad (4)$$

$$F \geq H \quad (5)$$

in which: M_s – stability moment; M_o – overturning moment; F – friction force; H_p – horizontal slip force.

Details about an optimized variant of these methods can find in the work [4].

The main disadvantage of these procedures is the necessity of the structures with vertical regularity and without partial floors. Otherwise, the influence of the superior modal components will be too higher and the final analysis results may be incorrect.

Approximate methods
These procedures consider only a simple summation of the estimated values for ultimate earthquake loading capacity (shear) of the vertical masonry members. On principle, the equations are:

$$Q_k = R_{sh}\ A_k \quad (6)$$

$$Q_{str} = \sum Q_k \quad (7)$$

where: Q_k – shear capacity of “k” member; R_{sh} – medium values of the ultimate shear resistance; A_k – transversal area of “k” member; Q_{str} – total level shear of the structure; Q_k – available shear of “k” resistant vertical member.

The procedures with a high degree of the approximation omit almost all significant factors that disturb the “linear” behavior of the structure at earthquake actions. In these conditions, the use of the correction factors can become games of hazard.

For all methods the general formula of the final verification of each of plan directions is:

$$R_{p,L,(T)} = Q_{p,av,str,L(T)} / Q_{p,earthquke} \geq R_{req} \Rightarrow \text{OK !} \quad (8)$$

where: $R_{p,L,(T)}$ – parameter of the "earthquake safety"; $Q_{p,av,str,L(T)}$ – available shear force; $Q_{p,earthquake}$ – required earthquake shear force; R_{req} – control factor (< 1.0) given in design codes; p – number level; L – longitudinal earthquake direction; T - transversal earthquake direction.

Practical conclusions for irregular old structure seismic assessment methods

The manner of some specialists to decouple the structures with complicated and irregular forms in many independent ones (for seismic risk assessment) is totaling wrong. However, by exception, if the substructures don't have high stiffness differences, one may be separated. But, it must be corrected the values of the earthquake base shears and the distribution at the levels (for compensation the increase of the higher oscillation modes).

An effect comparable with the structure separation is the presence of the non-rigid floors (diaphragms). To compensate this fact, one must increase the weight of the higher modes for the "slender" pier rows.

Other simplification in the structural idealization can be the cutting of the significant appendixes (towers). This operation can be avoid only for small towers and it must be accompanied by some corrections. These modifications of the analysis parameters must be the increase of the earthquake base shear, general torsion weight and higher mode components (for middle levels).

OLD CUSTOM HOUSE FROM BOTOSANI CITY

Historic data

Botosani is an important city from NE of Romania, being the administrative centre of the same named district ("Judetul Botosani"). The position of the Botosani district is near the North-Eastern Romanian actual border, being included in earthquake "E" zone (PGA = 120 cm/s^2, for Vrancea epicenter).

Old Custom House (OCH) represents one of the most representative buildings from Botosani city, Fig. 9 and 10. This object was been registered as an architectural and historic monument, having a regional importance. The railway station of the city, having similar architectural characteristics, was build simultaneously, in 1872 year. The both structures supported three major earthquakes (1940, 1977 and 1986) at least.

Structure description

The main members of the OCH are the exterior massive masonry structural walls (50 cm thickness). The interior masonry walls have only 30 cm, but all vertical members include weak bricks (C50) and lime mortar (M10), Fig. 11 - 14. The floors include wood components (beams, planking and ceiling) and some iron beams (for "large" spans, especially).

The structure has two complete stories over the quota +/- 0.00 ("1st" and "2nd"), two incomplete stories under this quota ("1st under" and "2nd under") and one partial level over the second complete one ("3rd"). The last level means the "appendix" of the staircase. This very irregular configuration resulted from the very inclination of the location.

The main deficiency of the structure is the non-verticality of many interior structural walls. Another disadvantage consists in the eccentric disposal of the high staircase "tower" (the 3rd story over the quota +/-0.00). This position favors the general torsion effect at earthquake actions.

Figure 9. Old Custom House (OTH). Main façade

Figure 10. OTH. Staircase tower detail

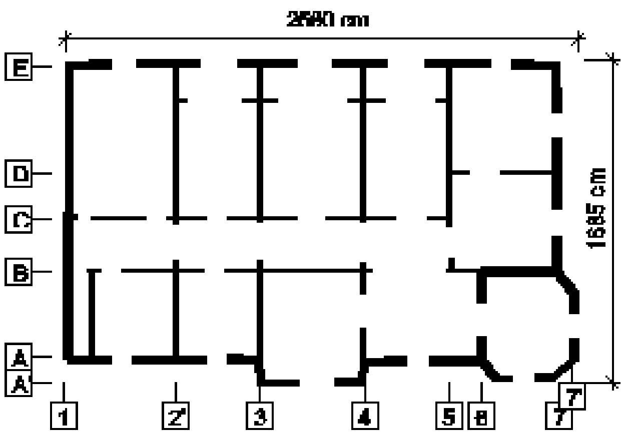


Figure 11. OTH. Ground floor plan

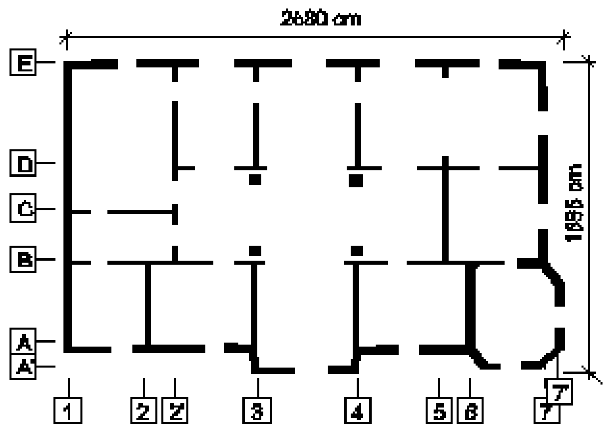


Figure 12. OTH. The 2nd main floor plan

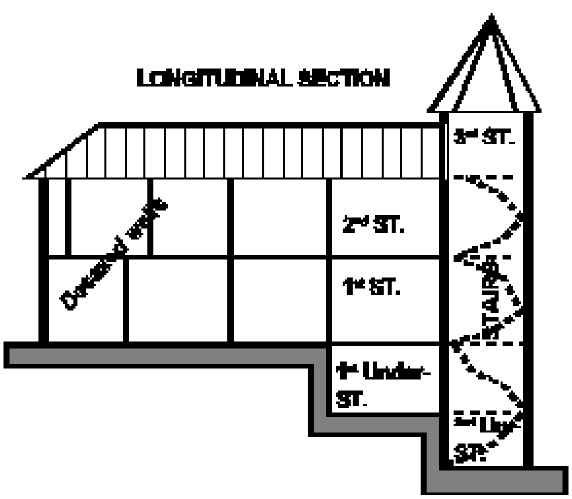


Figure 13. OTH. Longitudinal section

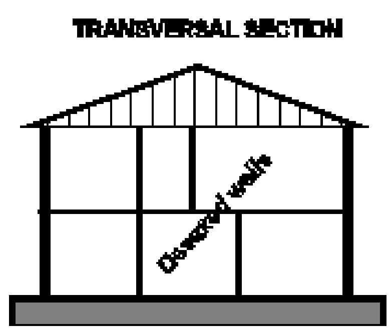


Figure 14. OTH. Transversal section

Adopted analysis procedures
The expertise investigations of OCH structure were been conducted by the author of this paper. The FEM idealization is too laborious for a short time to prepare an expert's rapport. On the other hand, the walls of the first and second "under"-stories are quasi-solids (being 80 cm thickness) and do not present structural degradations.

Consequently, one decided to use a method based on the assimilation of the masonry pillars with the quasi-rigid members over the quota +/-0.00. The values of the required earthquake forces were been determined with a majored ductility factor ψ (reduced behavior factor K), after relationships given in [4]. These corrections were necessary because the building supported three major seismic movements. Sure, the structure have some damages and its initial inelastic deformation capacities were been diminished.

Each level was been separately considered. The main level was the ground floor (1^{st}). All superior masses were been transmitted to the structural walls of this base level. For the second level (2^{nd}), the des-axed walls do not were been considered (as vertical structural members) in earthquake analysis.

The important third level (staircase tower) was eliminated in the assessment analysis, but it was compensated by the amplification of the earthquake base force (with 20 % transversal; 15 % longitudinal) and of the modal participation factors for second and third oscillation modes (with 5 % and 15 %, respectively).

The presence of the non-diaphragm floors was been compensated by the using of the correction factors, (0.7 for earthquake action normal on the girders, 0.6 for parallel). These quantifications resulted from previous studies of the author regarding the flexibility of the wood floors.

Main results of the seismic risk assessment
The structural analyses leaded to the values $R_{0,L} = 0.48$ and $R_{0,T} = 0.52 < R_{req} = 0.60$. This means a majored sensibility at seismic actions of the old structure. This feature represents a first trouble for structural safety. The general strengthening is necessary.

The examination in situ showed the strong degradation (corrosion) of the main iron floor beams. Finally, the section reduction leaded to the fissures in some transversal interior walls of the second level. This is the second trouble for structural safety. The local strengthening is necessary.

Proposals for the structure strengthening
The general strengthening interventions will increase the values of factors R (over the value 0.8) and the local strengthening interventions will eliminate the local deficiencies of the OCH structure.

After restoration, "Old Custom House" will have a new destination: a tourist's hotel. The strengthening, proposed by the paper's author, will keep the original exterior aspects.

General interventions
It was been proposed the following measures:
- Injections with cement past of the fissures
- Jacketing on a single face with reinforced cement-mortar of the interior thin walls (axes 2 – 6 and B – D); first floor: thickness jackets = 8 cm and steel bars = $\Phi10/100 \times \Phi10/100$ mm/mm; second floor: thickness jackets = 6 cm and steel bars = $\Phi6/100 \times \Phi6/100$ mm/mm
- Introduction of the two RC portal-frames at the first level (axes 3/B-C and 4/B-C).

Local interventions
It was been proposed:

- Strengthening of the used iron beams (with added steel plats or by transforming in RC elements)
- Local strengthening of wood second floors, too flexible, by stuffing new wood beams
- Changing of the wood degraded lintels with the RC ones.

NICOLINA INTERNATIONAL RAILWAY STATION FROM IASI CITY

Historic data

Iasi is the second city from Romania, after Bucharest. This city was the capital of the medieval stat of Moldavia about 250 years. The position of the Iasi district is near the Eastern Romanian border, being included in earthquake "C" zone (PGA = 200 cm/s^2, for Vrancea epicenter).

Nicolina International Railway Station (NIRS) represents one of the important "ideological" buildings from Iasi, Fig. 15 and 16. This monumental structure was build between 1954 - 1955 years, after the architectural style preferred by the horrible dictator Stalin in the Soviet Union. Of course, this architecture manner was been applied in the other communist countries. NIRS was been registered as an architectural monument. This massive structure supported two major earthquakes (1977 and 1986).

Structure description

NIRS has an approximate dual structure, Fig. 17 - 20. The vertical members are weak confined masonry walls and rare RC interior columns. Some of the transversal walls were been framed with cantilever truss RC systems. The horizontal members are the RC girders and plates. The RC walls and thick ground floor of infrastructure represent a true rigid foundation box.

Over the quota +/- 0.00, the central (A) corpus has three stories and the lateral corpuses (B and C) have only two stories. The floor of the quota +4.00 interrupts between axes 11 and 14, (in corpus A).

It can remark two major earthquake disadvantageous structural conformations. First: In horizontal plan, the general torsion is important, because of the different position of the torsion and weight centers. Second: In vertical longitudinal plan, the retreat of the corps A and C leads to the increase of the higher oscillation modes, especially for central corpus A.

Figure 15. Nicolina Railway Station (NIRS). Main façade

Figure 16. Nicolina Railway Station (NRS). Posterior façade

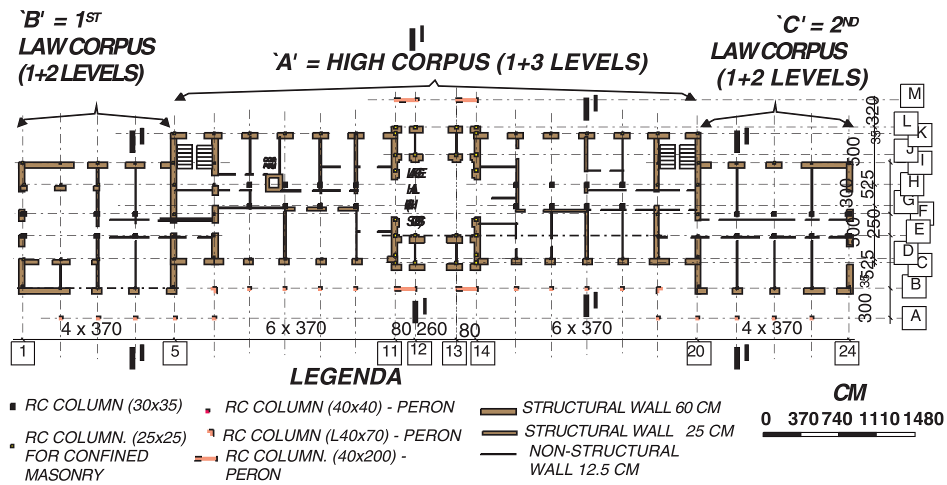


Figure 17. NIRS. Ground floor plan

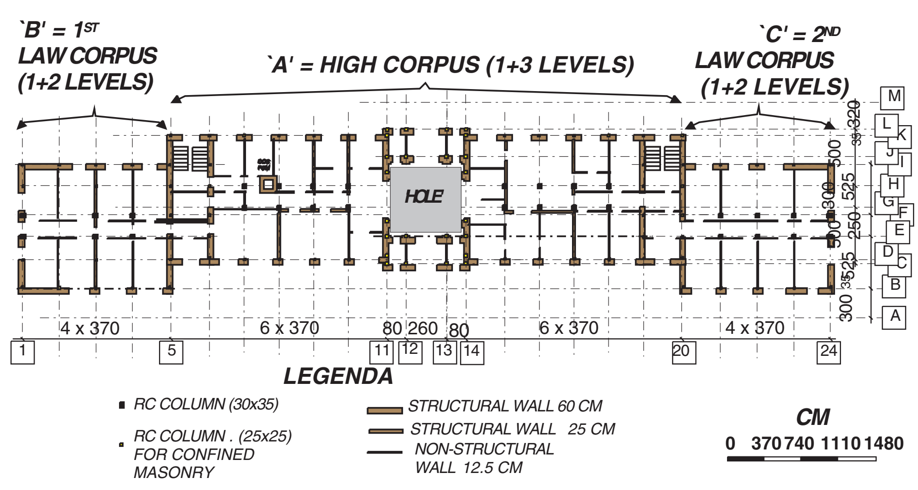


Figure 18. NIRS. The second floor plan

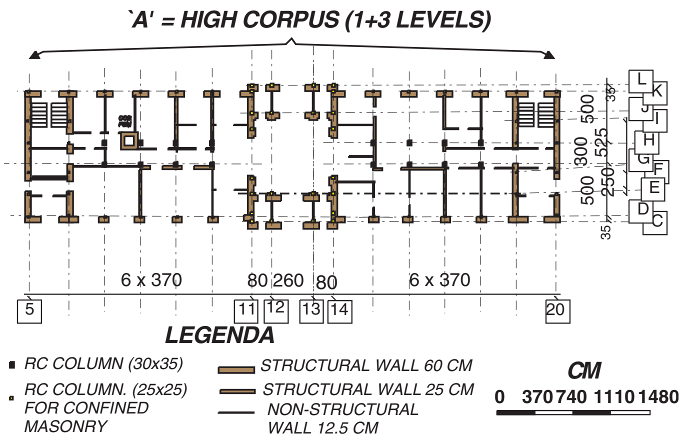


Figure 19. NIRS. The 3rd main floor plan

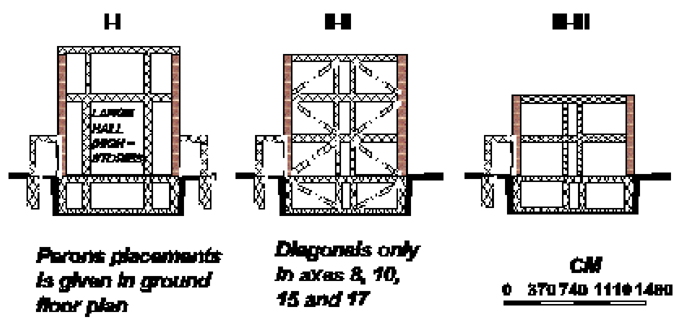


Figure 20. NIRS. Transversal sections

Adopted analysis procedures

The expertise investigations of NIRS structure were been conducted by the author of this paper. The large extension of the structure makes very laboriously its FEM idealization. On the other hand, the buried part (infrastructure) is a quasi-rigid component, with insensible earthquake oscillations amplitude.

Consequently, one decided to use of the methods based on: (a) the assimilation of the masonry pillars with the quasi-rigid members (including small RC columns from the confining of these subsystems) in cooperation with the RC frames (interior and exterior at the perrons from the ground level); (b) the spatial frames (with infill masonry panels) assimilation. Of course, one retained only (super)-structure (parts over the quota +/-0.00).

In the method (a), each level was been separately considered. The main level was the ground floor (1st). All superior masses were been transmitted to the structural walls of this base level. The important presence of the third level (at the corpus A), was compensated by the amplification of the earthquake base force (with 10 % transversal; 5 % longitudinal) and of the modal participation factors for second and third oscillation modes, (3 % and 8 %, respectively). The presence of the great hole in a central floor was been compensated by the using of the correction factors, (0.95 for earthquake action). The design earthquake spectra, computed with relations (1) and (2), was been corrected to tack into account of the two major seismic movements supported by the building. One computed the correction factor values with the relations given in [4].

In the method (b), the simplifications were minor. In the vertical plan frames with diagonal RC bars, these additional members were been idealized by the amplification of the infill masonry panels. The values of the required earthquake forces were been determined with a majored ductility factor ψ (reduced behavior factor K), after relationships given in [4]. This correction was necessary because of the two major seismic movement supported by the structure.

Seismic risk assessment. Main results

The structural analyses leaded to the following values of the parameter of the earthquake safety:

- $R_{0,L} = 0.86$ and $R_{0,T} = 1.49 >> R_{req} = 0.70$, using the method (a)
- $R_{0,L} = 0.91$ and $R_{0,T} = 1.54 >> R_{req} = 0.70$, using the method (b).

The final values (R parameters obtained with two procedures) differ with 3 – 5 %. The closed results show the validity of the method (b). The both variants lead the same conclusion: The structure has a reduced sensibility at seismic actions. Of course, by this point of view, the general strengthening is not necessary.

However, the examination in situ showed some degraded infill masonry (in axes 8, 10, 15 and 17) at the ground floor. The local easy strengthening interventions are necessary, even the general safety is good.

Proposals for the structure strengthening

The rehabilitation of NIRS structure is in an advanced phase. The retrofitting includes the solutions of the author. There are been proposed only some local strengthening interventions:

- Injections with cement past of the mentioned fissures
- Soft jacketing on a single face with reinforced cement-mortar of the same degraded walls (axes 2 – 6 and B – D); thickness jackets = 4 cm and steel bars = $\Phi4/100 \times \Phi4/100$ mm/mm.

CONCLUSIONS

The seismic risk assessment analyses of the old masonry structures often present a high degree of the difficulty. The main causes are the following:

- The initial inclusion of the building materials with a significant initial dispersion of the mechanical characteristics
- The uncontrolled modifications of the mechanical characteristics caused by the ageing phenomenon, the exposed faces being the most affected
- The presence of the discontinuities (fissures, even cracks), often invisibles
- The great irregularities of the horizontal or/and vertical conformation.

All these facts make very uncertain the exact computing idealizations.

The risk estimation procedures need use many approximations in these conditions. The statistic media of the mechanical properties may be one of these approximations. Another approximate input data may be the mediate dimensions of the structural members (transversal sections for each member, heights and thickness for each member etc.).

Nevertheless, there are unacceptable the incorrect data results. The wrong schematizations of the masonry old buildings will lead to the undesirable final, certainly. The presented studies emphasized the limits of the most used simplifications for irregular masonry structures:

- The elimination of the important towers, without the use of the adequate corrections, can lead to a large underestimation of the earthquake demands (forces, stresses).
- The separation in independent substructures with significant differences between their stiffness matrixes will lead to the uncontrollable errors.

The better analysis procedure could be one based on the most realistic idealization, where all members (with their properties), restraints, releases and loads are correct tacked into account. The computing programs based on the Finite Element Methods (FEM) or Distinct Element Method (DEM) can be such procedures. Unfortunately, the use of these procedures is prohibitive, in usual practice. In these conditions, one must use other procedures.

The methods based on the assimilation of masonry pillars with the quasi-rigid members may give good results, if the engineers will respect the main features of the real structures.

Through the assimilation of the masonry structures with the (infill) frame ones, reliable results will be obtained, only in the case of the slender pillars (similar with the columns).

The strengthening works must keep the original aspect of the historic and architectural monuments, in accord with Venice Chart provisions. The author proposed intervention solutions for the two such monuments (Custom Old House and Nicolina International Railway Station), which respect their initial architectural aspects. The interior reinforced jackets and injections are masked and the main building material will remain the original brick masonry.

REFERENCES

1. P100-92. "Earthquake protection design code for the civil, industrial and agrarian-zoo-technical buildings (in Romanian)". Bucharest: INCERC, 1992.
2. P2-85. "Masonry buildings design code (in Romanian)". Bucharest: INCERC, 1985.
3. Cismigiu A, Cismigiu M. "The seismic pathology of religion-related buildings in Romania and their treatments according to article 10 of Venice Chart". Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Paper no. 1216. Oxford: Elsevier Science Ltd., 1996.
4. Olaru D. "RC and masonry structures. Seismic risk evaluations and RC jacketed strengthening design procedures". Proceedings of the 11th European Conference on Earthquake Engineering, Paris, France, Paper no. 901. Rotterdam: Balkema, 1998.
5. Agavriloaie L, Olaru D. "Particularities of the seismic analysis of the historical structures. Parametrical studies about specific strengthening methods". Proceedings of the 12th European Conference on Earthquake Engineering, London, United Kingdome, Paper no. 701. Oxford: Elsevier Science Ltd., 2002