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SEISMIC UPGRADING OF TELECOMMUNICATION CENTER IN SKOPJE, NORTH MACEDONIA

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Abstract

This paper presents detail analysis of the stability and safety of the existing structural system of the Telecommunication Center facility in Skopje, to define the possibilities and conditions to meet the technical standard Telecommunication Industry Association TIA-942-A, which is requirement for the eventual change of the building function. Considering that it is a building of the high importance in terms of its function, built more than fifty years ago, it was necessary to examine the need for additional structural strengthening and initial cost estimation. The necessity of structural interventions to meet the required technical standards were defined based on (i) input data from limited in situ technical investigations, (ii) assessment of the seismic potential of the site and (iii) analysis of the load-bearing structure for the anticipated imposed loads while simultaneously providing the required structural stability and safety for gravity and earthquake actions according to the existing seismic code in North Macedonia. Findings from the performed analysis enforce the need for both global and local structural strengthening.

Keywords: telecommunication center, nondestructive testing, bearing and deformation capacity, time-history analysis, seismic strengthening.

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1 INTRODUCTION

The complex of the Telecommunications Center in Skopje, North Macedonia was constructed during the seventies of the last century as a representative of the reinforced concrete buildings typical for the post-earthquake communist period, (fig.1). This creation of the Macedonian architect Janko Konstantinov was exposed at the 2018 New York MoMA exhibition on the topic "Concrete Utopia - Architecture in Yugoslavia 1948-1980", as a representative from North Macedonia. Like other similar buildings from this period, the construction of the building is characterized by bold structural solutions, large spans and storey heights, followed by robust dimensions of the structural elements and the inevitable facade made of popularly called “nature-concrete”.

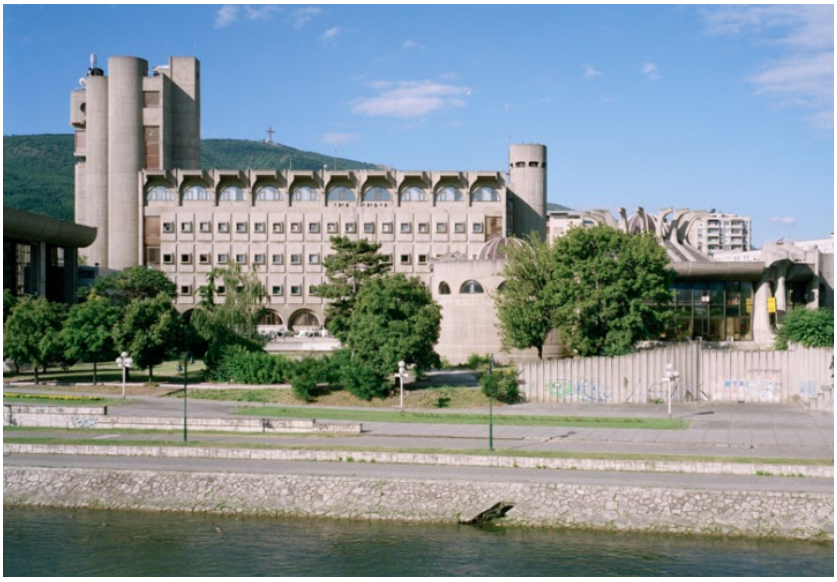

Figure 1: The complex of Telecommunication Center in Skopje, North Macedonia.

To define the possibilities and conditions to meet the technical criteria for level TIER III defined by the standard TIA-942-A [1], which was set as a need for the possible future function of the facility, analysis of the stability and safety of the existing structural system of the Telecommunication Center facility in Skopje was carried out. Considering that it is a building of the high importance in terms of its function, built more than fifty years ago, it was necessary to evaluate its existing stability. Accordingly, the purpose of the analysis is to examine the need for additional strengthening of load-bearing structural elements in order to perform an initial cost estimation. For this purpose, in accordance with the multidisciplinary integrated approach developed in Institute of Earthquake Engineering and Engineering Seismology, IZIIS-Skopje [2], the following activities were carried out:

1. Assessment of the seismic potential of the site
2. On-site building inspection and investigation for providing relevant input structural data for further analysis, which encompasses (i) review of the available technical documentation, (ii) detailed inspection of the building and identification of its main structural system, (iii) definition of quality of built-in materials by non-destructive testing of structural elements and (iv) determination of dynamic characteristics of the structure by experimental in-situ testing using ambient vibration technique
3. Analysis of stability of existing building for gravity and seismic loading, consisting of (i) 3D static and equivalent seismic FE structural analysis for the verified structural

system and built-in materials, (ii) analysis of the bearing and deformation capacity of the existing structural system with identified quality and quantity of built-in materials, and (iii) dynamic response including nonlinear time history analyses of building structure for earthquakes of different intensity and frequency content expected on the considered site.

The knowledge gained through the above investigations and analysis has been used to define preliminary technical solution for strengthening of individual structural elements with a cost estimation for its realization, considering the current conditions of the market and the specifics arising from the method of execution, thus, to define the justification of entering such a process. The actual cost estimation is not subject of this paper and can be find in [3,4].

2 ASSESSMENT OF THE SEISMIC SITE POTENTIAL AND IDENTIFICATION OF THE LOAD-BEARING STRUCTURE

2.1 Assessment of the seismic site potential

Considering the given limited time framework, the necessary field investigation to define the seismic potential of the particular site have not been done, however, data and results of detailed research for several locations near the existing facility were considered. The seismic potential was assessed based on the results of the investigated location of the Mother Teresa monument, which is the closest to the location of the Telecommunication Center, (fig 2).

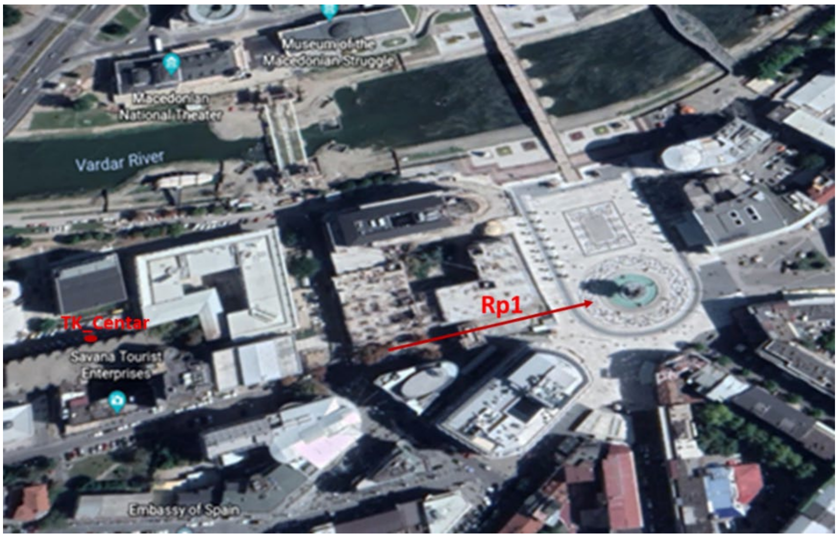

Figure 2: Telecommunication Center in relation to the location of the Mother Teresa monument.

The level of acceptable seismic risk for TK-Center was defined by the standard TIA-942-A. The expected average maximum accelerations at the bedrock for near and far source earthquakes were estimated based on the seismic hazard analysis carried out for the location of the Mother Teresa monument. The upper limit of the average maximum accelerations of the bedrock for a return period of 475 years is 0.29g and at the foundation level is 0.36g, due to the estimated dynamic amplification factor. For time-history analysis of the seismic response of the structure the following set of recorded earthquakes have been selected:

- ACC1: Ulcinj (Albatros) E-W, recorded during the Montenegro earthquake of 15.04.1979 with a magnitude of M=7.0.
- ACC2: El Centro N-S, USA, 1940, with magnitude M=6.7.
- ACC3: Robic N-S, recorded during the Furlania (Italy) earthquake of 15.09.1976 with magnitude M=6.1.

- ACC4: Petrovac (Oliva) N-S, recorded during the Montenegro earthquake of 15.04.1979 with a magnitude of M=7.0
- ACC5: Bar, N-S, recorded during the Montenegro earthquake of 15.04.1979 with a magnitude of M=7.0.

2.2 Identification of the load-bearing system

Overview of available technical documentation: The complex of the Telecommunications Center - Macedonian Post Office in Skopje was built in three phases in the period between 1974 and 1982, (Telecommunications Center, Administration and Counter Hall, fig. 3). The subject of analysis is the Telecommunication Center (fig. 3 upper), which together with the tower, from which it is separated by an expansion joint, constitutes the first phase of the construction of this complex.

From the available (uncompleted) technical documentation, the following was observed [3,4]:

- The structural system represents a moment resisting frame consisted of reinforced concrete columns and beams and reinforced concrete floor slabs
- The design calculations are carried out for a live load of 8 kN/m^2
- Seismic analysis is provided as for building with high importance (I category) in a zone with seismicity of IX+ degrees according to MCS, corresponding to the first seismic code in the country, (Temporary Technical Provisions for Building in Seismic Regions, Official Gazette SFRJ 39/64), valid for the time when the building was designed
- The proportioning of the load-bearing elements is carried out for concrete quality class of 30MPa (MB30) and plain reinforcement GA 240/360

Figure 3: The Macedonian Telecommunication Complex, Telecommunication center (upper), Administration (lower left), Counter Hall (lower right).

On-site investigations of the structural elements: In order to obtain/confirm the structural system and the existing state of bearing elements, additional technical investigations and control measurements of the geometry of structural elements were performed in-situ. Considering the daily function of the building, non-destructive testing (categorized as an informative method), followed by a minimum number of semi-destructive investigations for identification of the built-in reinforcement in the selected structural elements were carried out. As a result, form-work plans of the individual storeys and characteristic cross-sections of the building have been prepared, (fig. 4).

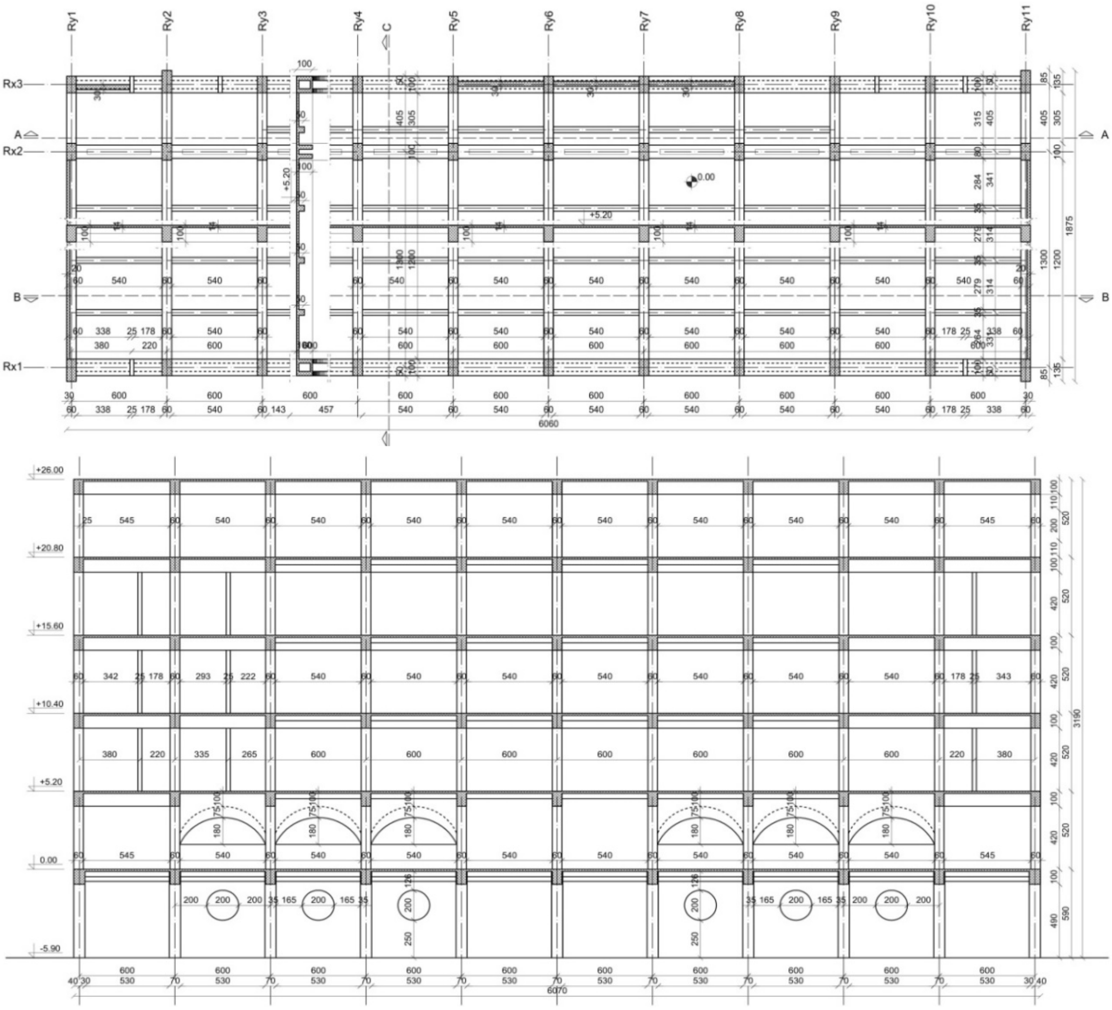

Figure 4: Layout of the ground floor (upper) and longitudinal cross-section (lower) of the building.

Profometer

Digi-Schmid

Tromino

Figure 5: Nondestructive testing equipment.

The NDT measurements have been done by the following modern equipment for determination of concrete grade and steel rebar disposition in the bearing elements, (fig. 5):

- PROFOMETER 5+ (V2.3.0, 55.60312) (rebar detection system) for identifying the presence of built-in reinforcement, giving information about the cover layer and approximate diameter of the detected metal,
- DIGI-SCHMID 2000, (Modell ND) mechanical device designed to define the compressive strength of a material (primarily concrete),
- TROMINO® - portable ultra-mobile seismometers for definition of structural fundamental dynamic characteristic, (natural frequencies).

Majority of measurements were conducted and recorded in the basement and on the ground floor, while for the upper floors the critical positions were only confirmed. A total of 24 measurements were performed on selected available reinforced concrete elements, (fig. 6).

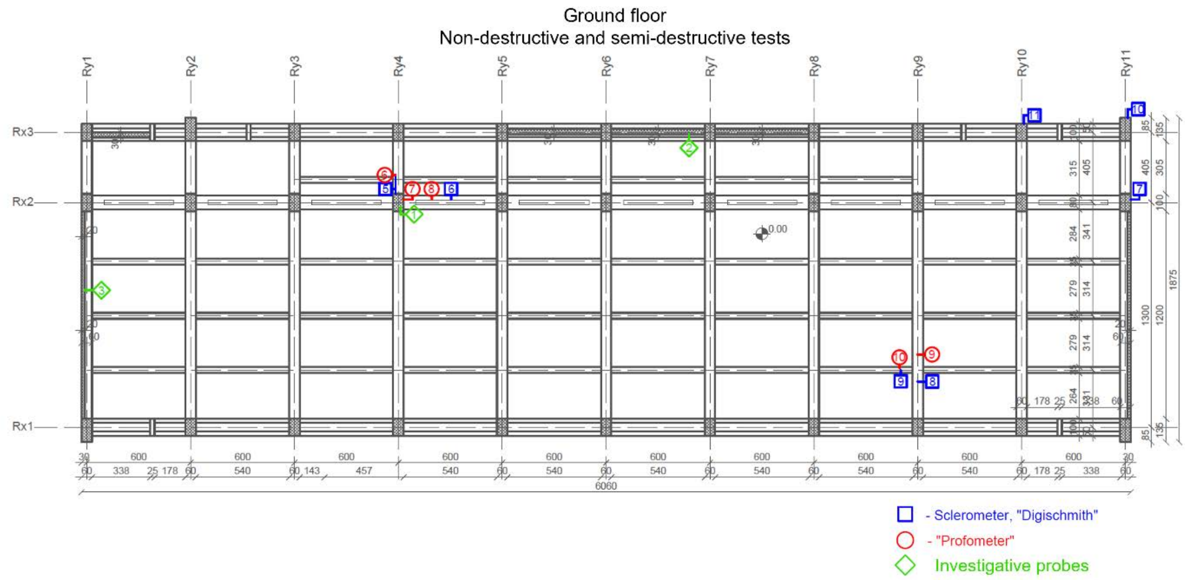

Figure 6: NDT measurement points on ground floor

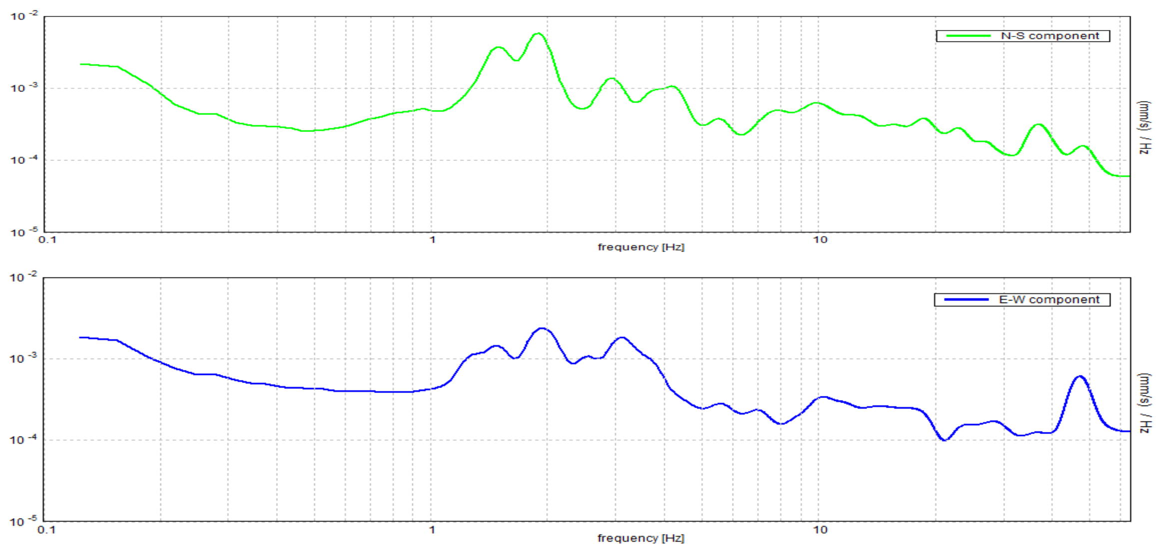

Figure 7: Natural frequency in N-S and E-W direction.

The dynamic characteristics of the sturture were determined using the ambient vibration method, non-destructive "in situ" method based on measurement of structural vibrations caused by ambient forces. The experimentally obtianed dynamic characteristics is of particular importance when calibrating the mathematical models, for a more relevant analysis during the assessment of structural seismic stability, (fig. 7).

To confirm the exact rebar diameter minimum number of destructive investigation probes, were carriod out (fig. 8):

Figure 8: Column rebar diameter measurement (investigation probe 1)

Summary on building structure investigation: The overall site investigations resulted in identification of the main load-bearing structure of the building in its current state, (fig. 4). The principal structural system represents a reinforced concrete frame system. Selected important information are listed below:

- The compressive strength of the built-in concrete in the columns is higher than 40MPa (MB40), while in the shear walls, beams, and floor structure is 30MPa, (MB 30).
- Reinforcement configuration in beams, columns, and floor structures is defined.
- The distance of the shear reinforcement in the columns is 20cm (near the joints) an 40cm at the midspan and in the beams is 15/30cm, accordingly.
- The natural periods of the structure in both directions are $T^{N\text{-}S}$=0.55s, $T^{E\text{-}W}$=0.52s.
- The columns are reinforced by longitudinal bars with 40 mm diameter and stirrups with 12mm diameter placed on 20-40 cm, (fig. 8).
- Floor plate thickness is d=14cm.

During the visual inspection of the building, no structural damage was found, no settlements or structural distresses, nor damage or repairs due to previous earthquakes were observed. The general conclusion is that the structure has been designed and analysed according to regulations (gravity and seismic codes) at the time of construction on the building, built with exceptional quality and very well maintened.

However, considering that the building was built before the adoption of the current seismic code PIOVS'81 [5], which prescribes additional criteria that the facilitiy should satisfy, it is necessary to perform an analysis of the existing structure for the anticipated imposed gravity and seismic loads.

3 ANALYSIS OF EXISTING STRUCTURE ACCORDING TO TIA-942-A STANDARD

To assess the need for structural interventions in order to satisfy the required technical criteria for level TIER III of the TIA-942-A standard, the following structural analyses were performed for gravity and seismic actions: (1) elastic 3D analysis of a mathematical model by application of the finite element method, (2) Analysis of the bearing and deformation capacity, and (3) Nonlinear time-history analysis for the seismic parameters defined in Section 2.1 [3, 4].

3.1 Linear-elastic 3D Finite Element Analysis for gravity and seismic actions

For the mathematical model of the defined structural system, (fig. 9), static and lateral force analysis for shear base coefficient equivalent to 15% of the total building weight -design seismic force (according to PIVOS'81) have been performed applying the RadImpex TOWER 7.0 [6]. Gravitational load analysis was carried out taken into consideration the crucial technical requirements for TIA-942-A - TIER III, as follows:

- additional live load of 12 kN/m^2 on the third and fourth floor structure
- additional live load of 2.4 kN/m^2, suspended on the bottom side of the third and fourth floor slab.

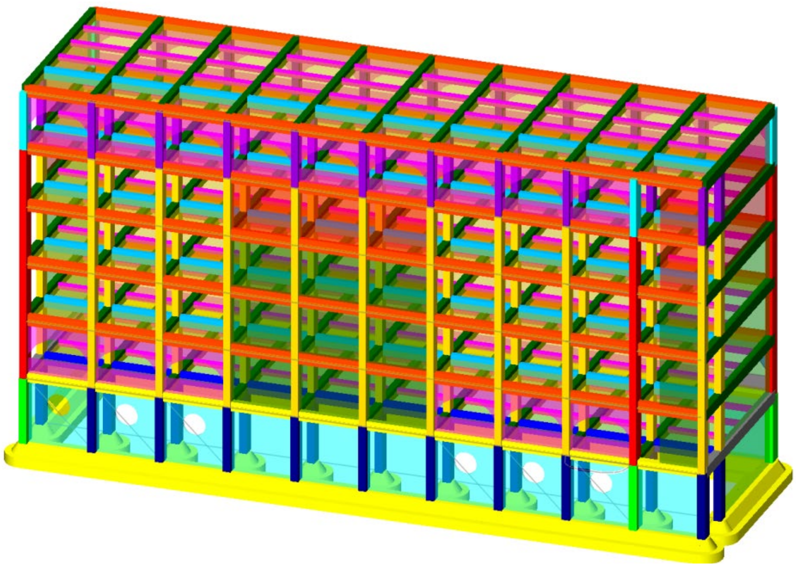

Figure 9: 3D Finite element mathematical model.

The building structure has six storeys (basement, ground floor and four stories), with a total weight of 147759 kN and a total seismic force of 21722 kN. The obtained natural periods were $T^{N\text{-}S}$=0.582s, $T^{E\text{-}W}$=0.525s, and they are very similar to the experimentally measured ones, (fig.7). The total horizontal displacement of 1.76 cm and 2.01 cm are obtained for transversal and longitudinal direction respectfully, which are less than the maximum allowed (4.33cm), according to the PIOVS'81.

The obtained axial stresses in the columns meet the criteria against brittle failure from vertical loads prescribed in PIOVS'81, (σ_0^{max}=6.90 MPa < $\sigma_0^{allowed}$=7.175 MPa). However, certain requirements for reinforcement detailing of load-bearing elements to avoid brittle failure due to seismic actions are not fully met. This especially applies to the distance of shear reinforcement. The distance of the stirrups in the columns is 20cm near the joints and 40cm in the midsection, which is in accordance with the 1964 seismic code, but not completely in accordance with the PIOVS'81 that prescribes 15 cm in the midsection and 7.5 cm near to the joints.

As expected, in most cases the built-in reinforcement in the beams and slabs on the third and fourth storey is less than the required one, which is due to the additional live loading according to the TIA-942-A standard. At the floor levels where there are no such live loads, the built-in reinforcement corresponds to the required one obtained from the analysis.

Consequently, the elastic analysis of the structure carried out according to the current seismic code in the country, shown that the building exposed to the additional loads according to the TIA-942-A - TIER III requirements and seismic actions does not meet the overall safety criteria.

3.2 Bearing and Deformation Capacity

Bearing and deformation capacity at element level (Qi-δi relationship) is calculated by inhouse developed software [7]. The cumulative storey Q-δ relationship represents sum of the bearing and deformation capacity of each element, at that story level and considering direction. The storey ductility capacity is defined as ratio between maximum story displacement (δ_u) and displacement at the yield point (δ_y), i.e., $\mu = \delta_u / \delta_y$.

Bearing capacity of the ground floor, selected as most critical one, is presented on fig.10 and it is larger than the required design seismic force (see 3.1).

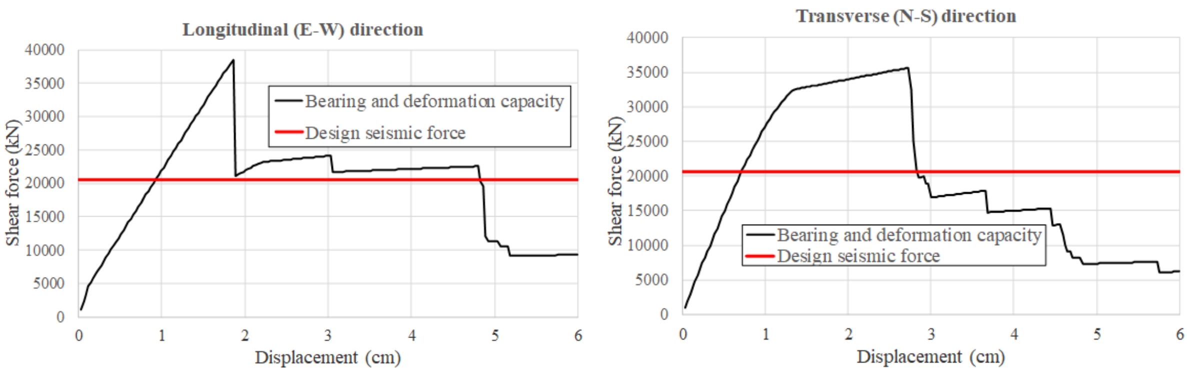

Figure 10: Bearing and deformation capacity for ground floor.

3.3 Nonlinear time-history analysis

Nonlinear time-history analysis has been performed by inhouse developed software [7], on the lump-mass structural model that assumes concentration of structural characteristics at story level and using the Q-δ bilinear relationship obtained from the analysis of bearing and deformation capacity, (fig.10).

The results from the dynamic analysis are used for evaluation of the structural performance in nonlinear domain, i.e., verification of the capacity of the structure in respect to the demand for different earthquakes and different levels of input acceleration, as follows: (1) displacement capacity of the structure (δu) in respect to the displacement demand due to an earthquake (δ_{RQ}); (2) displacement ductility capacity of the structure (μ) in respect to the ductility demand ($\mu_{RQ} = \delta_{RQ}/\delta y$).

In accordance with the above:

- if $\delta_{RQ} \leq \delta y$, i.e., $\mu_{RQ} \leq 1$ – the system is in the elastic state,
- if $\delta y \leq \delta_{RQ} \leq \delta u$, i.e., $\mu \geq \mu_{RQ} > 1$ - the system is in the nonlinear range,
- if $\delta_{RQ} > \delta u$, i.e., $\mu_{RQ} > \mu$ - the system experiences deep nonlinearity and possible failure.

A nonlinear time-history analysis has been performed for the structure according to the described procedure and applying selected earthquakes (Section 2.1) with gradual increase of the intensity, (0.24g, 0.27g, 0.30g, 0.33g and 0.36g). Fig.11 illustrates storey displacements required by the earthquakes with selected intensity of 0.30g (design earthquake) and 0.36g (maximum expected earthquake), compared to the capacity displacement in points "Y" and "U". Ductility capacity of the ground floor level versus ductility demand for each of the earthquakes is presented on fig. 12.

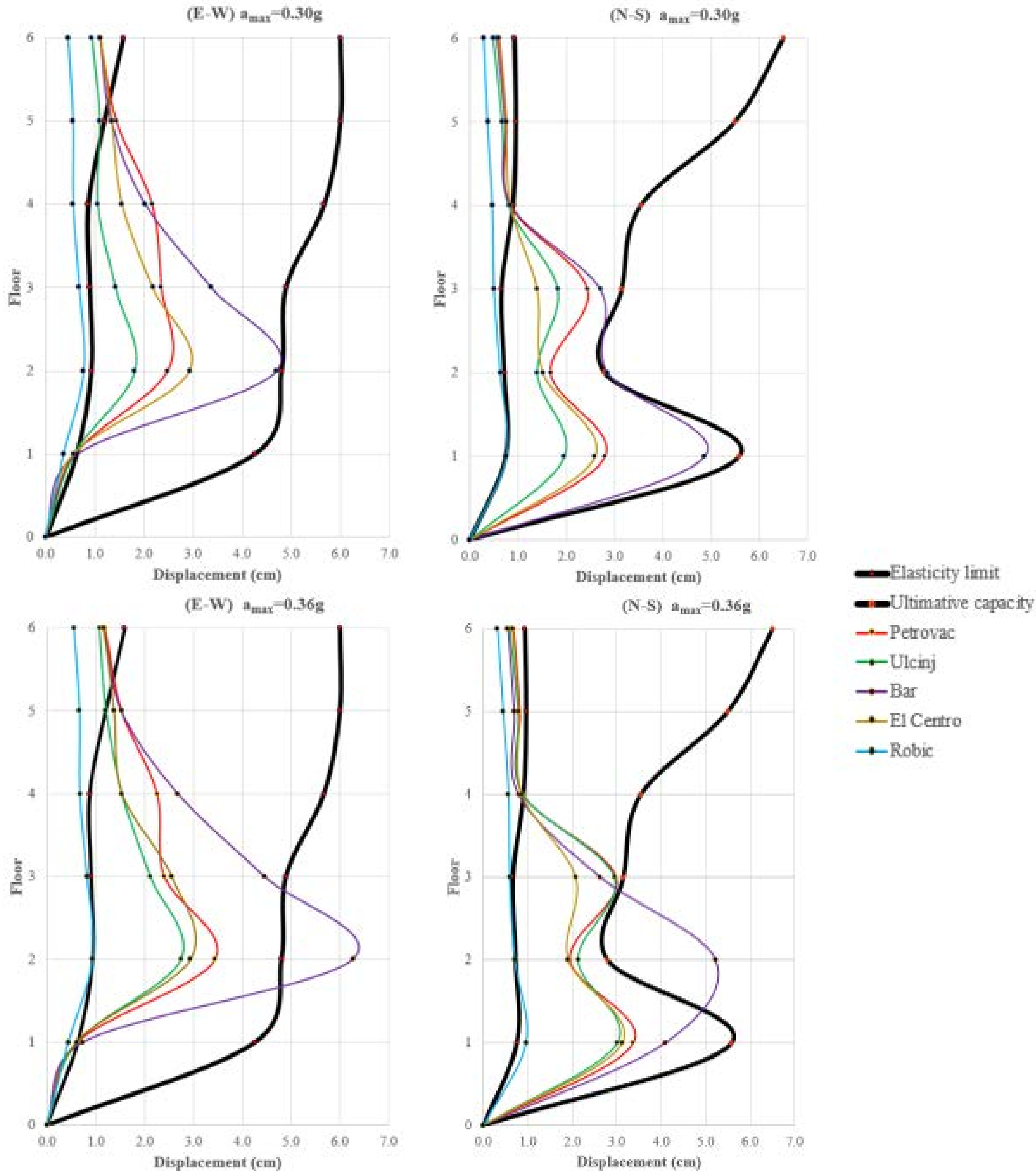

Figure 11: Required displacements versus deformation capacity for longitudinal (E-W) direction (left) and transverse (N-W) direction (right).

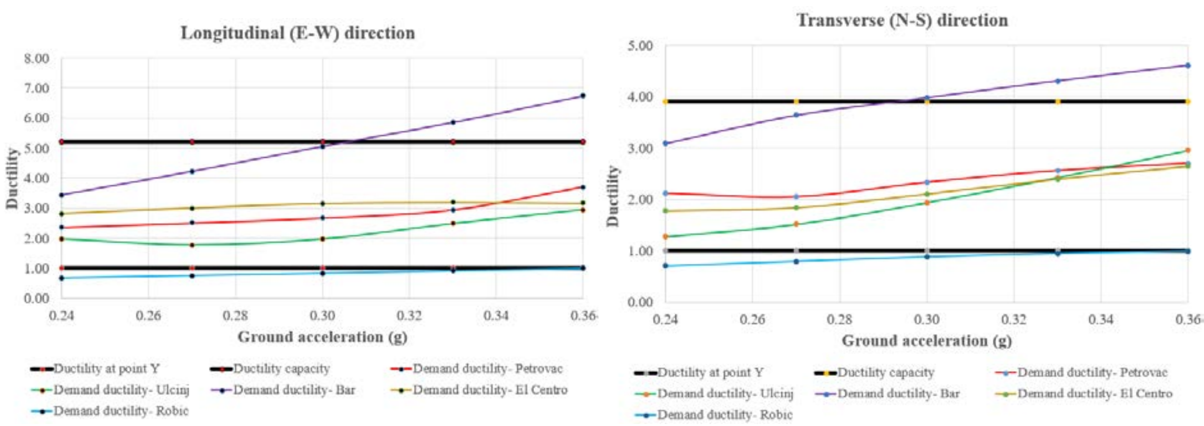

Figure 12: Comparison of demand ductility to ductility capacity for ground floor for longitudinal (E-W) direction (left) and transverse (N-S) direction (right).

3.4 Summary on the structural stability and safety

As a result of the overall investigations and analyses shown in the previous Sections, it can be concluded that the structural system was originally designed for gravitational and seismic actions, (according to the seismic code at the time of design) and was built and maintained with high quality.

However, it does not satisfy completely the requirements according to the current seismic code in the country PIOVS'81 and the new requirements according to the standard TIA-942-A - TIER III. This is primarily due to the additional criteria related to the importance class of the building and rules for reinforcing detailing of the load-bearing elements to avoid brittle failure during seismic actions.

The vertical load-bearing elements (columns and shear walls) possess the necessary bearing capacity for seismic actions defined according to PIOVS'81, (fig. 10), however the time-history analysis shows that the ground floor does not have sufficient deformation capacity and ductility for maximum expected earthquake actions (figs. 11 and 12). Also, the horizontal RC elements on 3rd and 4th floor have not enough bearing capacity due to additional exploitation loads according to TIER III.

Finally, according to the structural analysis results, there is a need for global and local strengthening of the building's structure to satisfy completely the current seismic code PIOVS'81 and the specific TIER III criteria.

4 PRELIMINARY SOLUTION FOR SEISMIC RETROFITTING OF THE EXISTING STRUCTURE

The proposed solution for seismic retrofitting, based on building's specifics and the conditions for execution, consists of:

1. Strengthening of the reinforced concrete slab on the third and fourth floor due to the additional imposed load, according to TIA-942-A - TIER III standard. It anticipates new secondary steel beams, (fig.13, left). The beam is formed by inserting steel profiles below and above the slab, interconnected one to another and coupled with the existing slab.

2. Strengthening of the longitudinal main reinforced concrete beams on the third and fourth floor due to the additional imposed load, according to the TIA-942-A - TIER III standard. It anticipates inserting steel profiles to reach the required bending moment capacity in the mid-span, (fig.13, right).

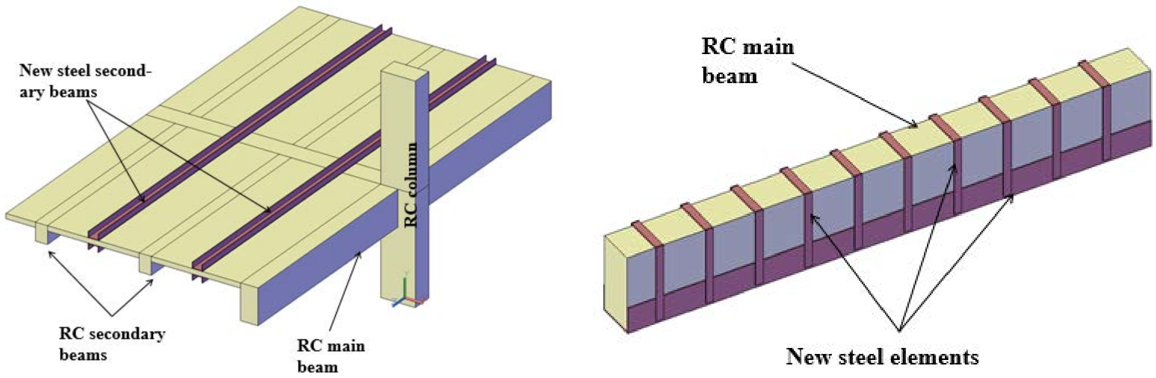

Figure 13: Strengthening of the 3rd and 4th storey: slab (left) and main RC beam (right)

3. Strengthening of reinforced concrete beam-column joints due to seismic action, which anticipates interconnected steel strips, mounted on the top and the bottom edge of the beam (fig. 14).

4. Strengthening of the reinforced concrete columns due to the additional imposed load and seismic action, which anticipates strapping with steel strips which are interconnected and connected to the strips on the strengthened beams, (fig.14).

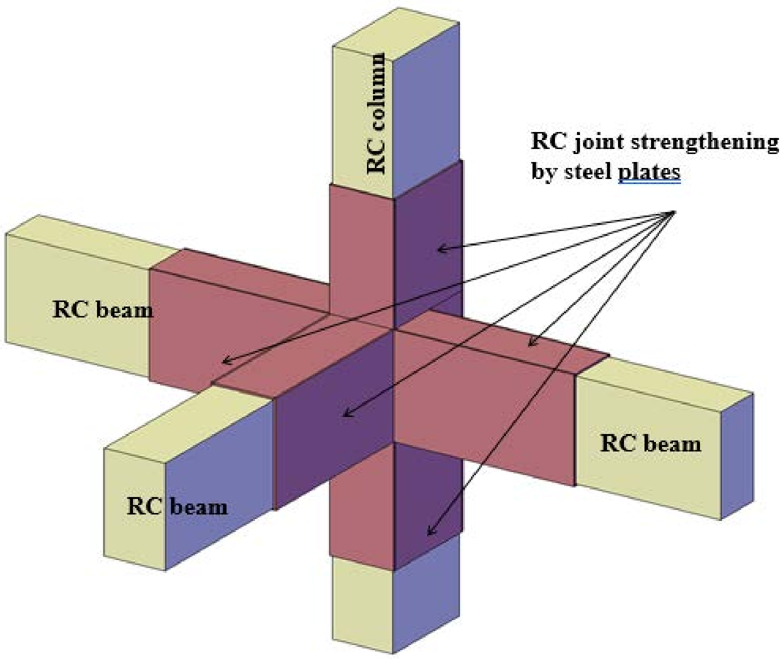

Figure 14: Strengthening of RC columns, beams and joints.

5. Global strengthening of the structure for horizontal seismic action. Introducing additional reinforced concrete shear walls from the basement to the 2nd storey, on their own foundations, at locations where they will not interfere with the equipment and regular functioning of the facility, will reduce structural flexibility, and prevent non-structural damage due to the frequent earthquakes.

5 CONCLUSIONS

Telecommunication Center facility in Skopje has existed for more than fifty years. Due to the requirement for eventual change of the building function, detail analysis of the stability and safety of the existing structural system has been carried out according to current seismic code [5] and required technical standard TIA-942-A [1]. The necessity for structural strengthening has been proved and the most appropriate technical solution for strengthening that satisfies the strength and deformability requirements has been proposed.

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