SEISMIC REHABILITATION OF AN EXISTING PRE-1940 BUILDING. CASE STUDY

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SUMMARY

This paper presents one pilot project financed by JICA that aims to promote new retrofitting techniques and encourage the citizens from Bucharest to accept and support seismic rehabilitation of existing vulnerable buildings. The seismic assessment of the selected building revealed some major deficiencies of the original design and established that the lateral capacity of the building is significantly less than the present seismic demand. In order to propose the most appropriate solution for rehabilitation of the existing structural system, three different retrofitting strategies were extensively analyzed. Their advantages and disadvantages as well as a concise presentation of the retrofitting design process are also reported in the paper.

INTRODUCTION

Bucharest, the capital and the most important city of Romania have experienced several harmful earthquakes throughout history. The Romanian code of earthquake resistant design indicates that the city has a moderate to high seismic hazard level with a PGA (Peak Ground Acceleration) of 0.24g with 40% exceedance probability in 50 years.

In 1977 earthquake most of the victims (1424 deaths) and economic losses were recorded in Bucharest. In this city with more than 2 millions inhabitants, the most vulnerable constructions are medium and high rise buildings built before 1978 earthquake resistant design code. While the buildings before 1941 were built without considering seismic design, between 1941 and 1978 the design spectrum was not appropriate for mid and high rise buildings when considering the characteristics of strong ground motions recorded in the city during 1977, 1986 and 1990 earthquakes.

After 1990 the Romanian Government and local authorities started a national program for evaluation of seismic resistance of vulnerable buildings. This program was later integrated into a national strategy for seismic risk reduction. Nowadays this ongoing process of seismic evaluation and retrofit of existing vulnerable buildings represents a high priority for Romanian Government and civil engineers.

In October 2002, after four intensive years of preparation, National Center for Seismic Risk Reduction was founded as a result of joint collaboration between Japanese International Cooperation Agency and Romanian Ministry of Transports, Constructions and Tourism. The development of the seismic evaluation and seismic retrofitting manuals represents one of the main outcomes of the JICA Technical Cooperation Project for Seismic Risk Reduction in Romania. In order to encourage new retrofitting techniques and to persuade the citizens from Bucharest to accept and support the retrofitting works for the existing vulnerable buildings, JICA and NCSRR promoted two retrofitting pilot projects. The Japanese experts and the Romanian engineers

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from NCSRR provided technical expertise and supervised the design process carried out in partnership with PROIECT BUCURESTI, one of the major and oldest design companies in Bucharest.

In Bucharest, the most vulnerable constructions are represented by medium and high-rise RC buildings erected before 1940, especially because they were designed without taking into account any seismic forces. After 1977 earthquake, 23 out of 31 collapsed buildings correspond to the above described type. Figure 1 presents two such buildings that partially collapsed 30 years ago.

Figure 1: Pre-1940s medium and high-rise buildings that partially collapsed during the earthquake on March 4, 1977 (http://nisee.berkeley.edu)

In this context, one of the JICA/NCSRR/Proiect Bucureşti projects was aiming to retrofit a pre-1940 building. The selected structure is located in the centre of Bucharest on Ştirbei Vodă Street at no. 20. It is an 8-storey block of flats that is owned by its residents. Both major earthquakes in twentieth century – in 1940 and 1977 – caused severe damages to structural elements and unfortunately only some limited rehabilitation measures were taken. A seismic evaluation of the building was performed in 2000 and it revealed that it has a very vulnerable structure.

This paper briefly presents the design steps that were followed in order to establish the most appropriate retrofitting solution and it highlights the main technical details of the proposed rehabilitation methods.

BUILDING SPECIFICATIONS

The construction process for the residential building on Ştirbei Vodă Street at no. 20 started in 1934 and it was completed in 1935. This building has 9 levels that include one partial underground level and the attic. While the height of the underground level is only 2.60 m, the upper stories height is 3.20 m. The building footprint at the ground level has 450 m^2 and the total unfolded area is nearly 4250 m^2.

The building has an irregular shape both in plan and elevation (Figure 2). It resulted from the architectural restrictions imposed by the neighbouring buildings and the actual shape of the plot of land. The top story and wooden roof levels have setbacks of almost 2.00m and 5.00m, respectively, from the main façade facing the Ştirbei Vodă Street (see figure 2).

The structural system of the building consists of reinforced concrete columns, beams and slabs. According to the Romanian design practice before World War II, the position of the vertical elements resulted only from architectural criteria and, as can be seen in figures 4 and 5, the columns were arranged in a non-uniform pattern. The partition walls are made of brick masonry. The thickness of these interior walls is either 9 or 14 cm. The exterior enveloping walls are 28 cm thick and are also made of brick masonry. The building structure is supported by relatively small reinforced concrete foundation beams.

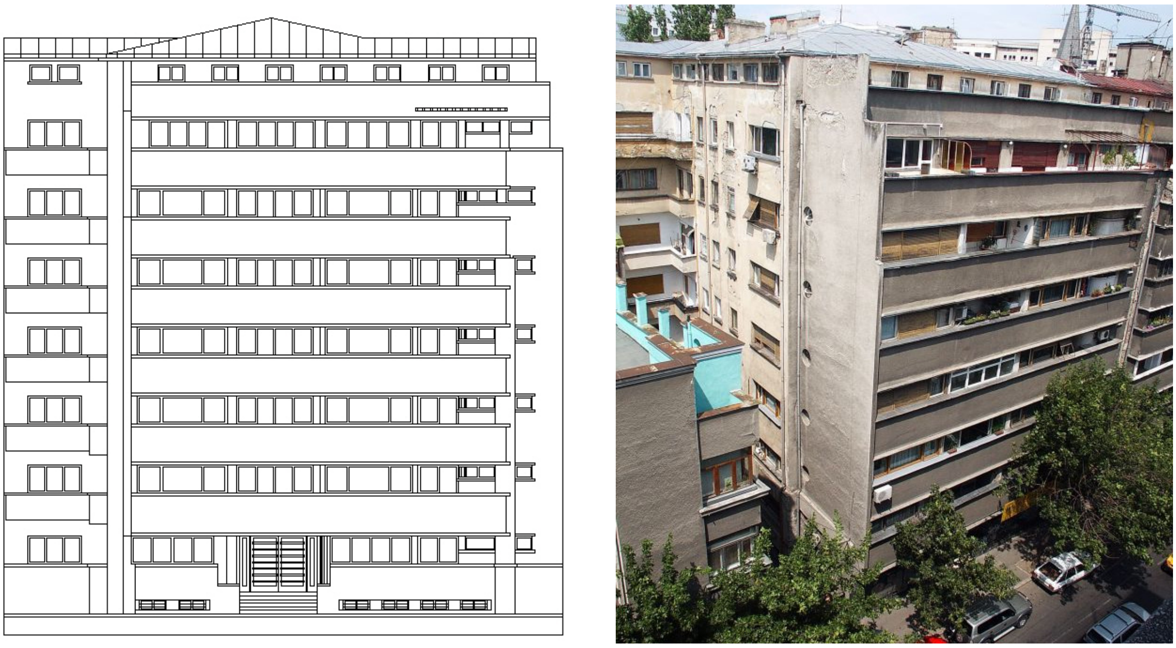


Figure 2: Main façade of the building facing the Ştirbei Vodă Street

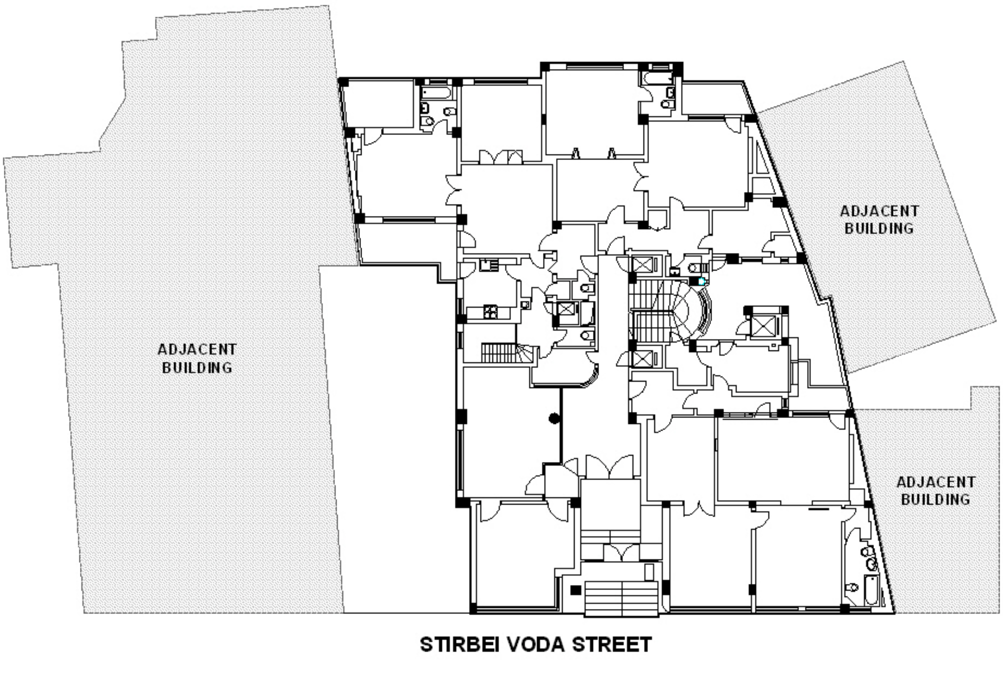


Figure 3: Ground floor layout of the building

Although in several axes well-defined RC frames can be identified, many beams are supported by other beams and there is a great number of columns that are connected with beams only in one direction, as can be seen in figure 5. In these circumstances it is not appropriate to register this construction as an RC framed building.

The structure of the building has 41 reinforced concrete columns with a large number of cross-sections adapted to the architectural requirements. These sections of the existing columns are much smaller than the corresponding sections of the columns designed according to the current requirements. The sections of the existing beams are also smaller than expected. For example there are several beams with cross-sections of only 150x350mm or 200x400mm.

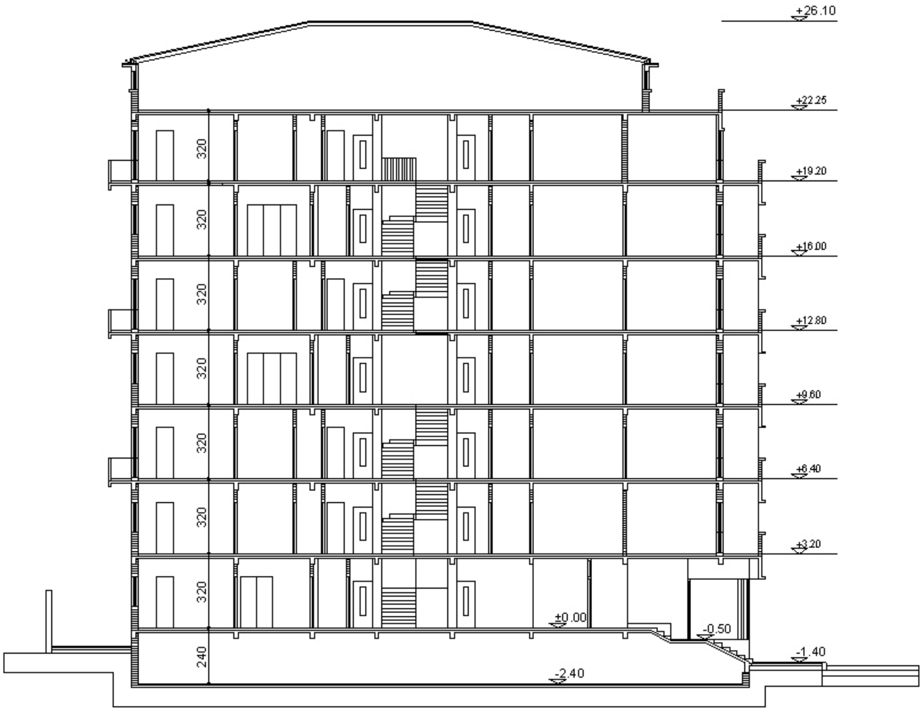


Figure 4: Elevation of the building

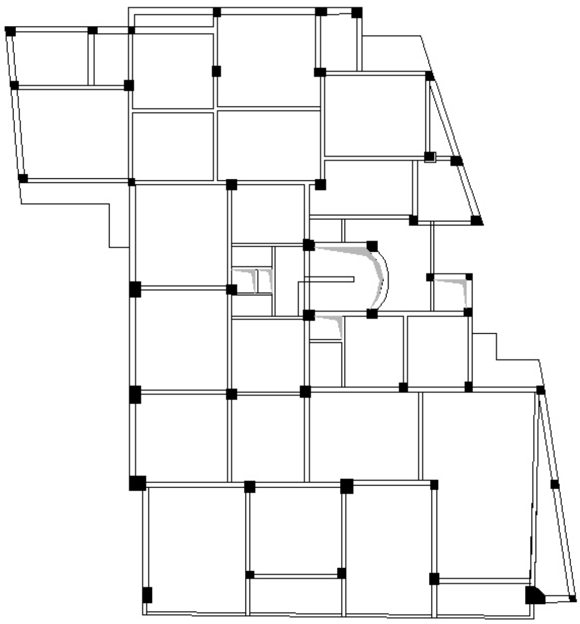


Figure 5: Arrangement of the beams and columns at the second story level

SEISMIC EVALUATION OF THE BUILDING

The residential building on Ştirbei Vodă Street at no. 20 is almost 72 years old. During its lifetime, it has experienced 5 important seismic events generated by Vrancea seismic source: 2 major earthquakes on November 10, 1940 (M=7.4 on Richter scale) and March 4, 1977 (M=7.2) and other 3 large earthquakes on August 30, 1986 (M=7.0) and two events on May 30 and May 31, 1990 (M=6.7 and 6.1).

Unfortunately, there are no written documents regarding the damages caused by 1940 earthquake, but the old inhabitants remembered that 3 columns were retrofitted along the first five stories by RC jacketing according to the proposal of Prof. Mihai Hanganu - the engineer that designed the structure of the building. Also, some cracks in the brick masonry walls were repaired with cement mortar.

After the destructive Vrancea earthquake on March 4, 1977, the building was severely damaged. The visual inspection performed immediately after the earthquake recorded that wide-opened cracks appeared in the columns from the western side of the building; many beams had cracks near the supports; the inclined RC ramps of the staircase had many cracks and a great number of the interior and exterior masonry walls had X-shaped or boundary cracks. Moreover, some parts of the southern exterior walls at the attic level and almost all the southern parapet above the 7th story had fallen down. During the summer of 1977, PROIECT BUCUREŞTI design institute prepared a retrofitting project for this heavily damaged building. It proposed the following retrofitting measures:

- RC jacketing of 12 interior columns; it aimed at increasing both the flexural and shear capacities of the damaged and weaker columns.
- Introducing a new, rigid exterior RC frame along the western side of the building; it aimed at transferring the high efforts that are induced by seismic action from the already damaged columns to the new exterior frame.
- RC shotcreting of some lightly damaged brick masonry walls.
- Replacing some heavily damaged walls with new ones made of reinforced brick masonry.
- Injecting with epoxy resin the cracks in the beams and inclined ramps of the staircase.

Unfortunately, due to the large number of buildings that needed to be retrofitted, the authorities decided that the retrofitting works should include only the injection of the cracks with epoxy resin; replacing the severely damaged walls and the fallen parapet; RC jacketing for 4 columns, but only to the first two stories and replacing the damaged finishes. The approved retrofitting started in the fall of 1977 and they were completed in the summer of 1978.

Next important earthquakes that followed in 1986 and 1990 caused moderate damage to the building. While some of the exterior cracks are still visible, the interior ones were hidden when the inhabitants replaced the interior finishes.

The most recent seismic evaluation of the building was performed in 2000 by PROIECT BUCUREŞTI and it included exterior and interior visual inspection of the building, non-destructive testing and seismic analysis. Unfortunately, in time, some of the original documents and drawing were lost or destroyed. Nowadays in the existing drawings there is no information about the longitudinal and shear reinforcement of the structural elements. Also the original engineering reports and calculations are missing. In consequence several columns and beams were subsequently scanned using a rebar detector in order to identify the existing reinforcement. Furthermore, the concrete cover of the columns was uncovered in three positions in order to verify the scanning results.

All major earthquakes in the recent history have repeatedly proved that proper selection of the load carrying system is essential to good performance under both vertical and horizontal loading. This observation is particularly appropriate in earthquake-resistant design where the intensity and orientation of loading are highly uncertain. Also, it has been observed that a properly selected structural system tends to be relatively forgiving of oversights in analysis, proportion, detail and construction. In consequence, buildings having simple, regular, and compact layouts incorporating a continuous and redundant lateral force resisting system tend to perform well in strong seismic motions, while complicated and non-uniform structural systems that introduce uncertainties in the analysis and detailing or that rely on effectively non-redundant load paths can lead to unanticipated and potentially undesirable structural behaviour.

For the selected building on Ştirbei Vodă Street, the structural system does not have a simple and direct path to transfer the gravity loads to foundations. Moreover, since the building was designed only for gravity loads, its structural system does not satisfy many other requirements of the modern seismic design:

- the constructive detailing used in 1934 required shorter anchorage and overlapping lengths for the reinforcing bars; besides all the longitudinal and transversal reinforcements are made of plain bars resulting in a weaker bond between the steel bars and the surrounding concrete;
- for the RC columns, the spacing between stirrups is too wide and there is almost no confinement action;
- for the RC beams, the area of the bottom reinforcement near the supports is too small and because of this the earthquake action caused flexural cracking at the bottom side of the beams.

Even from the beginning, the visual inspection of the building revealed a poor quality of the structural concrete. This observation was validated by non-destructive testing methods: both, Impact Hammer Test and Ultra Sonic Pulse Velocity Test showed that the concrete class is close to C8/10.

Due to the considerable stiffness and geometrical irregularities, a 3D model of the structural system was created. The results of the modal analysis showed that the structural system is very flexible. Because of this characteristic, the structure experienced large lateral displacements in the previous earthquakes that caused severe damages to the brick masonry walls and RC elements. As can be seen in figure 6, the first period of vibration of 1.71 sec. corresponds to a combination of translation along Stirbei Voda St. with torsion. The second mode of vibration that corresponds to almost pure translation perpendicular to the main façade has a vibration period of 1.55 sec.

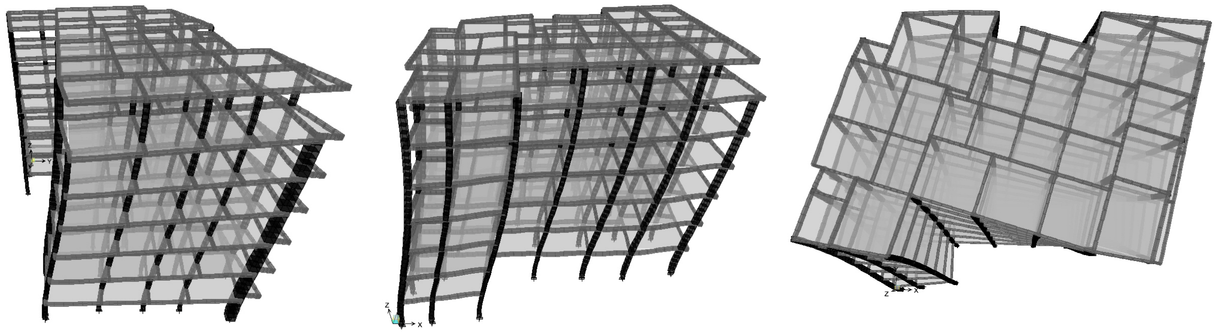


Figure 6: First three eigen-modes of the structural model

The seismic input was defined according to Romanian Code for Earthquake Resistant Design P100-92. The structure was classified as an ordinary residential building; therefore the importance factor is equal to α=1.00. The peak ground acceleration for Bucharest is 0.2g (with 63% exceedance probability in 50 years), therefore the seismic zone coefficient was taken as k_s=0.2. For Bucharest the design spectrum of acceleration is defined as in figure 8. So, for the fundamental vibration mode along Stirbei Voda St. the amplification factor is β_{OX}=2.29 and for the orthogonal direction it is equal to β_{OY}=2.44.

Due to poor detailing of the reinforcement, the inelastic deformation capacities of the existing structural elements are significantly smaller than those of the elements designed according to the modern requirements of the seismic codes. For this reason, the reduction coefficient due to the inelastic behaviour of the reinforced concrete structure was taken equal to ψ=0.40 instead of 0.20 as for newly designed RC framed buildings.

Finally, the modal participating mass ratios was taken as ε=0.85 even though, as can be seen in table 1, the modal participating mass ratio corresponding to first two modes of vibration are smaller than this value.

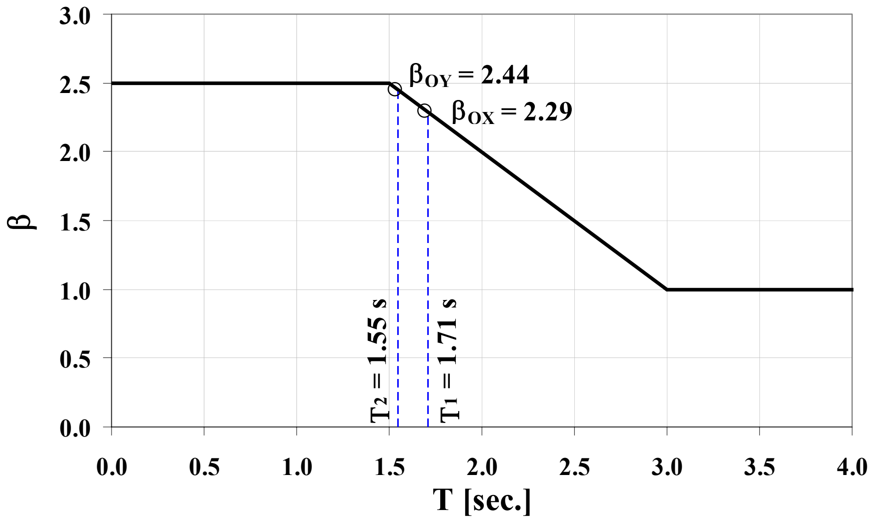


Figure 7: Design acceleration spectrum for Bucharest according to P100-92

Table 1: Modal participating mass ratios for the existing structure, [%]

Mode	Period	UX	UY	RZ	SumUX	SumUY	SumRZ
1	1.710	0.6	55.7	22.1	0.6	55.7	22.1
2	1.553	73.2	4.0	3.5	73.7	59.6	25.6
3	1.356	6.7	19.9	52.8	80.5	79.5	78.5
4	0.503	0.6	8.2	2.8	81.1	87.7	81.2
5	0.481	9.0	1.6	0.3	90.1	89.3	81.6
6	0.413	1.2	1.8	8.8	91.2	91.1	90.3

For these input data the base shear coefficient $c = \alpha{\cdot}k_s{\cdot}\beta{\cdot}\psi{\cdot}\varepsilon$ is equal to ***0.155*** for translation along Ştirbei Vodă Street and ***0.165*** on perpendicular direction.

The seismic forces for each horizontal diaphragm were determined according to a triangular distribution and they were graphically represented in figure 8. As a rough pre-evaluation the average shear stresses τ_{ave} in the existing RC columns were computed (table 2). The mean compressive stresses σ_0 in the RC columns are also presented in the same table.

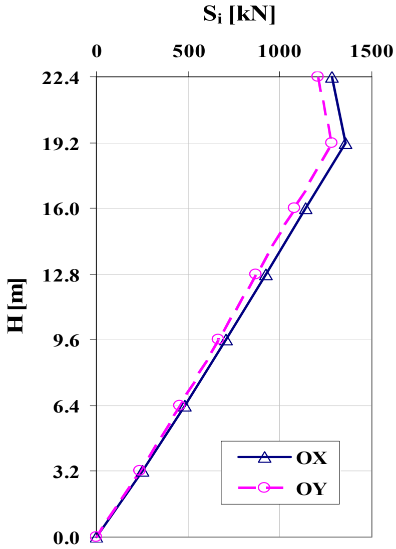


Figure. 8: Vertical distribution of seismic forces

Table 2: Average shear and compressive stresses in RC columns

Story	Earthquake load OY direction		Earthquake load OX direction		Gravity loads	
	Story shear [kN]	τ_{ave} [MPa]	Story shear [kN]	τ_{ave} [MPa]	Axial load [kN]	σ_0 [MPa]
ET6	1279	0.22	1205	0.21	4237	0.73
ET5	2639	0.39	2487	0.37	9459	1.41
ET4	3781	0.56	3565	0.53	14680	2.19
ET3	4705	0.70	4436	0.66	19901	2.97
ET2	5407	0.81	5100	0.76	25123	3.75
ET1	5887	0.88	5553	0.83	30344	4.52
P	6137	0.92	5789	0.86	35565	5.30

Given the poor quality of the concrete, its design shear strength is no more than 0.4 MPa. As we analyse the average shear stresses from table 2, we notice that in the first five stories these values are higher than the respective shear strength. At the ground floor level the ratios between the shear capacity of the concrete and the average shear stresses are 0.46 for seismic motion along Ştirbei Vodă Street and 0.43 for perpendicular direction.

As can be seen also from table 2, at least for first three stories, the values of the average axial compressive stress are relatively high since the design compressive strength of concrete is only 5.5 MPa. Therefore, at these levels, the inelastic deformation capacity of the columns is extremely small.

According to Romanian Code for Earthquake Resistant Design P100-92, the seismic evaluation index is defined as the ratio of the lateral capacity of the building to the demand seismic force. For the next level of the seismic evaluation, a more complex evaluation method was used. It took into account both the flexural and the shear failure mechanisms of the RC elements, but it neglected the lateral capacity of the masonry walls because they were heavily damaged in the previous earthquakes and probably their remaining lateral capacity is small and too difficult to be assessed. For these hypotheses, the seismic evaluation indexes are equal to R_{OX}=0.48 along Ştirbei Vodă Street and R_{OY}=0.46 on perpendicular direction. These results are in good agreement with the ones obtained in the preliminary evaluation.

Since the masonry walls are considered only as dead load, the second evaluation procedure showed that for several columns the axial compressive stresses are more than 1.3 times higher than the design compressive strength of the concrete. Even though, the brick masonry walls are actually carrying some part of the gravity loads, it is obvious that the columns' sections are too small and they need to be retrofitted.

The story drift ratio was computed according to the procedure from P100-92. From table 3 it can be easily noticed that the drift ratio values are extremely large even for a new designed building. In these circumstances it is clear that the brick masonry walls resisted also the lateral forces induced by earthquake motion and limited the lateral displacement of the building, but they were heavily damaged and, at this moment, their lateral capacity cannot be evaluated with a reasonable accuracy. Therefore, their contribution to the overall lateral capacity of the building cannot be taken into account in the analytical model.

Table 3: Horizontal displacements and drift ratios

Story	**UX [m]**	**DriftX [%]**	**UY [m]**	**DriftY [%]**
ET6	0.40	0.89	0.42	1.10
ET5	0.37	1.42	0.39	2.03
ET4	0.32	1.91	0.34	2.53
ET3	0.27	2.31	0.28	2.97
ET2	0.20	2.58	0.20	**3.17**
ET1	0.12	**2.59**	0.12	2.90
P	0.05	1.64	0.04	1.55

Considering the above described deficiencies and the results of the structural analyses, the building was ranked in the first class of seismic risk (most vulnerable). As a result, the building should be retrofitted.

SEISMIC RETROFITTING DESIGN

After several discussions between Japanese experts from JICA and Romanian engineers from NCSRR and P.B. with the owners' representative, the retrofitting objective is set to obtain a minimum seismic index of R_{min}=0.8 and to limit as much as possible the disturbance of the inhabitants. Another constrain imposed by the architects was not to modify the external appearance of the main façade facing Ştirbei Vodă Street.

In the beginning of the JICA retrofitting pilot project, several alternative retrofitting strategies were examined. Three different strategies were selected for further investigation. Two of these alternatives aimed to reduce the seismic response by isolating the building base from the ground excitation or by supplementing a new energy dissipating system. The third strategy was to upgrade the seismic performance of the building by strengthening the existing structure. In order to select the most appropriate solution, the advantages and disadvantages of each retrofitting strategy, as well as their technological difficulties, were carefully compared.

Base isolation retrofitting strategy has several major disadvantages:

- One significant disadvantage is that the rooms of the partially underground story are used as apartments. In case of the cheaper base isolation system that is introduced at the underground story level, these apartments must be removed and their owners should receive substantial compensation. Otherwise, a significantly more expensive base isolation system should be introduced under the existing foundations. It requires erecting a new foundation system under the existing building.

- Since the building is very flexible and the structural elements are characterized by a brittle behaviour, it is indispensable to strengthen the upper structure, which represents a significant discomfort for the inhabitants.
- Isolating the building from the ground excitation together with strengthening the upper structure is by far the most expensive retrofitting strategy and it is not economically feasible.
- Finally, the most important disadvantage is a technological one and it is related to the fact that on eastern and western sides of the building, the space between the analysed structure and the adjacent buildings is only 5 cm. So, there is no space to create the necessary gap that allows unrestrained movement between the isolated structure and the foundation system.

As already mentioned, the second strategy was to introduce new energy dissipating devices. Although this retrofitting solution has many acknowledged benefits, for the particular case of the selected building, it is rather difficult to be applied because:

- In order to obtain the desired seismic behaviour, this retrofitting strategy requires eliminating the irregularity or discontinuity of stiffness or strength distribution. This goal cannot be achieved since it is practically impossible to modify the configuration of the structural system. Furthermore, due to the irregular layout of the building, it is almost not possible to uniformly distribute the damping devices.
- Connecting the damping devices to the existing structure introduces high stresses at the interface with the existing RC elements. Many of these elements should be retrofitted in order to resist these high connection stresses.
- Even though the seismic response of the building is reduced by the energy dissipating devices, the earthquake motion will still induce lateral deformations and stresses larger than the respective capacities of the existing structural elements. Thus, a generalized structural retrofitting that strongly disturbs the daily life of the residents cannot be avoided.
- The total cost for this retrofitting strategy is relatively high.

The third alternative is almost similar with the “classical” retrofitting solution that aims to increase the lateral stiffness and strength of the building by adding new stiff reinforced concrete elements and increasing the axial capacity of the existing columns by steel jacketing. It has two main inconveniencies: it requires a relatively long construction period and, in order to retrofit the interior columns, it will inevitably disturb the inhabitants. On the other hand, this retrofitting solution has two main advantages: it is very familiar to ordinary construction companies and most important it is the cheapest one amongst all the selected strategies.

In conclusion: because, for the first two strategies that aimed to reduce the seismic response, the technical difficulties are almost impossible to be surpassed within an economically feasible range, the third alternative was selected for retrofitting the building on Ştirbei Vodă Street at no. 20. After extensive investigation of all the restrains imposed by the retrofitting objectives – maintaining the same architectural layout; disturbing the residents as less as possible; keeping the same “historical” appearance of the main façade and, of course, upgrading the lateral strength of the building in order to obtain a seismic index of at least R_{min}=0.8 – the following retrofitting solution was proposed and analysed in detail:

- Introducing new exterior reinforced concrete frames and structural walls on the eastern, western and northern façades (see figure 9), on the entire height of the building. All the dimensions and positions of the existing windows and technological openings will be kept exactly as they are. The concrete class for these new elements will be C25/30 and they will be properly connected to the existing structure through reliable post-installed chemical anchors.
- Replacing two interior brick masonry walls with new reinforced concrete shear walls from the basement up to the sixth story. The boundary RC columns will be retrofitted and they will be adequately fastened to the new reinforced concrete web.
- Retrofitting most of the interior RC columns by steel jacketing in order to increase their axial capacity. The columns will be retrofitted from the basement level up to the fourth story.
- Adding a new foundation system for the new exterior RC elements and enlarging the existing foundations under the new interior RC shear walls.

A 3D Finite Element (FE) model was used for seismic analyses of the retrofitted structure. The model included all RC structural elements: the exiting columns, beams and floor slabs, as well as the added reinforced concrete frames and shear walls.

If we compare the results of the modal analysis presented in table 4 with those from table 1, we notice that the retrofitted building is much stiffer than the existing structure and the first mode of vibration is less influenced by torsion, as can be also seen in figure 10.

Table 4: Modal participating mass ratios for the retrofitted structure

Mode	Period	UX	UY	RZ	SumUX	SumUY	SumRZ
1	0.760	0.5	63.6	7.2	0.5	63.6	7.2
2	0.563	62.8	2.6	9.1	63.4	66.2	16.2
3	0.476	9.3	5.4	55.9	72.7	71.6	72.1
4	0.189	0.6	14.3	1.8	73.3	85.9	73.9
5	0.158	9.1	2.2	3.4	82.4	88.1	77.3
6	0.111	7.3	0.3	11.5	89.7	88.4	88.9

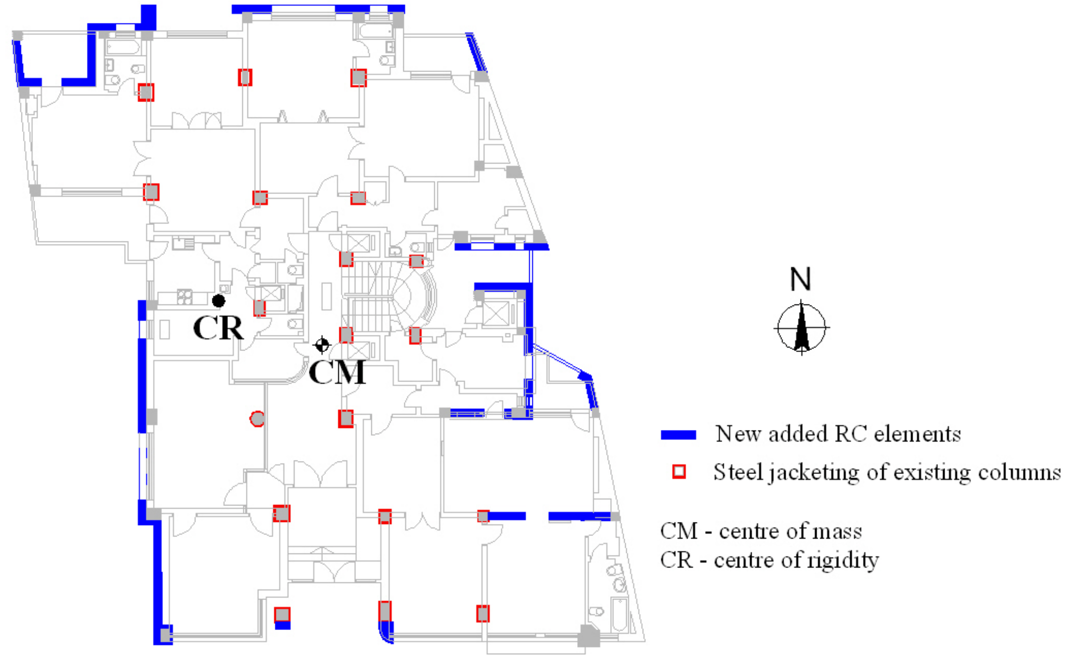


Figure 9: Plan of the strengthening solution

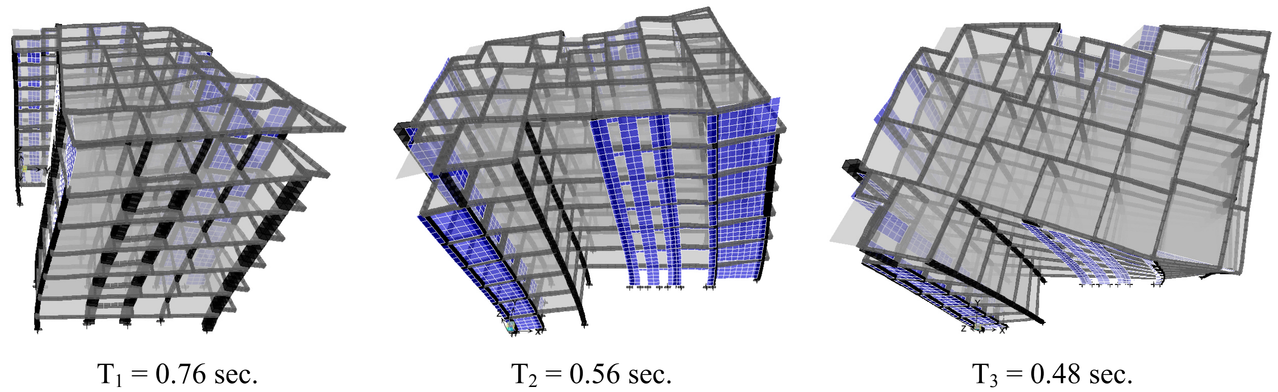


Figure 10: First three eigen-modes of the retrofitted structure

Since both periods of vibration corresponding to the main directions of the retrofitted building are smaller than 1.50 second, the spectral amplification factors have the same value equal to $\beta_{OX}=\beta_{OY}=2.50$.

According to Romanian code P100-92, for new RC shear wall buildings, the reduction coefficient due to the inelastic behaviour should be taken as ψ=0.25. But, giving the inappropriate detailing of existing structural elements, we assumed that the inelastic behaviour of the retrofitted building will not be as good as for a new designed building. For this reason, a higher value of ψ=0.30 was adopted for the reduction factor.

So, for the retrofitted structural system the base shear coefficient is equal to c=***0.127***.

Table 5 presents the inter-story drift ratios induced by seismic design forces acting along the Ştirbei Vodă Street (OY axis) and on perpendicular direction (OX axis). It can be easily noticed that the inter-story drifts have significantly decreased. However, in contrast with the same results corresponding to the existing structure, the drift ratio values on OY direction are around two times larger than for OX direction. This undesired phenomenon is the direct result of the architectural constraint that forbids modifying the existing appearance of the main façade. Because of this, the new added RC elements have an unbalanced distribution which produces a large eccentricity between mass and stiffness centres, as can be seen in figure 10.

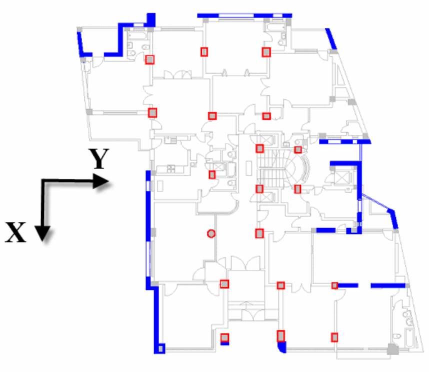
Table 5: Horizontal displacements and drift ratios for retrofitted structure

Story	UX [m]	DriftX [%]	UY [m]	DriftY [%]
ET6	0.06	0.28	0.11	0.63
ET5	0.05	0.29	0.09	0.68
ET4	0.04	0.32	0.07	**0.72**
ET3	0.03	**0.35**	0.06	**0.72**
ET2	0.02	**0.35**	0.04	0.66
ET1	0.01	0.32	0.02	0.52
P	0.00	0.18	0.01	0.25

Since in P100-92, there are no special provisions for buildings with large eccentricity between mass and stiffness centre, the Japanese experts proposed to examine the specific requirements from both the American Prestandard FEMA 356 of 2000 and the Japanese Building Standard Law revised in 2001. After a cautious investigation, it was decided to apply the more conservative approach. According to the Japanese provisions, if the eccentricity ratio is greater than 0.30 the seismic forces should be multiplied by an amplification factor equal to γ=1.50. So, the modified base shear coefficient becomes c = ***0.19*** for a seismic force demand of S_{nec} = 6760 kN.

One of the most important objectives in earthquake resistant design is to obtain a proper flexural yielding mechanism. Therefore, the newly added RC elements, as well as the steel jacketing retrofitting of the existing columns were carefully designed in order to avoid any type of brittle failure. The computed lateral capacities associated to the flexural yielding mechanisms of the entire retrofitted building and the corresponding seismic indexes are concisely presented in table 6.

Table 6. Horizontal capacity and the corresponding seismic index of the retrofitted structure

<table>
<tr><th>Global axes orientation</th><th>Seismic loading on</th><th>Lateral capacity S_{cap} [kN]</th><th>Seismic index $R = S_{cap} / S_{nec}$</th></tr>
<tr><td rowspan="4">
</td><td>positive OX direction</td><td>7409</td><td>1.10</td></tr>
<tr><td>negative OX direction</td><td>6771</td><td>1.00</td></tr>
<tr><td>positive OY direction</td><td>5768</td><td>0.85</td></tr>
<tr><td>negative OY direction</td><td>6036</td><td>0.89</td></tr>
</table>

Conventional earthquake resistant design of buildings relies on ductility of structural elements to enable redistribution of internal efforts and dissipation of seismic energy. Observations have shown repeatedly the necessity of attention to proportioning, in order to ensure that inelastic deformation occurs at appropriate locations, and detailing, in order to provide adequate ductility in these locations that yield. For this reason, the design team paid a special attention to find the suitable constructive details for the retrofitting project. The Japanese expertise in the field of seismic retrofitting was highly appreciated since it has always provided simple, yet reliable retrofitting details and construction methods. Some of these details are subsequently presented. While, figure 11 presents the connection details between the existing RC frame and the added interior RC walls, the constructive details for steel plate jacketing are presented in figure 12.

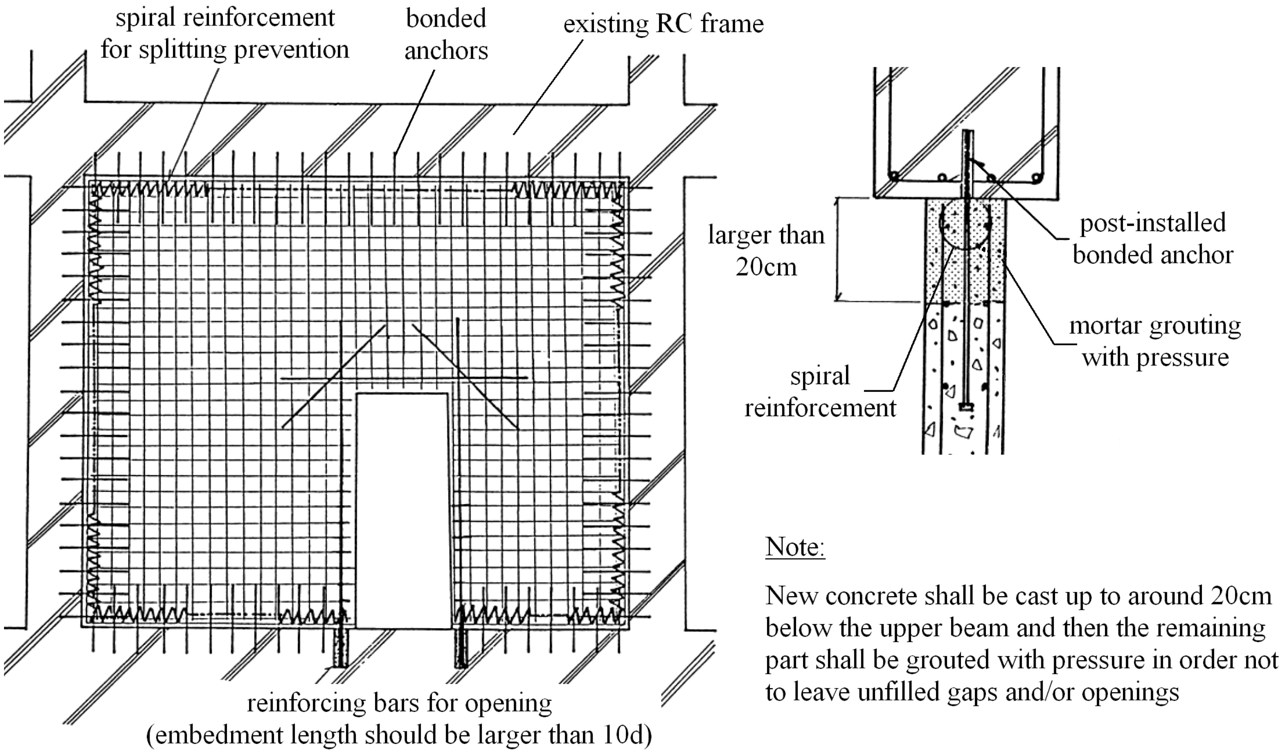


Figure 11: Bar arrangement for a new added interior RC shear wall

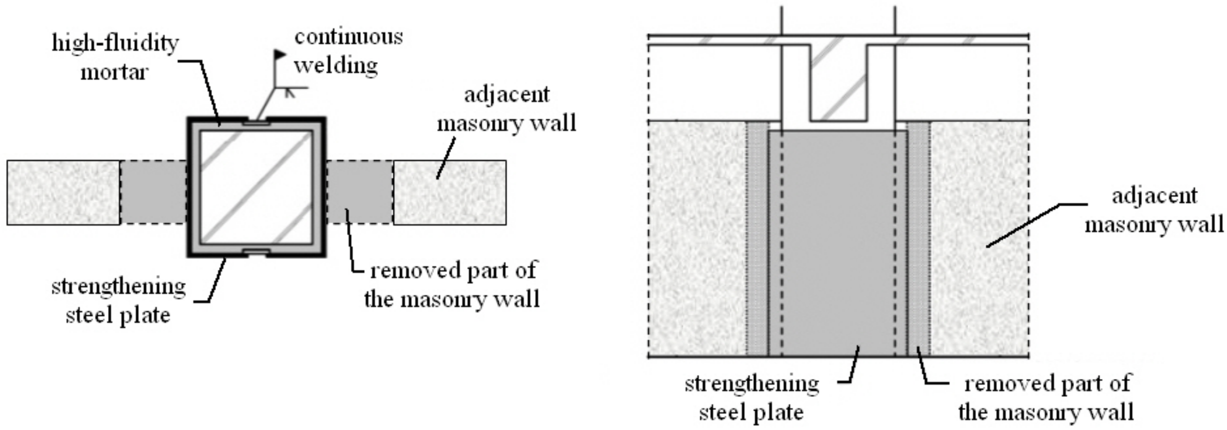
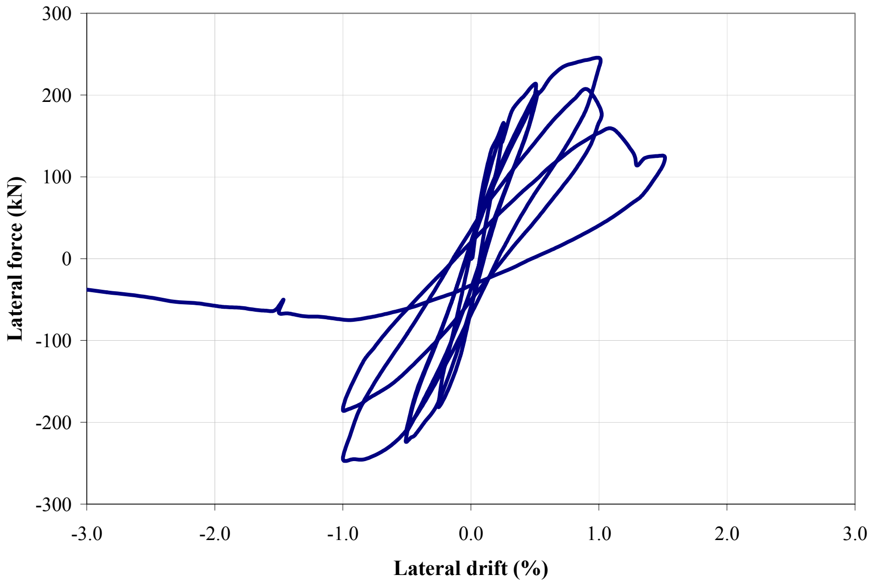
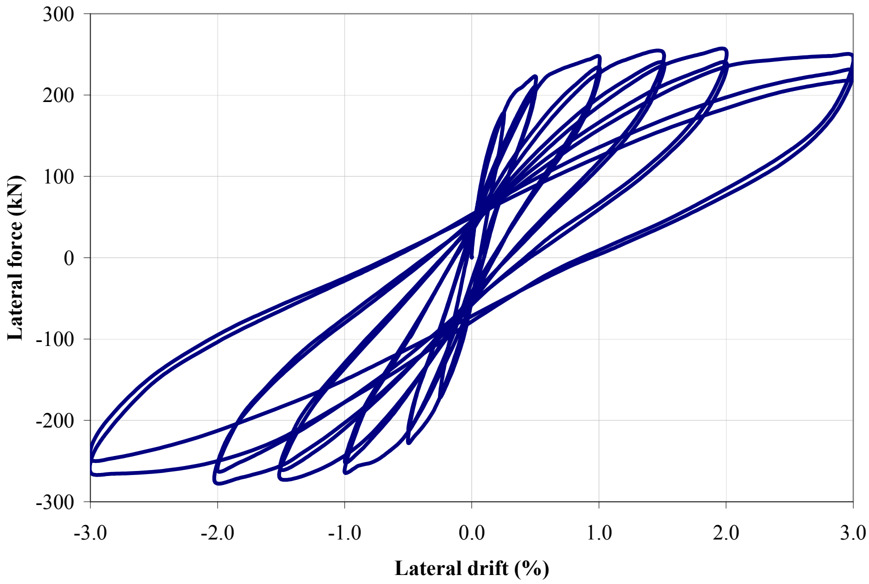


Figure 12: Proposed steel plate strengthening of the existing columns with attached partition walls

According to numerous experimental studies, the ductility of the RC columns with poor transverse reinforcement is significantly increased by steel plate jacketing. This was also proved by the experimental tests performed in the structural testing laboratory of the National Center for Seismic Risk Reduction. So, figure 13 presents the differences between the hysteretic curves corresponding to an old type pre-1940 column.



a. As-built RC column



b. Retrofitted column by steel jacketing

Figure 13: Hysteretic curves before and after steel plate strengthening for an old type pre-1940 RC column

CONCLUSIONS

The main goals of the retrofitting pilot projects promoted by JICA and NCSRR are to encourage new retrofitting techniques and to persuade the citizens from Bucharest to accept and support the retrofitting works for the existing vulnerable buildings. One of these pilot projects aimed to provide a feasible retrofitting solution for an historical pre-1940 building located on Ştirbei Vodă Street, in centre of Bucharest.

The seismic performance of the building was assessed based on several site visits, available structural drawings, non-destructive material tests as well as a comprehensive 3D finite element model. The seismic evaluation showed that the structural system has some major deficiencies, mostly because the building was designed only for gravity loads. The structural analyses revealed that the lateral capacity of the building is much smaller than the seismic demand. For these reasons, it was concluded that the building should be retrofitted.

Three different retrofitting strategies were extensively analyzed. Two of them aimed to reduce the seismic response of the building by isolating the building base from the ground excitation or by supplementing new energy dissipating devices. The third alternative was to strengthen the existing structure by adding new reinforced concrete elements and increasing the axial capacity of the existing columns through steel jacketing. Because, for the first two strategies, the technological difficulties are almost impossible to be surpassed within an economically feasible range, the third retrofitting alternative was selected. Then an iterative process of retrofitting design was performed in order to attain the desired seismic performance.

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