5th International Conference
″Computational Mechanics and Virtual Engineering ″
COMEC 2013
24- 25 October 2013, Braşov, Romania

NONLINEAR DYNAMIC ANALYSIS FOR „OD" REPETITIVE BLOCK OF FLATS DESIGN PROJECTS

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Abstract: *The block of flats stock erected between 1963 and 1976 consist of a large palette of functional schemes and constructive solutions mainly resulted from the architectural and urbanity conditions. In that period a great accent were put onto "repetitive design projects" which mean almost 90% of the existing apartment stock. From all these collective buildings more than 60% are represented by cast-in-place RC structural walls structural system, then 28% are represented by large pre-cast RC panels structural systems and about 9% for the RC frame structural system. The foremost parameters of the applied constructive systems in the period of P13 aseismic design code are: layout spans and RC structural elements cross section; total weight of the building; base shear force; RC structural walls shear area; compressive centric axial forces in case of RC frame structural systems; minimum percent for the reinforce area; fundamental periods of vibration and mass participation factors.*
Keywords: *nonlinear, dynamic, analysis, ductility, earthquakes*

1. CHARACTERIZATION OF P13 STRUCTURES

The principal applied structural system for multistory buildings used in that period where:

- Large pre-cast RC panels – for 8-9 levels buildings;
- RC frame system with cast-in-place columns, cast-in-place or pre-cast beams and pre-cast slab panels – for 7-15 levels buildings;
- Cast-in-place RC structural walls – for 7-11 levels buildings;
- RC central core and cast-in-place RC columns with cast-in-place or pre-cast beams and slabs – for 11 levels buildings;
- Soft and weak level structures (especially the 1st floor from the commercial reasons) – for 5-11 levels buildings.

From all these collective buildings more than 60% are represented by cast-in-place RC structural walls structural system, then 28% are represented by large pre-cast RC panels structural systems and about 9% for the RC frame structural system.
The foremost parameters of the applied constructive systems in the period of P13 aseismic design code are:

- layout spans and RC structural elements cross section;
- total weight of the building;
- base shear force;
- RC structural walls shear area;
- compressive centric axial forces in case of RC frame structural systems;
- minimum percent for the reinforce area;
- fundamental periods of vibration and mass participation factors. [1], [2], [3]

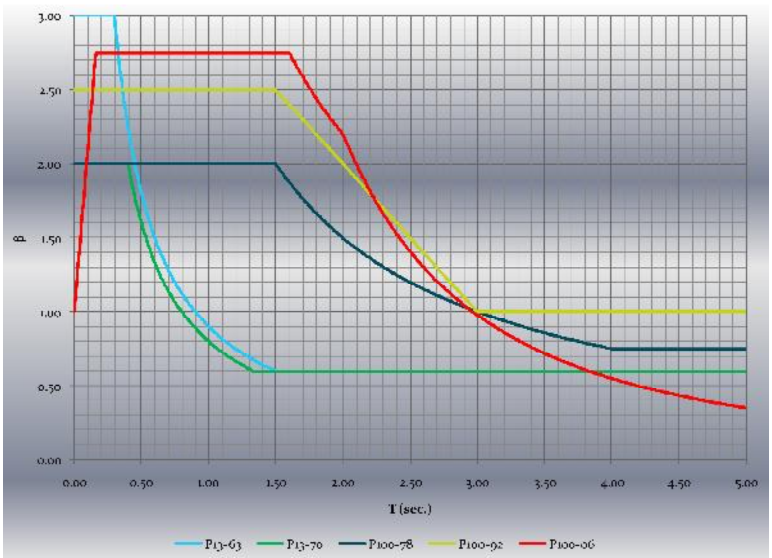
2. BALANCE ANALYSIS BETWEEN P13-63, P13-70, P100-92, P100/1-2006 AND P100/3-2008 ROMANIAN ASEISMIC DESIGN CODES

P13-63, the first Romanian Aseismic Design Code, has in the background the conventional seismic forces like in all the other countries.

The post elastic dynamic analysis and any concepts or principles concerning this aspect were not included in this code and this was a one of the major deficiencies of this code.
The major earthquakes from 1977, 1986, 1990, 2004 shows large incursions in the post elastic domain including first damages and cracks in the nonstructural elements (because of the admissible deflections and of the seismic energy dissipation) and secondary in the structural elements (beams, spandrels, columns, RC structural walls and foundations) because of the inadequate structural system and elements conformation .
In the P13-71 code were included general character provisions, following the assurance of structural ductility for cyclic actions and admitted for the first time the usage of advance methods for the technically justified cases (see Table 1). The antiseismic gaps were determined in concordance with the following relation:

$$\delta_{\min} = 2 + 40\left(C_1 \cdot T_1^2 \cdot \frac{H}{H_1} + C_2 \cdot T_2^2 \frac{H}{H_2} \right)$$

(cm) were C_1 and C_2 are the seismic coefficients for both buildings: T_1 and T_2, fundamental periods of vibration (in seconds); H_1 and H_2, the total heights of the buildings and H=min (H_1, H2).
Under the P13-63 and P13-71 codes circumstances the biggest seismic coefficients were considered for the vibration periods between 0.30 and 0.40 sec. all through the periods belonging in the 0.90 and 1.50 sec. interval the values decrease until five times.
The elastic and inelastic responses spectra of the Vrancea NS 1977 (recorded at INCERC Bucharest) shows the maximal response (in velocities, accelerations, displacements, energies) exactly for the periods belonging to the 0.90-1.50 sec. interval.
The upgrade of the Romanian macro seismic map after the major earthquake from 4^{th} of March 1977 put in evidence worse situations for the regions where the antiseismic protection degree increase under the new P100-92 aseismic design code.
Bucharest was in the 7 antiseismic protection degree under the P13 circumstances and because of P100-92 code (and P100/1-2006; P100/3-2008) it arrive in a region with a value of 8 aseismic protection degree [meaning a PGA=0.20g and a corner period (Tc) about 1.5 sec. for P100-92 or PGA=0.24g and a corner period (Tc) about 1.6 sec.].
In this situation all the existing stock of buildings, especially the buildings with vibration periods belonging to the 0.3-0.4 sec. interval, were computed with low seismic forces. The social-cultural and the apartment buildings with the proper period of vibration in the 0.50-2.00 sec. domain, designed under the P13 codes has the design base seismic coefficient (c_B) of 3-4 times smaller than the P100-92 (and P100/1-2013) imposed coefficient, 2.5-3.5 times smaller than P100/1-06 (6.5-8.5 times smaller according to P100/3-2008). [4-14]



Graphic 1 – β comparison between aseismic design codes for Tc=1.6 sec.

Table 1 – Seismic design force according to Romanian codes

P13-63 Code	P13-71 Code	P100-92 Code	P100-92 Code
$S=c_B * G$ $c_B = Ks*\beta*\psi*\varepsilon$	$S=c_B * G$ $c_B = Ks*\beta*\psi*\varepsilon$	$S=c_B * G$ $c_B = \alpha*Ks*\beta*\psi*\varepsilon$	$S=c_B * G$ $c_B =(\gamma_I*\beta*a_g*\lambda/q)$

The nonlinear dynamic analysis carried out for 3D computational models, with the ANELISE program shows higher vibration periods and lower effective base seismic coefficients (C_B); it was considered three scaling values for the Vrancea NS 1977 (recorded at INCERC-Bucharest) accelerogram corresponding for PGA=0.08g, 0.20g and 0.25g (see Table 2). [1-14]

Table 2

<table>
<tr><th colspan="14">P13 Structures</th></tr>
<tr><th colspan="2" rowspan="4">P13-63</th><th colspan="3">RC Frames</th><th colspan="3">RC Dual Systems</th><th colspan="3">Soft and weak level</th><th colspan="3">Coupled walls</th></tr>
<tr><th>T_1</th><th>C_B</th><th>E_y</th><th>T_1</th><th>C_B</th><th>E_y</th><th>T_1</th><th>C_B</th><th>E_y</th><th>T_1</th><th>C_B</th><th>E_y</th></tr>
<tr><th>Sec.</th><th>%</th><th>tfm</th><th>Sec.</th><th>%</th><th>tfm</th><th>Sec.</th><th>%</th><th>tfm</th><th>Sec.</th><th>%</th><th>tfm</th></tr>
<tr><td>0.70</td><td>3.00</td><td>-</td><td>0.38</td><td>4.3</td><td>-</td><td>0.59</td><td>2.8</td><td>-</td><td>0.38</td><td>4.6</td><td>-</td></tr>
<tr><th rowspan="3">Nonlinear Dynamic Analysis</th><th>0.08</th><td rowspan="3">0.94</td><td>9</td><td>27</td><td rowspan="3">0.55</td><td>8.6</td><td>3</td><td rowspan="3">1.04</td><td>-</td><td>-</td><td rowspan="3">0.18</td><td>9</td><td>-</td></tr>
<tr><th>0.20</th><td>13</td><td>130</td><td>15.9</td><td>38.4</td><td>4.2</td><td>29.4</td><td>20</td><td>0.42</td></tr>
<tr><th>0.24</th><td>14</td><td>166</td><td>17.3</td><td>54.7</td><td>-</td><td>-</td><td>25</td><td>1.72</td></tr>
</table>

The close C_B values for PGA=0.20g and PGA=0.24g indicate the lack of resistance reserves in the case of a greater imminent earthquake.
The values E_y of the dissipated energy under post elastic deformations show us the next several conclusions:

- The plastic hinges occur for the P13 structures even the PGA value is 0.08g;
- For the soft and weak level buildings about 40% of the energy is dissipate by the first floor columns and the rest is uniform distribute on the vertical of the building;
- The energy dissipation appear for all kind of structural elements and the entire height of the building;
- The μ_θ (rotational ductility demand ratio) values result with high values for PGA=0.20g.

The comparison for the base seismic coefficients (in percent), for the 2nd importance constructions class and normal soil conditions are shown in the table 3.

Table 3

<table>
<tr><th rowspan="3">Buildings structural type</th><th rowspan="3">T (sec)</th><th colspan="3">P13-63</th><th colspan="3">P13-71</th></tr>
<tr><th colspan="3">Seismic Intensity Degree</th><th colspan="3">Seismic Intensity Degree</th></tr>
<tr><th>7</th><th>8</th><th>9</th><th>7</th><th>8</th><th>9</th></tr>
<tr><th>Flexible structures
for 10 levels buildings</th><td>1.00</td><td>2.20</td><td>4.40</td><td>8.80</td><td>2.00</td><td>3.40</td><td>5.40</td></tr>
<tr><th>Dual structures
for 10 levels buildings</th><td>0.55</td><td>3.10</td><td>6.10</td><td>12.20</td><td>4.00</td><td>6.50</td><td>10.40</td></tr>
<tr><th>Rigid structures
for 10 levels buildings</th><td>0.31-0.45</td><td>3.70</td><td>7.40</td><td>14.80</td><td>4.90</td><td>8.10</td><td>13.00</td></tr>
</table>

3. THE P13 STRUCTURES MAIN DEFICIENTES OBSERVED AFTER THE MAJOR EARTHQUAKES FROM 1977, 1986, 1990 AND 2004

The general behavior characteristics (damages and degradations, assurance level against the partial and total collapse) are determining from the codes deficiencies.

The case studies realized under the nonlinear dynamic analysis methodology and investigation spectral method shows the following general character conclusions:

- The dynamic amplification coefficient spectra (β) values decrease dramatically for the buildings with proper period of vibration T_1>0.3 sec. (P13-63) or >0.4 sec. (P13-71);
- The super unitary values of the behavior coefficient (q) do not show the real post elastic behavior of the P13 structures;
- The base seismic coefficients (C_B) resulted very small for the current P13 structures;

- Because of the lessons learned from the recent earthquakes appear to be necessarily to increase the seismic intensity degree in Bucharest and this multiply by 2 the P13 base seismic coefficient;
- The lowest values of the ductility capacity because of lack of the confinement reinforce;
- Brittle failure mechanisms for columns, spandrels, RC structural walls, joints and short columns;
- The lack of the horizontal and vertical reinforce in the RC structural walls webs;
- Bad RC structural walls conformation of the existing repetitive design projects (M1f4, M1f8 and OD);
- There are a lot of P13 structures which have just one longitudinal resistance line with insufficient stiffness, strength, shear area (OD), etc;
- The development of the Energy Dissipation Mechanism (EDM) is absolutely un-expectable and unfavorable, with localization of dissipation at one level (soft and weak story buildings);
- The simplifications accepted for that period of design together with the irregularly shapes of buildings produced lack of phenomena and great uncertainness;
- The flexible buildings suffer great dynamic amplifications under the 1977 earthquake;
- Erection deficiencies.

4. „OD" REPETITIVE BLOCK OF FLATS DESIGN PROJECTS

The "OD" section is a RC structural walls structure with one underground level and 10-11 ground levels. The destination for all the levels, including first floor, is for apartments. In the figure 1 is presented a simplified scheme for "OD" section.

The "OD" section were used between 1965 and 1976 for more than 8000 apartments, in buildings which have sometimes until 9 or 10 blocks, in different area from Bucharest.

Because of the lowest steel consumption and manpower the RC structural walls buildings were rapidly developed and used for multilevel structures.

The OD section structure is consist of: RC structural wall disposed on two principal directions, with 14-15 cm thickness. In the longitudinal direction there is only one RC structural wall with two coplanar segments, in the median axe. The transverse RC structural walls have 30x25cm bulbs in the building facades. The door openings are disposing unitary on the vertical. The spandrels are with 15x58 cm (the majority) and 15x28 for T4 pier.

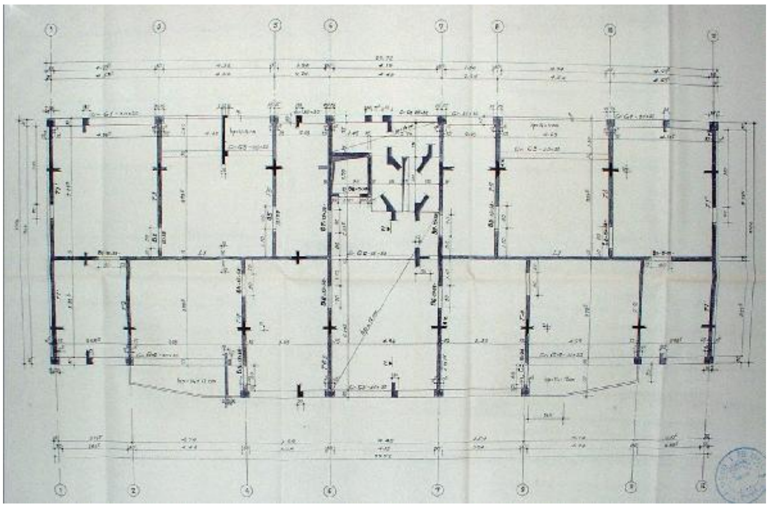


Fig. 1 – "OD-section" repetitive design project

The façade beams have 30x30 cm and the slabs are with 10 cm constant thickness.

5. THE DESCRIPTION OF THE APPLIED COMPUTATION METHODOLOGY

The main characteristics of the applied methodology are:

1. Establishing the Structural Computation Model (SCM):
   - The unfavorable length of the spandrel considered was equal with the clear span of it plus a half of the depth of the cross section;
   - The fix support was considered on the top level of the underground level;
   - The considered Bending Moment-Rotation diagrams for the plastic hinges sections were elastoplastic;

2. The principal accelerogram was Vrancea NS 1977, according with the response spectra;
3. For the Structural Elements Seismic Vulnerability (SESV) were determined the failure mechanisms and identified the brittle failure cases from shear forces. The ductility capacities were defined by the ratio h/x (where h=depth of the RC wall cross section and x is the real position of the neutral axis in the ultimate stage, computed by SEKON program, for medium resistances);
4. For the Structural System Seismic Vulnerability (SSSV) it were considered the behavior characteristics of the elements and the structural system response demands;
5. Taking into consideration that for all the OD sections, after the 1977 major earthquake, a lot of damages were observed (especially shear force brittle failures in all the spandrels) in the Structural Computation Model (SCM) were considered two variants: resistant spandrels case (active spandrels) and inactive spandrels.
6. The hypotheses of ductile failure for the RC structural walls were considered.

The analysis of both 3D structural models results underline a great importance of the spandrels for the transverse direction stiffness. The ratio between the stiffness in the case of active and inactive is about 4.8 which move the unfavorable position on the spectra (from T=1.2 sec. – inactive spandrels) to a favorable one (T=0.548 sec. – active spandrels).
The entire favorable responses tributaries to the movement in a favorable position on the spectra are: Reducing the absolute displacements and level drifts with 40% in comparison with the values obtained for the unfavorable spectra position; Reducing the total dissipated energy with 53%; Reducing the ductility demands; Increasing the sectional efforts (story shear forces and story overturning moments) with 25% in the active spandrels case.
The strength capacities computed for both cases are presented in table 4.

Table 4

Principal direction		Spandrels situation	$C_{B,Y}$ (%)
Existing	Transverse	Inactive	13.47
		Active	16.57
	Longitudinal	Inactive	10.70

where $C_{B,Y}$ is the base shear force coefficient in concordance with the capacity design rules.
The presence of the active spandrels on the transverse direction is very important increasing the strength capacities with almost 23%. The transverse direction strength capacity is greater with 60% than the longitudinal direction.

6. ESTABLISHING THE SPANDRELS FAILURE MODES

Because of the lack of knowledge and codes in that period the spandrels where under-dimensioned and the shear force capacity were not in concordance with the associated shear fore from the inelastic mechanism of the spandrels.
In the figure 2 there are presented the ratio of brittle failure tendency spandrels.

7. ESTABLISHING THE RC STRUCTURAL WALLS FAILURE MODES

To establish the failure modes of the RC structural walls it was used the following analysis methodology:
- The verifying were made for shear force brittle failure;
- Story shear forces $V_{max,demand}$ for each RC structural wall were considered the maximum non-simultaneously values presented by ETABS 2013 program, from the biographic analysis;
- The strength capacities for shear forces V_{cap} were computed for each level, both for the plastic and elastic zone, taking into consideration the CR2-1-1.1 code (for RC structural walls);
- Identification of the failure mode was made using the R_V indicator as follows:
  - $R_V = V_{cap} / V_{cerut}$
  - If $R_V>1$ – ductile failure
  - If $R_V<1$ – shear force brittle failure

The results in percent are presented in fig.3.

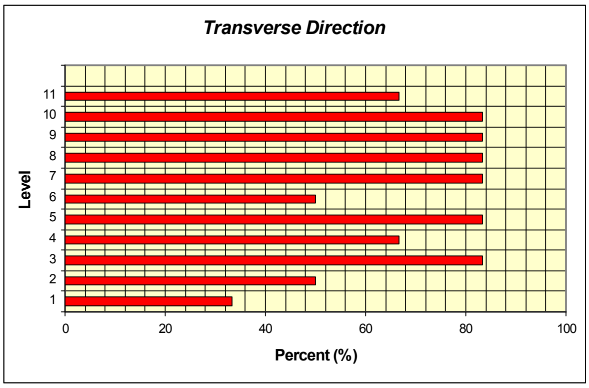


Fig. 2 – Brittle failure tendency for RC spandrels

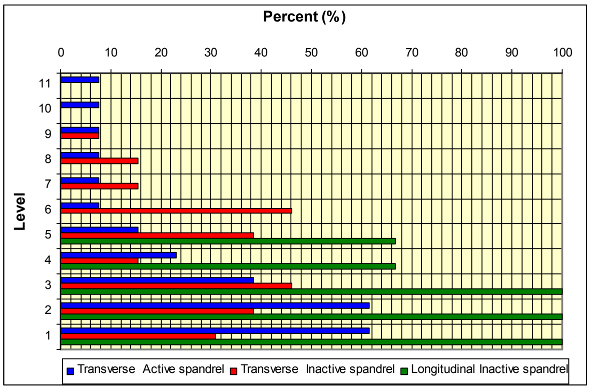


Fig. 3 – Brittle failure tendency for RC piers

The conclusions using this indicator are:

- All the RC structural walls has brittle failure modes; the number of levels and the level positions were different in function of hypothesis of the spandrels (active or inactive);
- The most advantageous hypothesis is the active spandrels one, because the number of the brittle failure levels is decreasing;
- Using or not the overstrength is not so spectacular;
- The effect of active/inactive spandrels on the longitudinal direction is not so important.

Taking into consideration all the results we may remark that for this structural system type is more advantageous to realize the hypothesis of active spandrels, with a ductile failure mode.
Using the Plastic Hinges Time History Analysis on the spandrels and RC structural walls "frame"-models, we may say that:

- The significant time length of the plastic mechanism is from 6.20 sec. to 8.00 sec;
- For some of the RC structural walls, the plastic hinges occur simultaneously on many levels (example T2 – first 6 levels).

8. THE STIFFNESS CRITERION VERIFICATION

The overall stiffness of the structural system determine the fundamental period of vibration. In this idea, in the inelastic response spectra diagrams, the values of proper period of vibration and the value of $C_{B,Y}$ (which represent the system strength capacity indicator) offer the spectral position of the structural system.
In our case increasing the stiffness we decrease the periods of vibration and we shall move the spectral position in a favorable area simultaneously with a reducing of the displacements and ductility demands.
Taking or not into consideration the participation of the spandrels (active or inactive) is very important for the transverse direction, to recover the demand responses.

9. DUCTILITY CAPACITIES ANALYSIS

The ductility capacities were verified using the ratio h/x. These ratios were determined for all the RC structural walls and piers, for each level in both ways of the seismic action.
Even that a lot of these walls have favorable values for these ratios, there are also several excessive values which must be improved (especially the RC structural walls with edge bulbs). The synthetically results are presented in the figure 4.

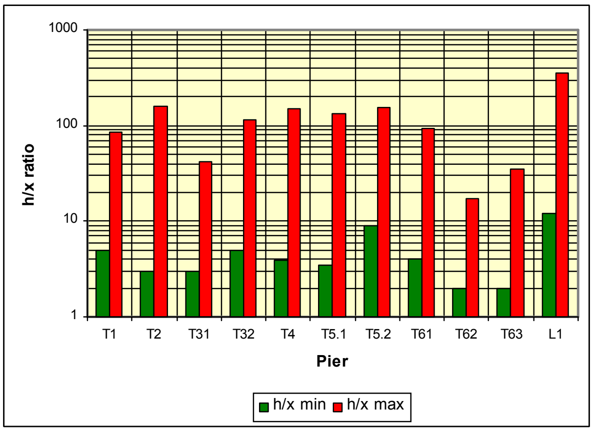


Fig. 4 – Ductility capacities

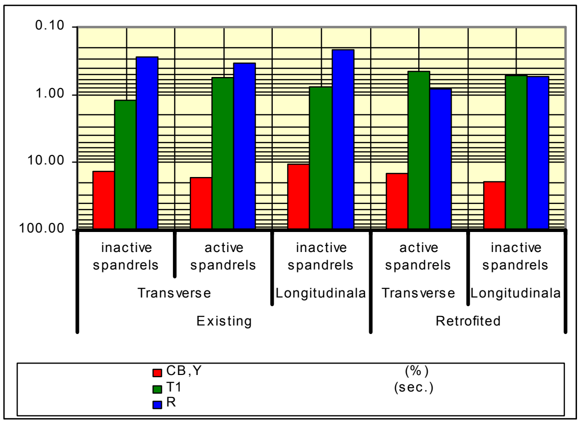


Fig. 5 – The upgrading of structural retrofitted system characteristics

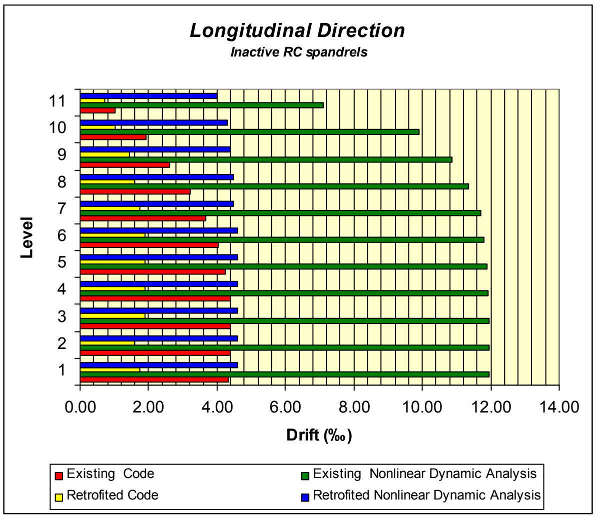


Fig. 6- Longitudinal direction – relative level drifts

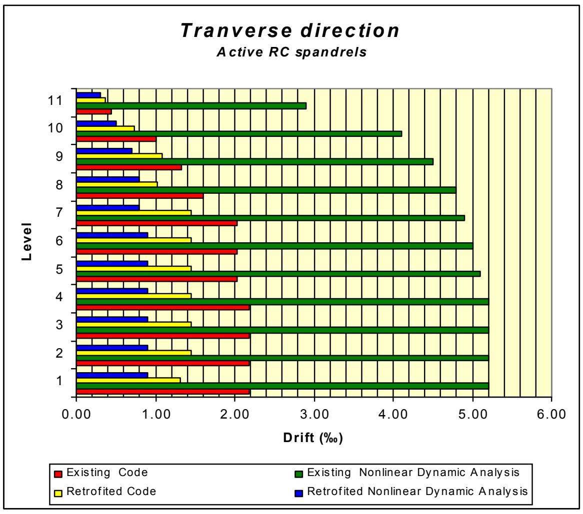


Fig. 7 – Transverse direction – relative level drifts

10. CONCLUSIONS

1. After the identification of the failure mechanisms for all the structural elements we established next characteristics:
    a. All the spandrels present shear forces brittle failures;
    b. All the RC structural walls has one or several levels with shear forces brittle failures;
    c. The infrastructure (substructure and foundation structure) is soft and weak but may be talked into consideration for the existing situation.
2. Taking into consideration the spandrels (to be active) is favorable from all the view points (periods of vibration, displacements, ductility demands);
3. The retrofitting intervention principle is to ovoid the shear forces brittle failure of the structural elements. In this way jacketing of the RC structural walls and re-casting the spandrels to improve the ductility capacity.
4. The retrofitting is oriented first to the longitudinal direction and for this we considered the implementation of a new frame structural subsystem applied on both facades. After the joining of these elements with the existing one we shall increase also the characteristics for the transverse direction;
5. The necessity of infrastructure structural interventions depends of the increasing of the strength and stiffness capacities for the super-structure.

Having into consideration to ovoid the shear forces brittle failure for all the structural elements (see figure 8).
Another step is to increase the nominal assurance degree R, on the longitudinal direction.
In the figure 5 are presented a stock of structural response characteristics for existing and retrofitted structure.
In the 6 and 7 figures there are presented drift comparisons for both longitudinal and transverse directions.

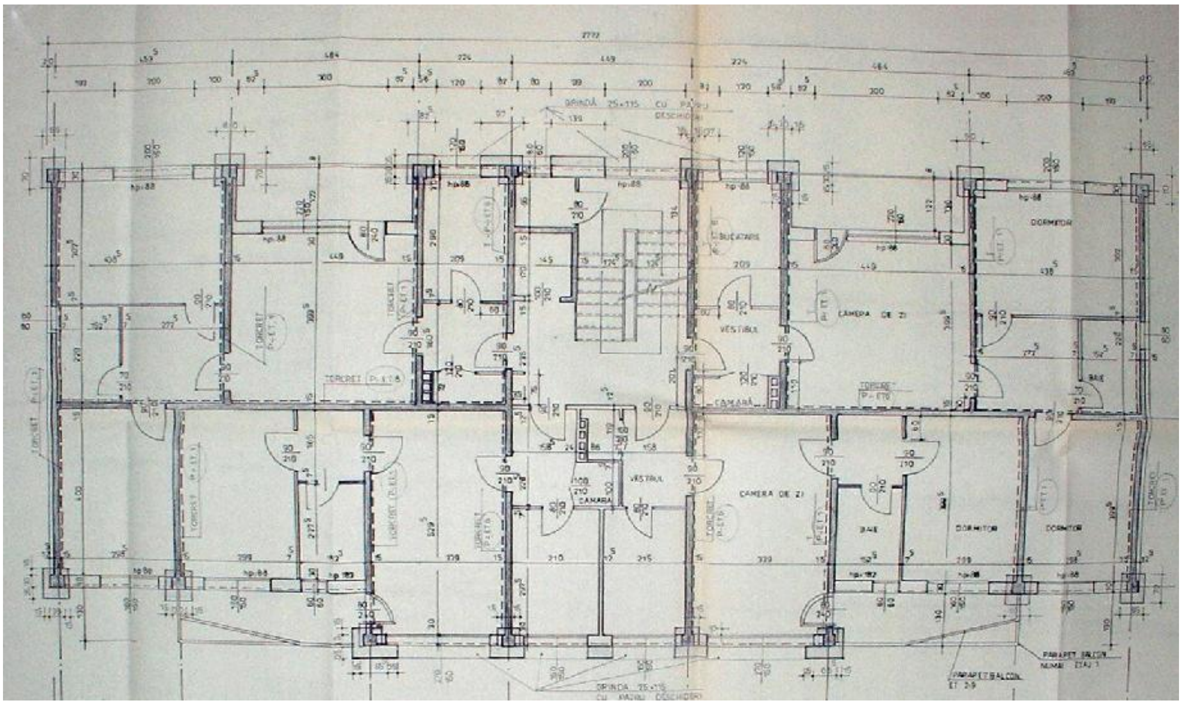


Fig. 8 – The proposed retrofitting solution

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