# Evaluation and retrofitting of a concrete wall building placed in Bucharest using multistage rubber bearings

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#### **ABSTRACT:**

This paper presents the seismic evaluation of an existing reinforced concrete shear wall structure located in Bucharest and proposes a retrofit solution based on Seismic Base Isolation Method using Multi-Stage Rubber Bearings. The seismic evaluation was based on static nonlinear analysis (Push-Over) on a 3D model. Nonlinear behaviour was assumed for the shear walls, beams and columns. The strength and displacement capacities were assessed using the European code (EUROCODE 1998-3). A retrofitting solution using Seismic Base Isolation method is designed. The isolating layer was obtained using several types of Multi-Stage Rubber Bearings, which were designed to sustain the loads and imposed displacement. The efficiency of the retrofitting solution was determined through a Time History analysis performed on the 3D model. The conclusion of the study is that the base isolation method based on multi-stage devices can represent an alternative to the classical retrofitting methods even for areas with high predominant period earthquakes that impose large displacements demands.

Keywords: retrofitting, multi-stage rubber bearing, seismic base isolation.

### **1. INTRODUCTION**

An important problem regarding the seismic risk in Romania is the existence of a large number of buildings designed and built before the occurrence of the 1977 Vrancea earthquake, which revealed the vulnerability of the multi-storey RC buildings, especially in the Romanian Plane region. The pulse-type earthquake having a large predominant period of 1.6s induced large displacement demands, and, as a result a number of about 28 multi-storey RC buildings collapsed and many other sustained heavy damage. Generally these buildings were characterized by a non-ductile behaviour and particularly the RC frames structures by low lateral force strength.

The current study presents the evaluation and the retrofitting proposal for a building designed and built in Bucharest during the 60's. The structural system of the building consists from shear walls having large flexural capacity but low shear capacity. This structural type was used on large scale in that period in Romania for apartment buildings. In the current study the evaluation of the shear capacity was done according to Level Three (N3) methodology considering the procedure defined in the European code (Eurocode 1998 part 3). In case of the retrofitting, the adopted strategy implies the reduction of the seismic demands. The reduction of the seismic demand was done through a Base Isolation system obtained using Multistage Rubber Bearings. The base isolating system was designed to ensure that only a small amount of seismic energy reaches the superstructure of the building thus the seismic demand on the structural elements decreases.

### 2. THE ANALYSED BUILDING (GENERAL PRESENTATION)

The analysed building is a shear wall structure designed and built during the 60's; the building has 11 stories above the ground. The plan conformation of the building can be seen in figure 1. The structural

system designed to carry the lateral seismic load is dominated by two large T section walls placed on each end of the structure.



Figure1. Story Plan

In the X direction, these walls are virtually the only two vertical elements capable to carry the seismic load on this direction. In the Y direction there is a set of lamellar concrete walls, also, but the behaviour under seismic loads is still dominated by those two T shape elements. In the current paper, only the results obtained for these T shape walls (named below as DT walls) are presented in detail.

Table1 Longitudinal and transversal reinforcement percent for DT walls

Story	Vertical		Horizontal		
~~~~)	End (%)	Web (%)	End(%)	Web (%)	
Cround	0.65	0.38	0.188		
loval	0.497	0.334	0.107	0.223	
level	0.863	0.334	0.134		
	0.668	0.214	0.094		
E1-E2	0.447	0.254	0.188	0.126	
	0.485	0.254	0.094		
E2 E0	0.631	0.048	0.094	Does not	
ЕЭ-Е9	0.389	0.111	0.188	exist	
E10	0.631	0.216	0.094	0.126	

Table 1 presents the longitudinal and transversal reinforcement percent for the DT wall section. For the DT wall section the reinforcement ratio is different from one part of the wall to another (at the same level), that is the reason why in table 1 there are different reinforcement ratios at the same level. This table should provide a qualitative rather than a quantitative view of the wall reinforcement. It should be mentioned that, between the levels three and nine the horizontal reinforcement in the web does not exist. It was a common practice during those years not to include horizontal reinforcement in the wall. Also we should have in mind that the web thickness is very small (15cm) compared with the height of the section (7.3m on X direction and 8.75m on Y direction).

## 3. BUILDING EVALUATION

The shear capacity of the DT wall was done according to the N3 methodology considering the European Code EC8 part 3. The structure was modeled in the Perform3D software assuming a nonlinear behaviour. The nonlinear behavior of the structural elements was modelled as follows [3]:

- For the reinforced concrete beams and columns a lumped moment-rotation hinge model was considered at each end of the element; for the columns the interaction between the capacity and axial force was accounted.
- For the structural walls a fiber model was used; for each fiber (concrete or reinforcement bar) a behavior rule was defined;

The materials used in the model were concrete B200 (C12/15) and steel OB37 ( $f_{yd}$ =210 MPa). The mean strength values corrected by global/local safety coefficients were considered for each material. The safety coefficients are defined in the above mentioned code for each loading type.

The seismic load was modelled by a set of horizontal forces placed in the mass centre of each floor (considering 5% accidental eccentricity). The vertical distribution of these horizontal forces was determined according the fundamental vibration modes on X and Y direction. The displacement demands corresponding to each limit state were assessed from the mean response of the equivalent single degree of freedom system to five artificial accelerograms compatible with the design spectrum (using the appropriate equivalency coefficients).

Having in mind that the behavior of the structural system under seismic load is dominated by the behavior of the two DT walls, only the results regarding these walls will be presented in detail in the current paper. Because the section of the DT wall is not symmetrical in relation with the Y axis, there are two different loading situations: one where the compression zone is in the web and another where this zone is in the flange.



**Figure2.** Shear Force –Rotation diagram at the base of DT wall X direction (compression in web –left part and in the flange – right part)

Figure 2 presents the Shear Force –Rotation diagrams recorded at the base of DT wall. The Rotation parameter was chosen for these diagrams mainly because in the European norms the shear capacity value is related with the rotation (through  $\mu_{\theta}$  parameter). The left diagram presents the situation in which the compression zone is in the web and the right diagram when it is in the flange. With red line it is represented the shear capacity controlled by the stirrups (V<sub>R</sub>), with yellow line the shear capacity controlled by the stirrups (V<sub>R</sub>), with yellow line the shear capacity values associated with the three limit states considered in the study (SLS, ULS and SLSV).

Each limit state was defined considering the following Mean Returning Intervals: for SLS the associated MRI is 50year, for ULS the associated MRI is 100 years and for SLSV the associated MRI is 475 years. The usage of the displacement demand concept (computed for each limit state) in association with shear force analysis is not very common. However, in the current study this approach was preferred mainly because it highlights the fact that even for a moderate seismic event (associated to SLS) a high damage level is expected for the studied building. The wall shear capacity is reached for a rotation much smaller than the rotation associated with SLS limit state. However, it should be mentioned that when the compression zone is in the web of the T section the shear capacity is

controlled by the stirrups and when the compression zone is in the flange the capacity is controlled by the diagonal crushing. For this direction the ratio between the total yielding force and the total weight of the building is 0.21, the displacement demand for ULS is 0.14m. The R3 coefficient (the ratio between the shear capacity and the shear demand associated with ULS limit state) is 0.18 for one loading case and 0.23 for the second loading case. The conclusions presented above (for the seismic load on X direction) are adequate for Y direction too, and are not presented in detail. The wall shear capacity is reached for a lateral displacement lower than the one associated to SLS limit state.

A general conclusion of the evaluation process is that the DT wall has a large flexural capacity (mainly due to the geometry of the section) but in the same time an important deficit regarding the shear capacity. The evaluation process revealed that the shear capacity at the base of the wall is reached for lateral displacements much smaller than those associated with the SLS limit state. Furthermore, for ULS, the induced shear forces exceed the capacity on the entire height of the wall. This type of building is prone to damages even for small seismic actions, mainly because the building has high stiffness and flexural capacity (due to the geometrical configuration), but small shear capacity.

# 4. RETROFITTING OF THE BUILDING

Any retrofitting strategy must start from the general conclusion presented in the previous section. The retrofitting solution should be able to increase the shear capacity of the building without increasing the flexural capacity. One possible strategy, considered a "classic" approach, involves the shear strengthening of the RC walls. This solution involves dramatic and long duration disturbance of functionality of the building. Furthermore, the ductility capacity of the walls has to be increased and it cannot be done without major interventions at the base of the structure. Another solution is to limit the energy that reaches the structure in case of a seismic event. This energy limitation can be obtained through a Base Isolation solution. In the current study the second approach was preferred.

Having in mind the particularities of the seismic load associated with the Bucharest region (long control periods,  $T_c=1.6s$ ), a classical base isolation solution will lead to a new set of problems associated mainly with the stability of these classic isolation devices for large imposed displacements. Previous studies [1] [2] performed by the authors revealed the fact that in order to overcome the large displacement demand imposed by the seismic load, it is needed to ensure a large damping level in the isolating layers and/or to use large isolation devices. In both situations the efficiency of the isolation systems will be significantly reduced.

In order to overcome the problems associated with a classical base isolation system the authors decided to use Multistage Rubber Bearings devices. Although in the past, these devices were mainly used to isolate light buildings [3] (which is not the case in the current study) the main characteristics of the devices (small lateral stiffness with large displacement capacity) made them the ideal candidates to be used in this study.

# 4.1 Design of the Multistage Rubber Bearings

The multi-stage rubber bearings are the rubber bearings which mainly consist in small rubber devices located at every of the four corners of one common stabilizing steel plate (see figure 3). Consequently, the horizontal displacement capacity is considerably increased while the horizontal stiffness of the device is maintained on a lower value (thus a longer isolated period can easily be achieved). For a large horizontal displacement the classical device tends to have major stability problems, while for the same displacement a multi-stage device exhibits a stable behavior. The conclusion is that the multi-stage devices are much more suitable for very large imposed lateral displacements.

The design of the multistage rubber devices implies performing an analysis regarding the stability of the device considering both buckling limit state and roll-out limit state. Even if a roll-out failure can occur only in case of dowelled shear connections (which is not the case here) a good design practice is

to limit the total lateral displacement to the roll-out value [4]. Regarding the buckling limit state, in contrast with the classic case, a multistage device can exhibit two distinct buckling modes: first (named in the study "local" buckling) which implies the failure of one individual device in a certain layer and the second which implies the buckling of the entire multistage device (named in the study as "global" buckling). The equations which characterize those two situations can be found in the table 2.



Figure3. Classic Rubber Bearing versus Multistage Rubber Bearing

Table2 Multistage	rubber bearings	(Stability limit states)
0	0	· · · · · · · · · · · · · · · · · · ·

e oraș en	Buckling Limit S	tate		
	"Local" Buckling	"Global" Buckling		
Critical Load	$P_{cr}^{local} = m_e P_{cr}^1 = m_e \frac{\sqrt{2}\pi GS_l A_l r_l}{t_r^l}$	$P_{cr}^{global} = \frac{\sqrt{2}\pi GS_l A_t r_t}{t_r}$		
Minimum distance between elastomeric devices (2d <sup>min</sup> )	The minimum distance between the elastomeric devices in order to impose a failure due to "Local" Buckling (see figure 3): $P_{cr}^{local} \leq P_{cr}^{global}$ $d_{i}^{minim} = \sqrt{\frac{4A_{t}}{m_{e}\pi\phi_{i}^{2}} \left(\frac{P_{cr}^{local}t_{r}}{\sqrt{2}\pi GS_{l}A_{t}}\right)^{2} - \frac{\phi_{l}^{2}}{16}}$			
Critical Displacement	$d_{cr}^{total} =$ For: $\frac{p_1}{p_{cr}^l} \in [0 - 0.3]$ $d_{cr}^{las}$ For: $\frac{p_1}{p_{cr}^l} \in [0.3 - 1]$ $d_{cr}^{las}$	$n_e d_{cr}^{layer}$ $v^{er} = 2R \left[ 1 - \frac{1}{2} \left( \frac{3\pi}{4} \right)^{\frac{2}{3}} \left( \frac{P}{P_{cr}} \right)^{\frac{4}{3}} \right]$ $v^{er} = 2R \left\{ \frac{\pi}{4} \left[ 1 - \left( \frac{P}{P_{cr}} \right)^2 \right] \right\}$		
Roll-out Limit State				
Roll-out Critical Displacement	but Critical lacement $d_{Roll-Out}^{total} = n_e d_{Roll-Out}^{tayer}$ $d_{Roll-Out}^{layer} = b \left(\frac{1}{1 + \frac{G}{p_l} \frac{h_l}{t_r^l}}\right)$			

The parameters presented in the table 2 are:  $m_e$  – the number of elastomeric devices per layer;  $n_e$  – the number of layers; G – shear modulus of the elastomeric devices;  $S_1$  – shape factor for the elastomeric devices;  $A_l/A_t$  – the area for one elastomeric device/ for all elastomeric devices in one layer;  $r_l/r_t$  – radius of gyration for one device/for the entire multistage device;  $t_r^1$  – total height of the rubber for one device;  $t_r$  – the total height of the rubber for the entire multistage device;  $\Phi_l = 2R$  – the diameter of one elastomeric device;  $P_l$  – the vertical load on one elastomeric device,  $p_l$  – the vertical pressure on one elastomeric device.

The values presented in table 2 were obtained considering that the distance between any elastomeric device, on a single layer, is large enough to ensure that the critical force associated with the "local"

buckling limit state is smaller than the force associated with the "global" buckling (thus the distance between any two elastomeric devices is larger than  $2d_{min}$ ). The equation for the distance  $d_{min}$  was obtained by imposing the above mentioned hypothesis.

The designing algorithm for the multistage devices relies on the assumption that the following parameters are known:  $m_e$  – the number of elastomeric devices per one layer;  $n_e$  – the number of layers; G – shear modulus of the elastomeric devices;  $\xi$  – the damping proprieties of the elastomeric material;  $S_1$  – shape factor for the elastomeric devices. Considering a targeted horizontal isolated period ( $T_h$ ) and a design response spectrum, a displacement demand can be easily computed. Imposing a value for the allowed distortion ( $\gamma$  = Total lateral displacement/total height of the multistage device) the geometry of the multistage devices is obtained. We should mention that the allowed distortion value used in the designing algorithm for the current study was  $\gamma$ =100%.

The Multistage base isolating solution designed for the building evaluated in the first section of the article consists in 16 multistage devices grouped in five distinctive types. For all of these devices the number of chosen layers was 5 and the number of elastomeric isolator on one layer was set up 4. The targeted isolated period was 4.5s, the shear modulus for the elastomeric material was chosen 0.7 MPa and the damping level for the devices was chosen 15%. Considering the design response spectrum defined for the Bucharest region, we obtained a designing displacement equal with 0.44m. In the table 3 the main features of the multistage solution designed considering the above mentioned assumptions are presented.

Туре	n <sub>e</sub>	m <sub>e</sub>	Th (s)	F <sub>1</sub> (m)	t <sub>l</sub> (m)	d <sup>crBuckling</sup> total (m)	$d^{crRoll-out}_{total}(m)$	2d <sub>min</sub> (m)
MS1	5	4	4.5	0.35	0.089	1.61	1.69	0.85
MS2	5	4	4.5	0.3	0.089	1.29	1.45	0.71
MS3	5	4	4.5	0.44	0.089	2.07	2.15	1.05
MS4	5	4	4.5	0.36	0.089	1.68	1.79	0.88
MS5	5	4	4.5	0.29	0.089	1.22	1.40	0.69

 Table3 Multistage rubber bearings (Retrofitting solution)

From the table we can notice that the displacement capacity of the multistage devices is at least double compared with the displacement demand. The location of each device was chosen so that the eccentricity in the isolating system to be at a minimum level.

### 4.2 Evaluating the solution through a Time History analysis

The isolation solution presented in the previous section was tested through a set of nonlinear time history analyses. The analyses were performed using Perform 3D software and considering the same model for the superstructure like in the evaluation process. The multistage devices were modelled considering a linear behaviour and using the Rubber Isolator model available in the software. The damping proprieties of the isolating devices were modelled considering pure damping devices which are acting in parallel with the isolation devices. For the superstructure a damping level equal with 3% was considered. This level is smaller than usual mainly because one desiderate of the base isolation method is to reduce the damages of the non-structural elements in the superstructure, thus the damping level of the superstructure will certainly be smaller.

The earthquake loading was modelled with a set of four accelerograms: a natural one recorded in Bucharest during the 1977 earthquake (named in the study Vrancea77) and another three artificial accelerograms compatible with the design spectrum recommended by the Romanian codes. The first accelerogram (Vrancea77) was amplified in order to obtain the design spectral value provided for Bucharest. Each recording was considered separately on both transversal and longitudinal directions.

The study was focused on four main aspects, the acceleration demand in the superstructure, the displacement demand both at the isolation layer and in the superstructure, the behaviour of the structural elements in the superstructure (if these elements remained in the elastic domain and if shear capacity is exceeded) and the shear demand in the walls. The analysis was performed separately on both longitudinal and transversal direction. Due to the lack of space and the fact that the conclusions are similar in both of these situations, only the results for the longitudinal direction will be presented.



## 4.2.1 Acceleration demand

Figure4. Acceleration demand on the longitudinal direction

Figure 4 presents the acceleration obtained in the longitudinal direction for each accelerogram. Blue colour corresponds to the input signal, red colour to the acceleration recorded at the base level above the isolating system and the green colour to the acceleration recorded at the top level. We can conclude that from the acceleration point of view the proposed multistage system is very efficient. The input acceleration is reduced in the superstructure almost twice and the amplification of the acceleration from base to top is insignificant. Lower levels of acceleration in the superstructure lead to smaller degradations in the non-structural elements during the earthquake. A significant reduction of the accelerated mainly due to the lengthening of the structure period. The lengthening of the period will generate larger displacement demands which are concentrated mainly at the isolating system level.

# 4.2.2 Displacement demand

In figure 5 it can be noticed the relative displacement demand recorded for the longitudinal direction for each accelerogram considered in the analysis. With blue colour is represented the relative displacement recorded at the base level above the isolating system while with red colour the relative displacement recorded at the top level.



Figure5. Displacement demand on the longitudinal direction

From figure 5 two conclusions can be drawn. First, that the relative lateral displacement recorded at the top level is almost identically with the one recorded at the base level (the diagrams are almost perfectly overlapped), thus the total drift of the superstructure is very small. A very small drift value implies that the superstructure is moving like a perfect rigid body and the damage level in the structural and non-structural elements is very small.

The second conclusions is that, in all the situations considered in the analysis, the displacement demand recorded at the base level is much smaller than the lateral displacement capacity of the multistage devices (see table 3). The smallest lateral displacement capacity is associated with the multistage device type 5 (MS5) and it is equal with 1.22m while the largest lateral displacement demand is associated with Vrancea77 recording and it is equal with 0.6m. Therefore the capacity is at least two times larger than the demand ensuring a safety factor (R3) equal with more than two.

Comparing the displacement demands recorded at the base level and the critical displacements associated with a Roll-out limit state presented in the table 3, we can conclude that the safety factor for this limit state is larger than two, also. As mentioned in the previous section, according to the codes, the Roll-out limit state is associated with a non-bolted connexion, which is not the case here, but a good designing practice is to limit the total displacement to this value for the bolted connexions too.

### 4.2.3 Energy balance analysis

This part of the study was focused on the behaviour of the structural elements under dynamic loads. The main idea was to determine if the structural elements will remain in the elastic domain or if they will have any inelastic behaviour. In order to determine if the structural elements will remain in the elastic range energy balance analyses were performed. Through this analysis it is possible to determine how the input energy is dissipated during an earthquake. In the figure 6 it can be noticed the results obtained for the longitudinal direction. These results should be used only as a qualitative evaluation tool rather than a quantitative one.



Figure6. Energy balance analyses on the longitudinal direction

In the figure 6 we can notice the energy balance diagrams for the longitudinal direction. The figure presents the energy dissipated during the analysis through various mechanisms: the kinetic energy in light blue colour, the strain energy in purple colour, the energy dissipated through Beta-K mechanism in green colour (Rayleigh damping), the energy dissipated by the isolating system in red colour and finally the energy dissipated through an inelastic behaviour of the structural elements in dark blue colour. We can notice that the superstructure remains in the elastic range; the energy dissipated through an inelastic mechanism is zero for all the cases. The main part of the input energy is dissipated in the isolating system through a viscous mechanism. The conclusion is that the multistage isolated system proposed in this study is extremely efficient, ensuring a behaviour in the elastic range for the entire superstructure, eliminating the necessity of increasing the ductility capacity of the structure.

### 4.2.4 Shear force evaluation

In the figure 7 it can be noticed the shear forces recorded at the base of DT wall on longitudinal direction for each accelerogram. In the figure it is also represented the shear capacity for the DT wall associated with the loading situation when the compression zone is in the web (red colour) and in the flange (blue colour). The conclusion is that the shear demand on the wall is smaller than the shear capacity in all the cases analysed in this study. Therefore we can conclude that main purpose of the retrofitting process was fulfilled, the shear demand is significantly reduced.

The main conclusion on the evaluation is that the isolating system based on multistage rubber devices is able to fulfil the requirements imposed by the codes: the acceleration demand in the superstructure is significantly reduced and is not amplified from base to top, the displacement demands recorded at the isolated level are at least two times smaller than the displacement capacity of the multistage devices, the total drift of the superstructure is very small ensuring a smaller damage level in the non-structural elements and as the energy balance analyses have shown, the structural elements remained in the elastic rage during the entire analysis.



Figure7. Shear Force recorded at the base of DT wall (longitudinal direction)

#### 5. CONCLUSIONS

The current study analysed a shear wall building placed in Bucharest which is emblematic for a large category of buildings from Romania designed and built during the '60. The building structure consists of reinforced concrete shear walls, featured by complicated shapes and low or inexistent shear reinforcement. The main deficiencies of the shear walls are: shear capacity not associated to flexural moment capacity and lack of sufficient ductility due to absence of any seismic detailing and usage of smooth (plain) steel. Nevertheless, the structure has high stiffness and relatively high lateral force strength (for the 60's period) due to the geometry of the wall sections, and can be considered an adequate candidate for base isolation retrofitting solution.

The Multi-Stage Rubber Bearings solution for isolating layer was chosen. Although, these devices were mainly used to isolate light buildings (which is not the case in the current study), the authors decided to use these devices in order to overcome the designing problems generated by the long predominant periods which are associated with the Bucharest area ( $T_c=1.6s$ ), and the large displacement demands. Each device was designed considering the stability criteria associated with both buckling and roll-out limit states. The efficiency of the retrofitting solution was determined through a Time History analysis performed on the 3D FEM model. The analyses revealed the fact that the base isolating system based on multistage rubber bearings is very efficient. The demand on the superstructure is significantly reduced and in the same time the isolating system is very stable on large deformations. The lateral displacement capacity of the devices is at least two times larger than the displacement demand imposed by the seismic load.

The general conclusion of the current study is that the base isolation method based on multi-stage rubber devices can represent an option for retrofitting and design of new structures in seismic areas featured by earthquakes with high predominant periods inducing large displacement demands, for which a classical base isolation solution can raise important problems regarding the stability of the classical isolating devices.

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