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Optimization System Moravamont - New Project of Moravamont 2000 for Construction in Seismic Conditions

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Abstract- According to the proposed order of this project, by the usage of the new elements in the adhesion prestressing, it is possible to surmount these spans also in the beam prestressing system without post-tensioning which is the case in the IMS system. By the usage of prestressed hollow slabs on the pathway that have been used for quite a long time, as well as double "P" slabs (by the usage of the universal prestressing on the pathway a quality monolithic joint is set up (by the reinforcement discharge from the above-mentioned elements and a column). Apart from the technological calculation, this joint is confirmed both by structural analysis as well as a dynamic calculation according to the 'Tower' system for P +12 storey. Seismic influences are taken over by the canvases used in the panel system. All further research studies were carried out at the Institute of Seismology - IZIS in Skopje IZIS under the supervision of prof. Predrag Gavrilović.

Keywords- *Adhesion Prestressing, Slabs, Column, Reinforcement, IMS System*

I. INTRODUCTION

A. Factory 'Moravamont' from Gnjilane

Factory 'Moravamont' from Gnjilane is one of the four plants in the system 'Vemont' in the former Republic of Yugoslavia. Apart from the system 'Vemont' the factory owns a panel system 'Adrijamont' as well as the IMS system of prof. Branko Žeželj. Next, the factory has its own raw material base-quarry, a concrete base as well as the contemporary hydraulic mould (steam curing), steel fixing equipment, internal transport system and all the necessary facilities which can be found in the enclosed documentation. In addition to the adhesive prestressing it also has the possibility to expand the core range of its products by the usage of the universal prestressing on the pathway and the innovation within the system as well as the possible combinations. This idea of the combination was supported by my prof. Momir Krastavčević at the post-graduate studies in Niš, in 1985.

Figure 1. Factory MORAVAMONT in Gnjilane

II. ANALYSIS OF THE SYSTEM

A structure P +12 floors is selected for further study and presented as an illustration of the most representative example of a structure design behavior on a non-linear ground that, in a certain way, shows the privilege of the Moravamont System application during seismic conditions.

Starting from the assumption that pillars and inter-floor panels receive exclusively vertical load (static and dynamic) and panels (internal and facade) receive seismic load, there has been a new solution within innovated systems (skeletal and panel) entitled MORAVAMONT 2000.

The new system was analyzed for a typical object of 4,8 and 12 floors with level span of 7,2m in both directions.

For the new projected system, a complete static dynamic calculation based on TAUWER system was done. It showed that all the elements of the system: pillars of 50/50cm dimension, hollow panels of d= 20cm and reverse panel of 25cm in thickness satisfied all the influences of all forces that were applied by the law.

The entire seismic influence was also transferred to reinforced montage concrete panel of 17cm thickness and the supporting facade panel as well.

In the newly presented system Moravamont 2000 it was particularly important to produce (firstly new elements in construction) on the already existing technological foundation with minor changes, followed by monolithic processing of the compounds create the adequate rigidity using simple and quick montage. Pillars are 50/50cm and are made in length - height of three floors to 9 meters, and its continuing is at the middle of the floor height with the use of continuation of armature with the system ERICO LETON. As far as the connection between prestressed P-Panel and hollow inter-floor panel is concerned, an elastic joint is created transversely. Such joint later on goes through monolithic processing with armature in both directions R Fi 20/12, 5 and goes through the same procedure with the rest of the elements excellently.

The connection between pillars and prestressed-Panel is very simple. The panels rely on pillars with discharged vinklas at the floor height in the montage phase.

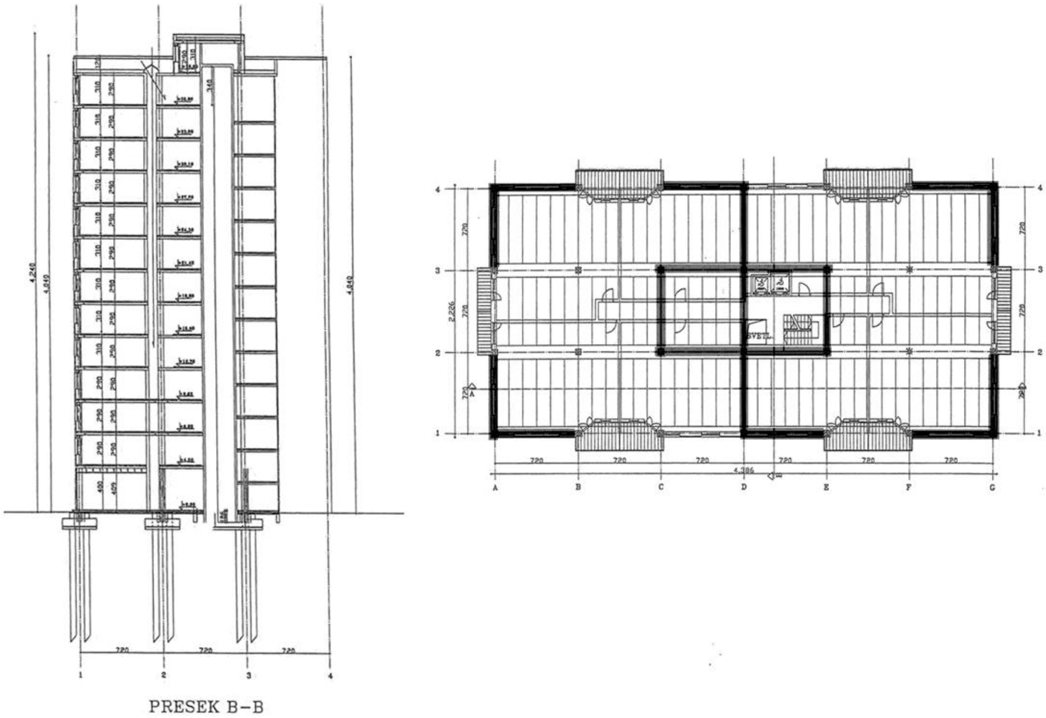


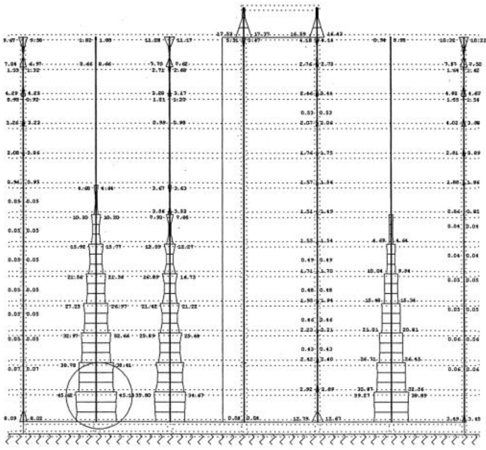
Figure 2. Typical floor

All that armature is connected with the network over the whole inter-floor panel and is watered with concrete of 6cm in thickness, which realistically looked at, makes the panel turn into absolutely rigid in its plane, enabling the transfer of horizontal forces onto panels and facade walls.

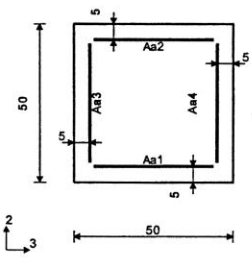
Such newly formed Moravamont 2000 system, actually takes the best qualities of framed and panel system within Moravamont system and with minor technological changes, it becomes economical and simple entirely prefabricated montage construction system.

The idea of universal prestressing on trails which allows huge savings (especially with surface elements of P-panel) is the addition to the technology of this new system Moravamont 2000.

Unlike the existing technology of prestressing where the wires were placed only in a linear mode, and had to be wedged with rubber hose due to a big negative tension at the end of truss, in the new solution the wires are moved under a small angle until negative axis of elements, so as to follow “fizo” line of the truss by its length. The appearance of pivot force on the trail is eliminated by the abutments on the trail because the forces are small due to the deflection angle (5-10) - Hoyer system (Fig. 5).



Greda 942-658 MB 30 (RA 400/500)
PBAB 87



li,2 = 2.90 m (λ2 = 20.09)
li,3 = 2.90 m (λ3 = 20.09)
Nepomerljiva konstrukcija

x = 0.00m
Merodavna kombinacija za savijanje:
1.60xI+1.80xII

N1u =	-12022.70	kN
M2u =	20.94	kNm
M3u =	61.08	kNm

Merodavna kombinacija za torziju:
1.30xI+1.50xII+1.30xIII

M1u =	-0.27	kNm

Merodavna kombinacija za smicanje:
1.30xI+1.50xII-1.30xIII

T2u =	77.06	kN
T3u =	-15.68	kN
M1u =	0.13	kNm

εb/εa = -2.359/-1.606 ‰

Aa1 =	45.62	cm2
Aa2 =	45.18	cm2
Aa3 =	45.55	cm2
Aa4 =	45.17	cm2
Aa,uz =	0.00	cm2/m

Nivo: 1 [2.90] - PBAB 87
MB 30 (d,pl=31.0 cm)
Gornja zona: RA 400/500 (a=2.0 cm)
Donja zona: RA 400/500 (a=2.0 cm)

X=7.20 m; Y=7.20 m; Z=2.90 m
Pravac 1: (α=0°)
Merodavna kombinacija:
1.60xI+1.80xII

Mu =	-421.89	kNm
Nu =	0.00	kN

εb/εa = -3.500/6.435 ‰

Ag1 =	42.60	cm2/m
Ad1 =	0.21	cm2/m

Pravac 2: (α=90°)
Merodavna kombinacija:
1.60xI+1.80xII

Mu =	-166.73	kNm
Nu =	0.00	kN

εb/εa = -1.884/10.000 ‰

Ag2 =	15.27	cm2/m
Ad2 =	0.08	cm2/m

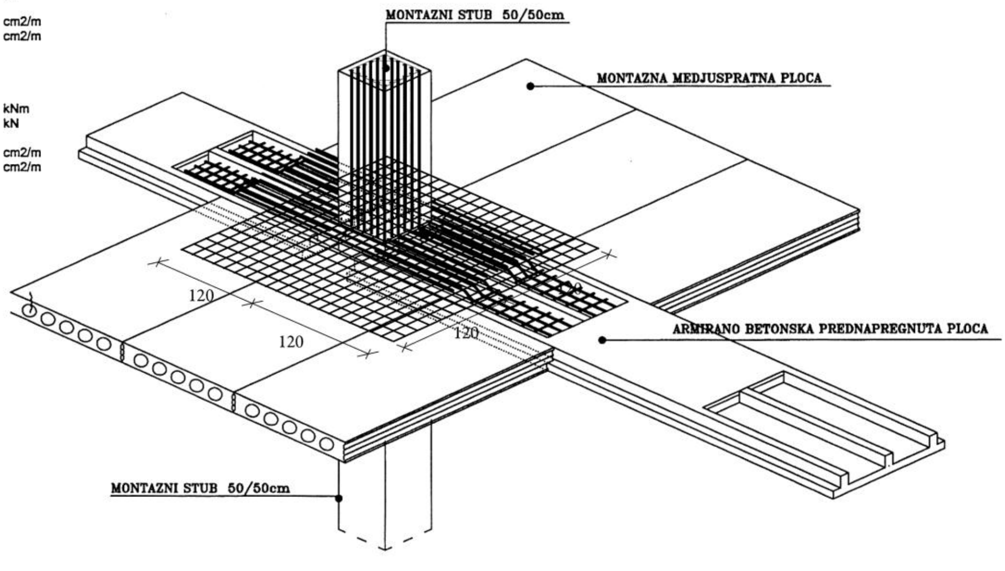


Figure 3. Tower programs for pillar. Connection between pillars and prestressed panel

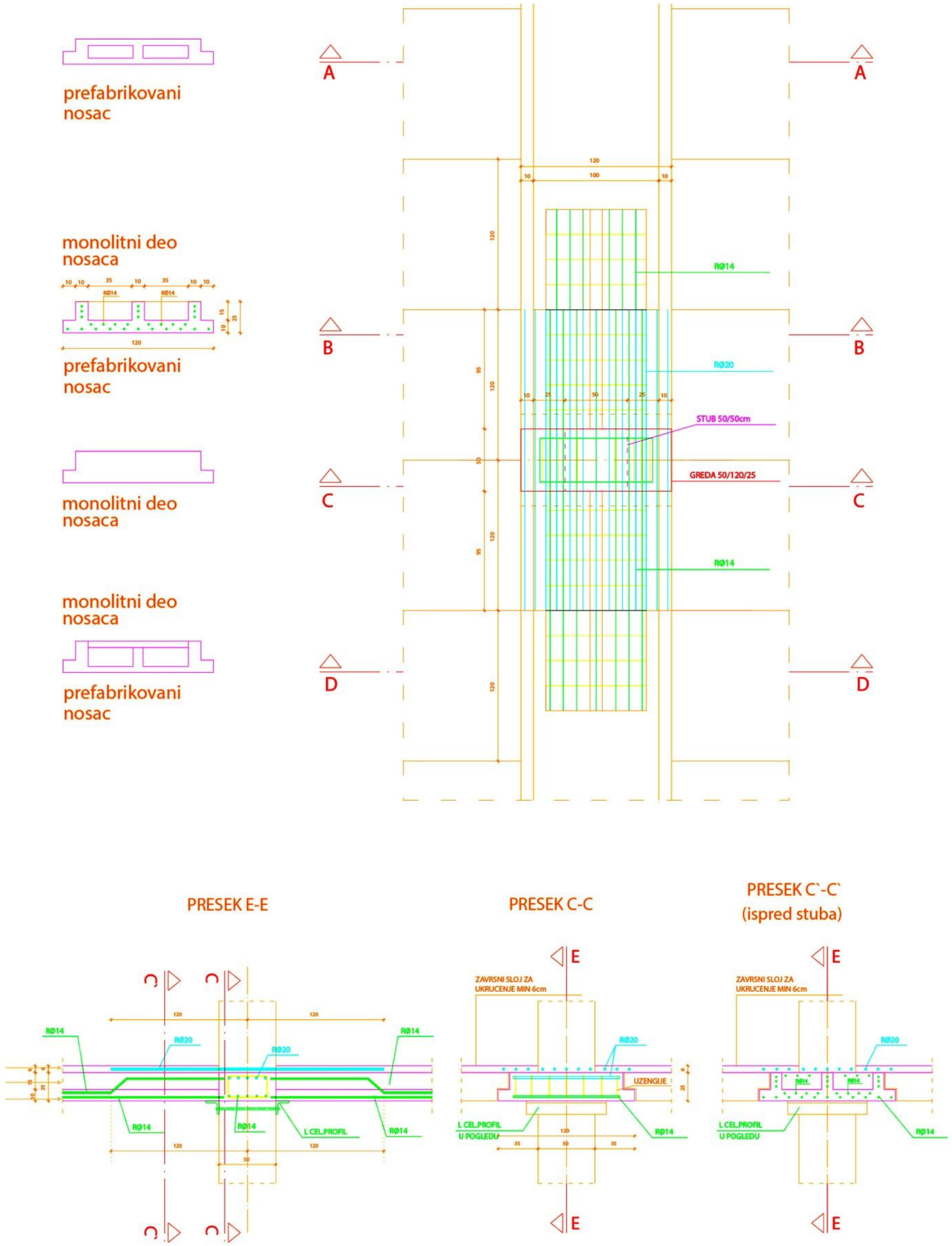
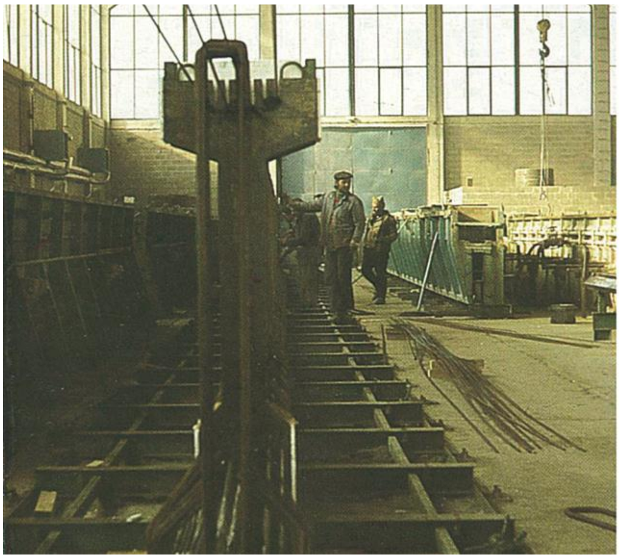


Figure 4. Detalss element prefabriceed (pillars and prestressed panel)



(a)

(b)

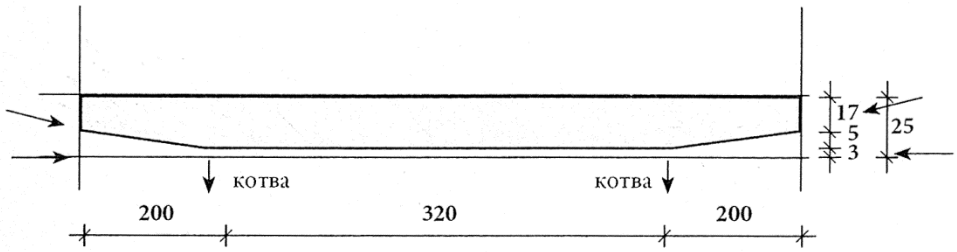


Figure 5. Hoyer system prestressed

To produce the "sandwich panel" (of the facade) in horizontal molds which are equipped with hydraulic tools to lift the platform, to take the elements out and transport them in the vertical position to the place of finishing, we use the thermal treatment of the concrete with circulation of the warm water at 80 Celsius degrees in snakelike tubes. Each table has its own cover which can also be lifted with hydraulic device, and electrical heater installed on its internal side which conducts a thermic processing of that part of the facade sandwich which is above the layer for the thermal isolation. Dimensions of these moulds (tables) are 12.000 x 36000 mm (Fig. 5-b).

As far as connecting the vertical panels are concerned (both internal and external) the modified method was used. Such was examined at the Institute of Technology SHIMIZ at CONSTRUCTION COMPANY and further supported with connection of armatures by the system ERICO-LENTON.

With that aim, experimental and analytical researches of panels were conducted aiming to confirm their relationship to the seismic action. Special consideration was given to connection of vertical wall panel with the method The Nisso Master Builders Splice Sleeve (NMB SS), where the ends are filled with plaster using Spice Sleeves (Embreco 602 Mortal, LL-602) which does not shrink and reaches high firmness in short time.

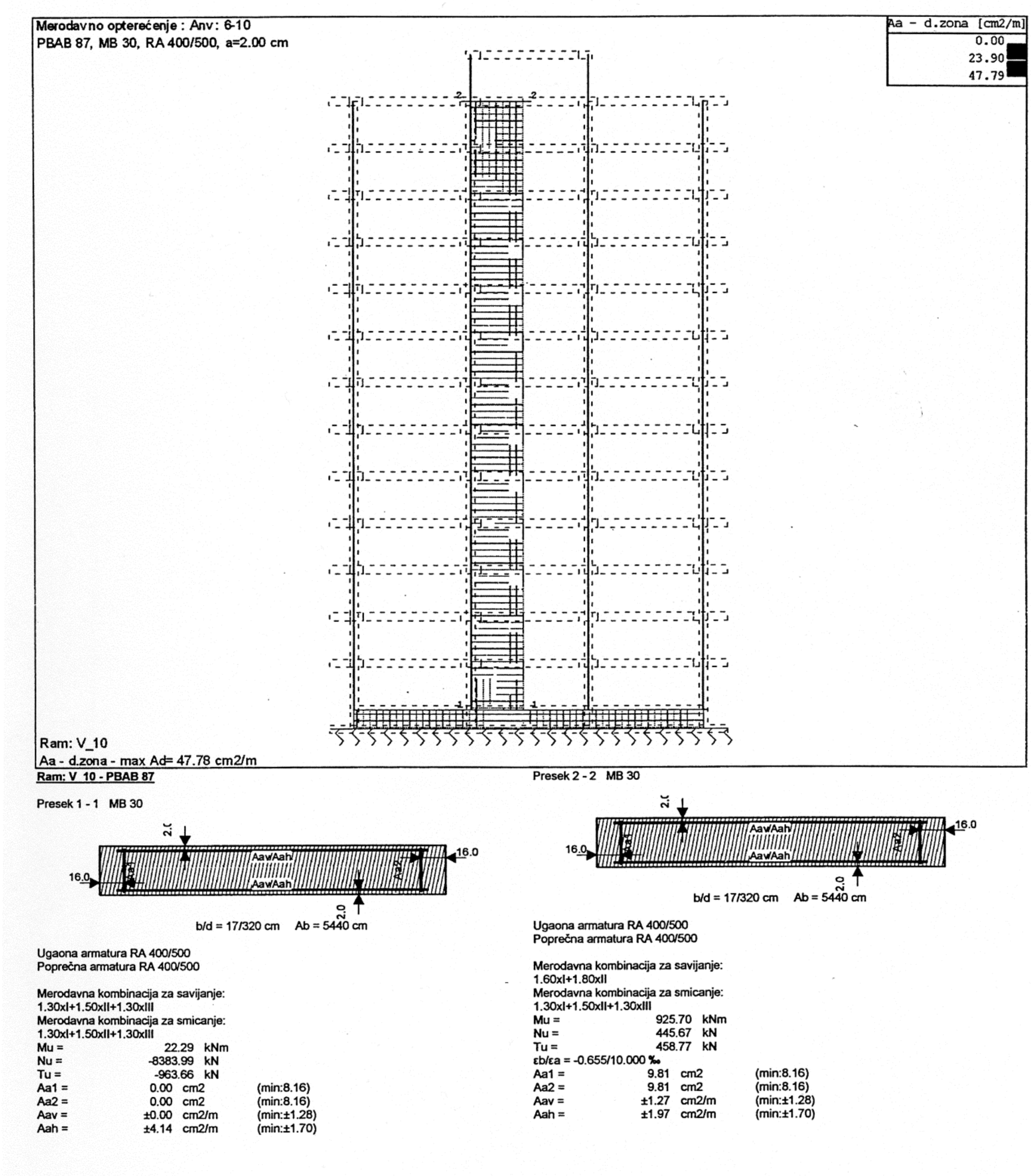


Figure 6. Tower programs for vertical panel

The advantage of this method is the fact that it does not require skilled labor, equipment and thermal energy and makes savings in material.

The procedure is quick, economical and simple. Strengthening around windows is made on the part of the panel which is weakened by the windows, so as to not diminish its role significantly in seismic terms, which was confirmed with static and dynamic calculations TAUER.

P +12 floor facility represents a typical example, and it is the subject of non-linear analysis, as well. In this part of the Study, the results of spatial structural System elastic analysis for seismic effects according to the regulations for seismic activity - level IX have been presented. During the procedure, the necessary elastic dynamic features of the structure design have been obtained, along with all the elements needed for the evaluation of the structural condition and the System as a whole. The obtained results of the analysis are the initial data for non-elastic dynamic analysis of the facility structure conduction.

Fig. 7 shows the schematic structure drawing with clustered masses on the floor level along with the features of the "floor-diagram" in which the floor rigidity, elastic strain limits, plasticity lines, plastic and strain limits, definition of ductility and entire hysteresis dependence are defined (Fig. 2).

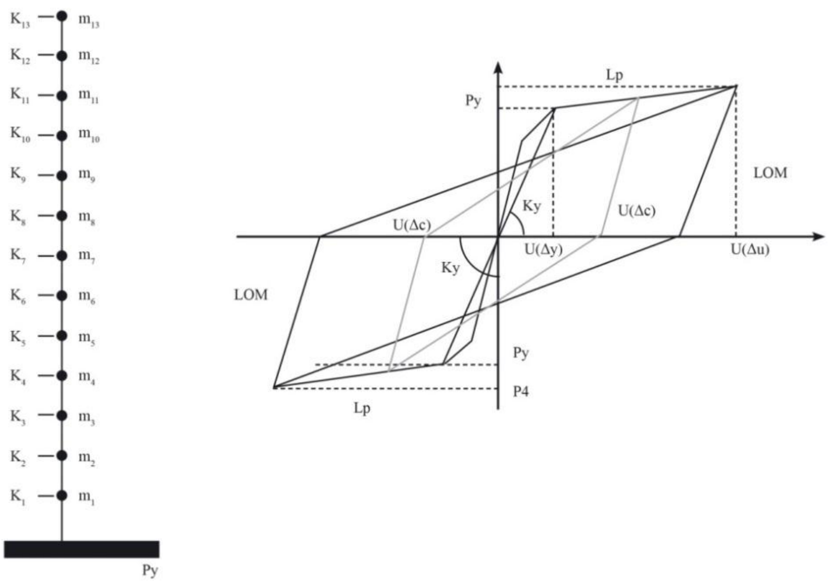


Figure 7. Dynamic model of the building for non-linear dynamic analysis of the construction response to the seismic activity of the registered earthquake records

Two typical time histories have been selected for non-linear response of the structure to the real seismic activities of the earthquakes that really occurred and that were registered under the names - El Centro, N-S component with the original peak ground acceleration of a(g) = 0.33g shown in Fig. 8 and Montenegro earthquake Petrovac - Oliva, N-S component of the original record from 1979, shown in Fig. 9.

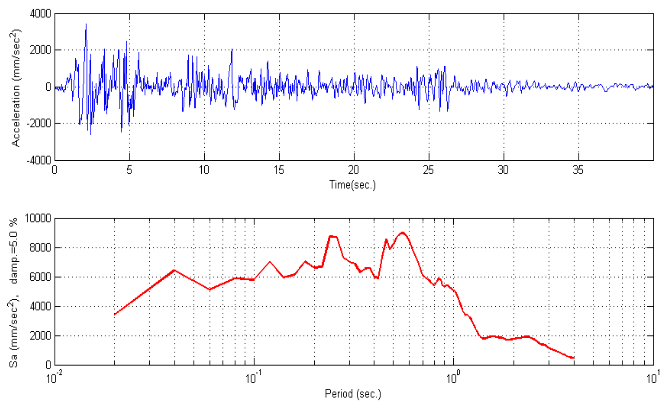


Figure 8. El Centro earthquake recording and range, 1941, N-S component is shown

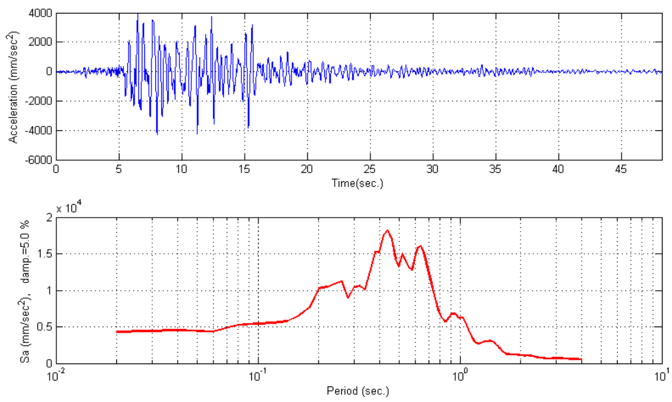


Figure 9. Earthquake recording and range, 1979 Petrovac, N-S component is shown

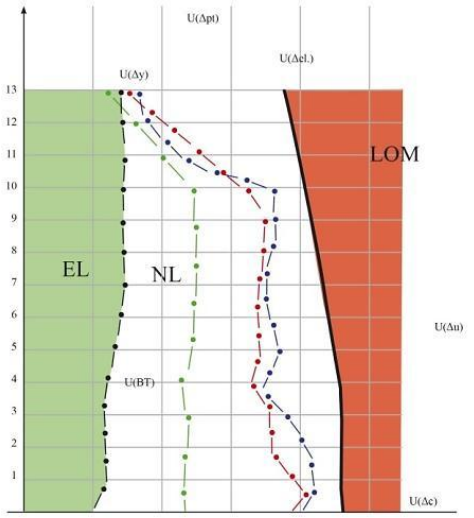


Figure 10. Diagram of floor stress-strain direction x-x

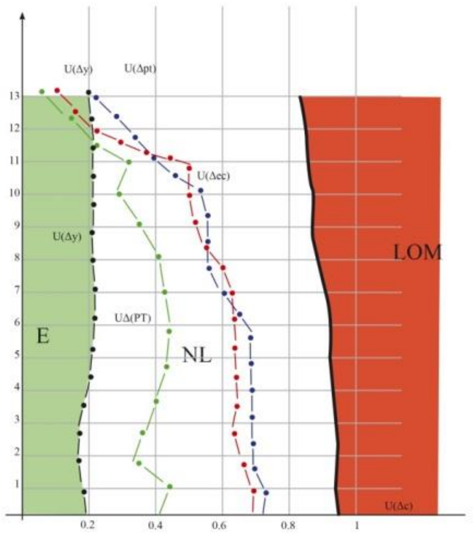


Figure 11. Diagram of relative floor shear strain direction y-y

U (Δy)-elastic strain (Δy flow start)

U (Δpt-Petrovac N-S earthquake strain a = 0.30g

U (Δgl)-El Centro N-S earthquake strain a = 0.30g

U (Δbt)-Bitola (1994) earthquake strain a = 0.30g

U (Δu)-boundary capacity of relative floor shear strain a=0.30g

Line U(Δy) in the diagrams shows the capacity i.e. the elastic strain limit of each floor - the El Centro area - (coloured in green) which represents the field of operation of elastic structure condition.

Line U(Δu) - the ultimate line that marks the structural failure inception (field "FAILURE" - coloured in red) i.e. boundary state and the limit strain capacity after which the failure starts. Structure field of operation between these two lines represents a NON-LINEAR STRUCTURE BEHAVIOUR during expected earthquakes.

Structure response lines for given actions, that is, the strain caused by the earthquakes El Centro *U(Δel) coloured in blue, Petrovac earthquake U(Δpt) and Bitola earthquake U(Δ bt) - coloured in green* are the construction response that is, maximum strain that appears due to earthquake effect assigned in each floor. A brief summary, before the analysis is that the ratio μ = U(Δu)/U(Δy) is a ductility capacity and it is within the limits of 3,5-4 for the designed structure. It is expected and rational solution. The strain ratio on each floor divided by elastic strain gives the value of "REQUIRED - ductility μr = U(Δdj)/U(Δy), that is, the obtained strain in any floor divided by elastic strain gives us the required ductility of a certain floor for the specified earthquake effect.

III. Conclusions

- All structure strains are between elastic and boundary ones, which means that the structure operates in non-linear condition but without a failure. The highest required ductility is on the first floor and its value is 4, all other values are less than the boundary one.

- El Centro earthquake requires the highest level of strains for the same accelerations and the local type of earthquake - Bitola requires the smallest one.

- The maximum required ductility is on the lower floors while on the upper floors it is smaller.

- For the maximum expected earthquake activity, in accordance with the regulations, the structure operates in a non-linear domain but possesses sufficient stability and strength.

- In conclusion, it is clear that the structure is well and rationally designed and the structural behavior is resistant during the expected maximum earthquake activity with a time interval of 500 years recurrence according to the Regulation criteria and it can resist the seismic effects without a major damage.

Non-linear analysis is carried out according to the DRK Programme of the Institute for Earthquake and Seismology Engineering, University "St. Cyril and Methodius", Skopje.

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Biography

MA Živko P. Cuckić, BSc in the field of Construction, Assistant Professor at the Department of the Surface Systems and Structural Dynamics at the Faculty of Civil Engineering in Priština, 1982-1987, Director of Binačka Morava from Gnjilane and the Regional Road Fund, completed his specialization studies for the Prefabrication in Prague, approved by the Academic Lubor Janda, an author of several Strategic Projects especially the ISG in New York City upon the Formation of free economic zones in Old Serbia, via cooperation between the EU and Russia. The owner of the Company "Dadge International LLC".