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# THE QUANTITATIVE SEISMIC ANALYSIS OF A PRECAST REINFORCED CONCRETE FRAME HALL AND THE ESTABLISHMENT OF RETROFITTING SOLUTIONS. PART I – THE DYNAMIC ANALYSIS OF THE EXISTING STRUCTURAL SYSTEM

BY

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**Abstract.** Prefabricated Reinforced Concrete (PRC) frame structures designed and executed in the pre-1989 period in Romania, occupy a leading spot in the list of industrial, commercial, social and cultural buildings. In this research study, a structure of this type is analyzed. In the introductory part of the study, the types of PRC frame systems specific to seismic zones and their component elements are highlighted. The second part shows the main structural deficiencies of the PRC frame structures required to severe earthquakes. In the following chapters, the dynamic analysis related to the studied structure is developed, as well as a verification of the lateral stiffness. Thus, it was found that the PRC structure develops lateral displacements higher than the admissible limit. In these conditions, some solutions are proposed for retrofitting the structural system using

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vertical steel braced frames, shear walls, steel-RC composite columns and steel beams on the bay direction.

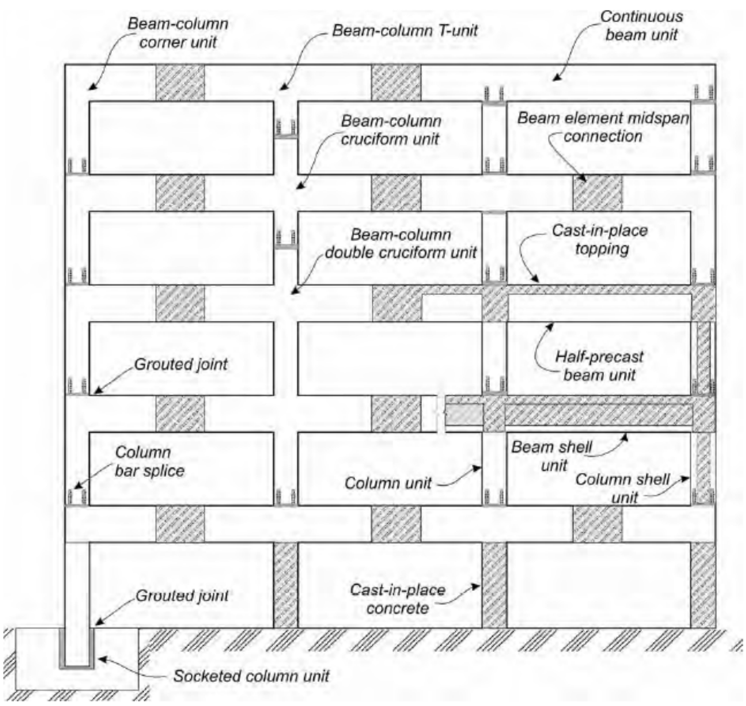
**Keywords:** PRC frame structure, modal analysis, relative storey displacements, retrofitting solutions, additional elevation solutions.

## 1. Introduction

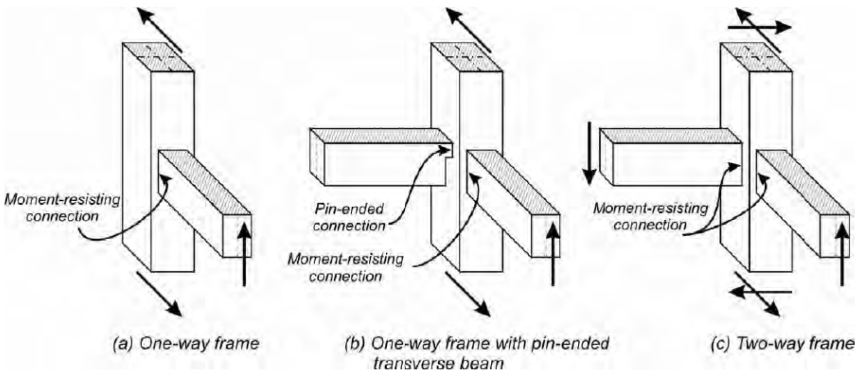
Precast concrete building structures are widely used as structural systems in many earthquake-prone countries. These systems can be combined in different ways to obtain an appropriate and effective structural concept that fulfils the needs of specific buildings. The most common systems are (FIB 43, 2008), (FIB 27, 2003):

i. precast moment resisting frames (see Fig. 1);
ii. precast structural wall systems;
iii. precast dual wall-frame systems;
iv. precast floor and roof diaphragms (see Fig. 2).

The above list is not unique as there are many variations possible to achieve the same objectives that architects and engineers are now successfully exploring (FIB 43, 2008).




(a)

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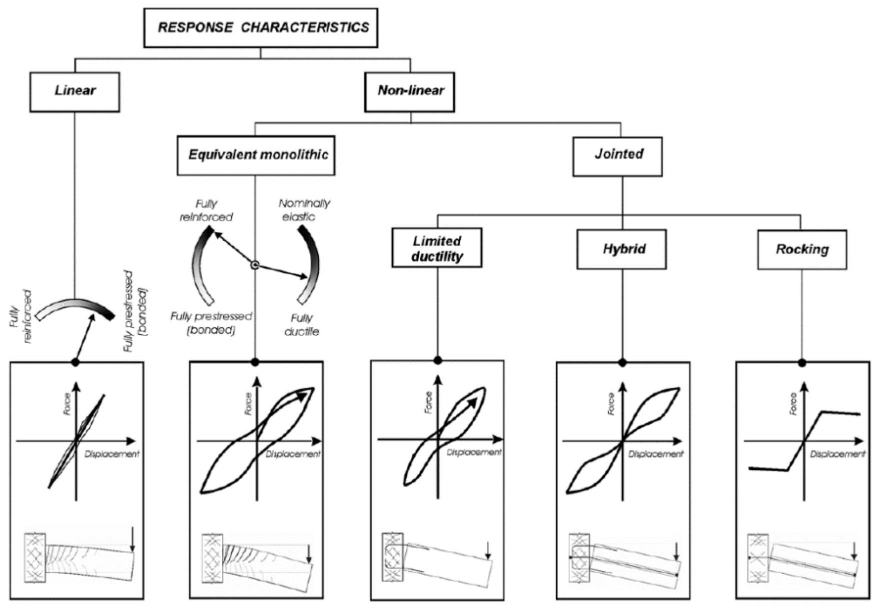



(c)

Fig. 1 – “(a) Common arrangement of elements, components and connections in precast concrete moment-resisting frame construction; (b) Classification of precast concrete moment-resisting systems according to the in-plane lateral force resisting mechanisms; (c) Classification of precast concrete moment-resisting systems acording to the lateral force-lateral displacement response characteristics” (FIB 27, 2003).

The construction of moment resisting frames and structural walls incorporating precast concrete elements usually fall into two broad categories; either “equivalent monolithic” systems, or “jointed” systems. The distinction between these types of construction is based on the design of the connections between the precast concrete elements (FIB 27, 2003).

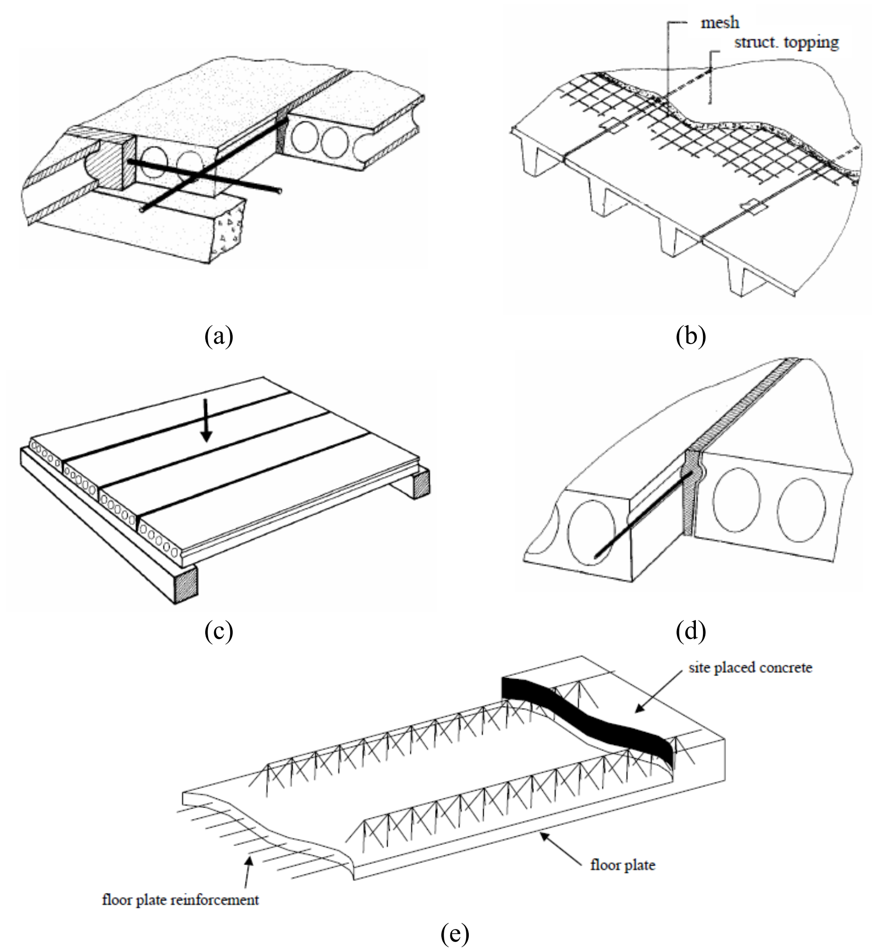



Fig. 2 – "Representation of the: (a) typical hollow core floor system; (b) typical double-T floor system; (c) transverse load distribution; (d) joint detail with longitudinal shear key; (e) composite floor plate floor system with lattice girders" (FIB 43, 2008).

The connections between precast concrete elements of equivalent monolithic systems (cast-in-place emulation) can be subdivided into two categories (FIB 27, 2003):

i. strong connections of limited ductility;

ii. ductile connections.

In jointed systems the connections are weaker than the adjacent precast concrete elements. Jointed systems do not emulate the performance of cast-in-place concrete construction. The connections between precast concrete elements of jointed systems can be subdivided into two categories (FIB 27, 2003):

i. connections of limited ductility;
ii. ductile connections.

These categories are detailed in the specialized literature and have not been developed in this research article.

Precast floor units (see Fig. 2) must be detailed to accommodate the localized displacements that are likely to be imposed on them by the actions of the primary lateral load resisting system. The two procedures used to accommodate these actions are:

i. Isolate the precast components from any high displacement demands, with sliding supports and compressible joints. This method is preferred for relatively brittle extruded or slip-formed hollow core sections, and for some of the more brittle beam and block flooring systems;
ii. Reinforce the precast units, and any composite topping concrete to provide adequate ductility to resist the required gravity loads during, and after, the imposed displacements (FIB 27, 2003).

## 2. Lessons from previous earthquakes regarding the seismic response of poorly designed and detailed precast concrete frame buildings

The main deficiencies specified in the specialized literature and recorded by precast concrete frame buildings are (FIB 27, 2003):

i. improper design and detailing of ductile elements;
ii. inadequate diaphragm action;
iii. poor joint and connection details;
iv. inadequate separation of non-structural elements;
v. inadequate separation between structures.

The retrofitting solutions for precast RC frame systems were developed in chapter 5.

### 2.1. Improper design and detailing of ductile elements

For low-rise precast concrete structures, the columns or wall panels are the ductile elements, with the floor and/or roof elements designed to be stronger than these ductile energy-dissipating elements in accordance with accepted capacity design principles (Park & Paulay, 1975). It is of interest that older moment resisting frames and structural walls incorporating precast concrete often have been observed to perform badly during earthquakes. The observed failures have been mainly due to brittle (non-ductile) behavior of poor connection details between the precast elements, poor detailing of the elements and poor design concepts. Fig. 3(a) and Fig. 3(b) show examples of major damage to precast concrete structures as a result of severe earthquakes.

As a result, the use of precast concrete was shunned in some countries in seismic zones for many years" (FIB 27, 2003).

(a)

(b)

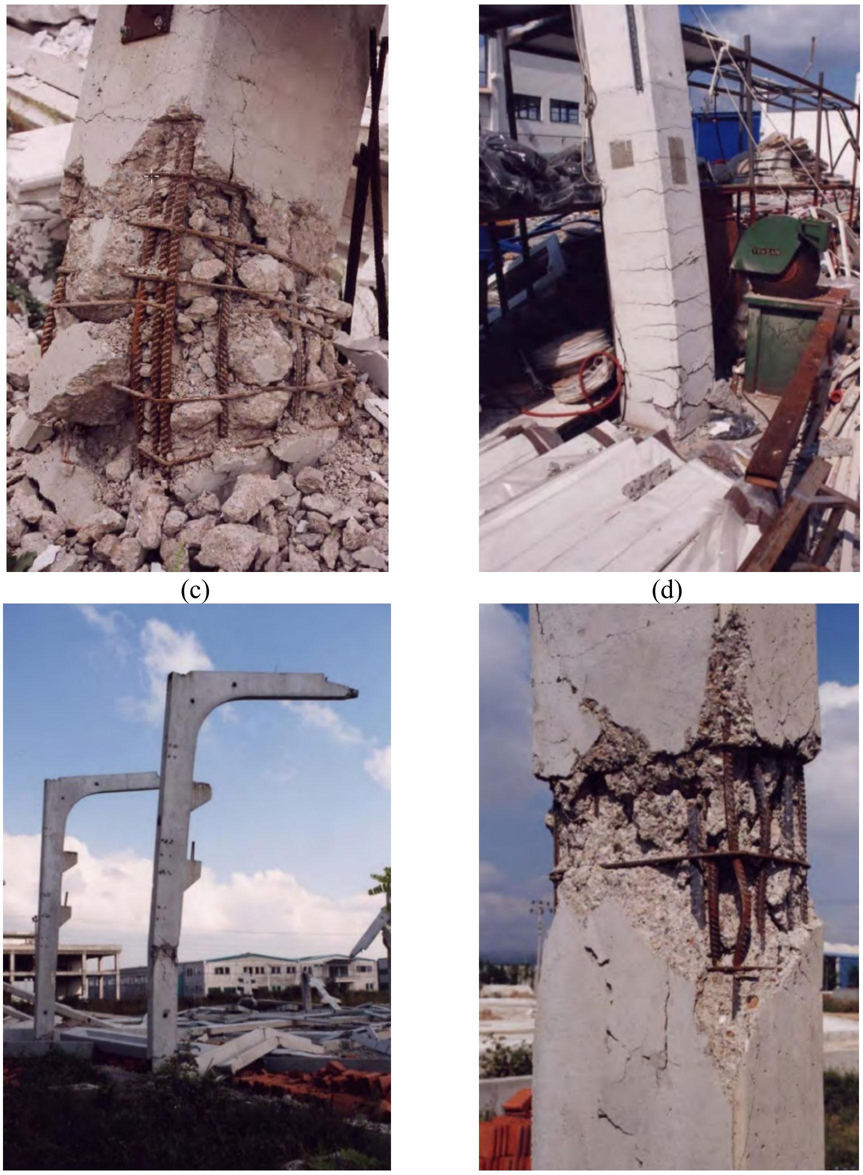
Fig. 3 – "(a) Collapse of precast buildings in Tangshan, China in 1976; (b) Collapse of precast buildings in Leninakan, Armenia in 1988" (FIB 27, 2003).

Many precast concrete frame structures collapsed during the 1988 Armenian earthquake (EERI, 1989). These structures were typically nine storeys in height and contained hollow-core floor slabs. Some of the structures had some walls in one direction but these walls typically contained large openings. The beam-column connections were made by welding the beam bars to steel angles protruding from the precast columns. The floor diaphragms were poorly connected to the frame elements. Column splices were made by welding the vertical column bars. This detail resulted in significant eccentricities of the column bars at the splice location. The columns were poorly detailed with inadequate confinement and had column ties with only 90° bend hooks. These hooks were ineffective once spalling of the concrete cover occurred in the columns (FIB 27, 2003).

Also, in Fig. 4 can be observed: (a) the failure of poorly detailed precast concrete column in the 1994 Northridge Earthquake; (b) the shear failure of precast concrete column at foundation socket (1999 Kocaeli Earthquake); (c) the severe distress in precast concrete column (1999 Kocaeli Earthquake); (d) the close-up of poor column details (1999 Kocaeli Earthquake). Details on the failure mechanisms can be found in the FIB Bulletin (FIB 27, 2003).

(a)

(b)



(c) (d)

(e) (f)

Fig. 4 – “(a) Failure of poorly detailed precast concrete column in the 1994 Northridge Earthquake; (b) Shear failure of precast concrete column at foundation socket (1999 Kocaeli Earthquake); (c) Shear failure at base of precast concrete column (1999 Kocaeli Earthquake); (d) Flexural hinging at base of precast concrete column (1999 Kocaeli Earthquake); (e) Severe distress in precast concrete column (1999 Kocaeli Earthquake); (f) Close-up of poor column details (1999 Kocaeli Earthquake)” (FIB 27, 2003).

### 2.2. Inadequate diaphragm action

As well as carrying the gravity loads, floors need to transfer the imposed seismic forces to the supporting structure through diaphragm action. Adequate connections to transfer diaphragm forces and adequate support of precast concrete floor units are basic requirements for a safe structure. It is essential that floor systems do not collapse as the result of imposed movements caused by earthquakes which will reduce the seating area at the supports of the precast floor units. Fig. 5 shows the collapse of a precast concrete hollow core floor which became unseated during the 1994 Northridge earthquake.

There were many examples of failures of diaphragms in the 1999 Kocaeli earthquake (Saatcioglu *et al.*, 2001), (EERI, 2000).

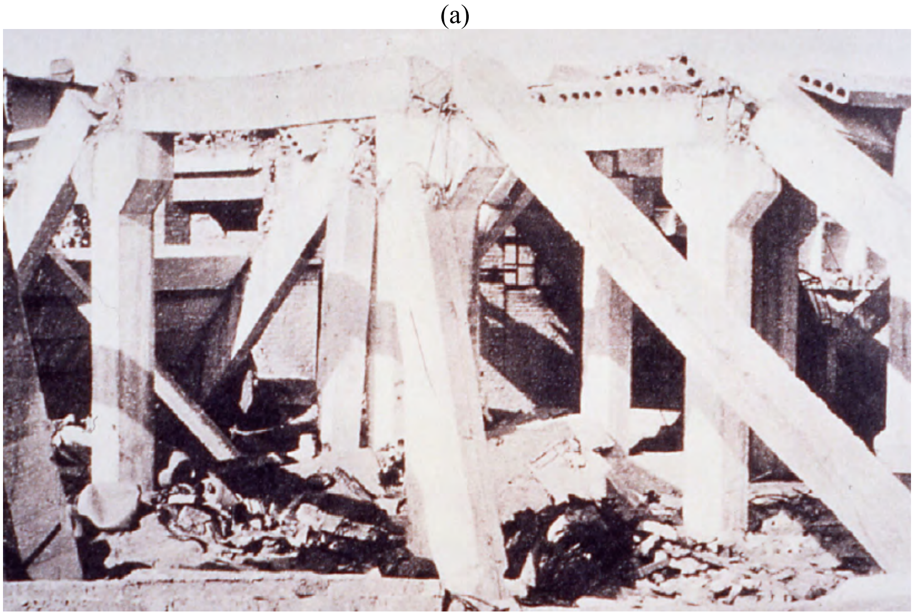
The diaphragms that failed were very flexible, made with corrugated fibre concrete or thin metal sheets and the panels of these diaphragms were poorly connected to each and to the main structural components (FIB 27, 2003).

Fig. 5 – "Collapse of precast concrete hollow core floor (1994 Northridge Earthquake)" (FIB 27, 2003).

### 2.3. Poor joint and connection details

Beam-column joints of moment resisting frames incorporating precast concrete elements have often failed in earthquakes due to poor details. A significant cause of failure has been the brittle failure of reinforcing bars in the vicinity of welds. Fig. 6(a) shows the failure of a poorly detailed moment-resisting connection between beams and columns as a result of the Tangshan Earthquake in China in 1976 (FIB 27, 2003).

(a)



(b)

Fig. 6 – "(a) Failure of a poorly detailed beam-column connection (1976 Tangshan Earthquake, China); (b) unseating of precast concrete beams off corbels of columns" (FIB 27, 2003).

Fig. 7(a) shows the shear distress in a precast column of a one-storey parking structure in Sherman Oaks that occurred during the 1994 Northridge Earthquake. Although the interior columns of this structure had been retrofitted following the 1971 San Fernando earthquake, no retrofit was carried out on the exterior columns. The 406 × 610 mm exterior columns were reinforced with #10 (32 mm diameter) vertical bars and had peripheral #3 (9.5 mm diameter) ties at 400 mm spacing" (FIB 27, 2003).

(a) (b)

Fig. 7 – "(a) Beam-to-column connection failure and shear failure of column in precast concrete parking structure (1994 Northridge Earthquake); (b) brittle failure of beam-column pin connection at top of corner column (1994 Northridge Earthquake)" (FIB 27, 2003).

Not only did many of the inadequately reinforced columns suffer major shear distress but there were also several examples of failures of the connections between the beams and the columns. This connection consisted of connection pins connecting the beams to the columns (FIB 27, 2003).

In some cases, the pins were misplaced such that the pins were outside of the transverse reinforcement of the column, resulting in side shear failure of the cover concrete (see Fig. 7(b)) (FIB 27, 2003).

**2.4. Inadequate separation of non-structural elements**

Fig. 8 shows the collapse of a precast concrete industrial building (Saatcioglu *et al.*, 2001). The building was under construction at the time. The presence of the stiff masonry infills that interacted with the columns resulted in column failure at the top of the partially collapsed walls where the vertical bars were spliced (FIB 27, 2003).

Fig. 8 – "Failure of columns due to interaction with stiff masonry infill (1999 Kocaeli Earthquake)" (FIB 27, 2003).

**2.5. Inadequate separation between structures**

Fig. 9 shows an overall view of a precast concrete parking structure in Mexico City that partially collapsed in the 1985 Michoacán earthquake (Mitchell *et al.*, 1986). This frame structure suffered from pounding with an adjacent parking structure due to inadequate separation between the two structures. It also experienced failures at the precast joints between the beams and the columns resulting in brittle failures and poor energy absorption. It is essential that adequate separation between structures be provided to prevent pounding between the structures during an earthquake (FIB 27, 2003).

Fig. 9 – "Collapse of precast concrete parking structure due to pounding against adjacent structure in Mezico City (1985 Michoacan Earthquake)" (FIB 27, 2003).

## 3. Input data regarding the dynamic analysis of the existing precast RC frame system

The current analytical study includes the first part (dynamic analysis) of the "quantitative evaluation" (Kassem *et al.,* 2022) of a **P**refabricated **R**einforced **C**oncrete (PRC) frame hall, with the general data represented in Table 1.

Following the visual inspection of the technical expert, which corresponds to the limited knowledge level KL1 and to the trust factor CF=1.35 (P100-3, 2019), the following structural deficiencies were noted:

i. the PRC beams exhibit a pin configuration in the marginal zones because there is no monolithic concrete and RC beam-column joints (see Fig. 10 (a));
ii. the **P**recast **H**ollow **C**ore (PHC) slab strips" (Liu *et al.*, 2022) are one-way pin supported on the existing beams (see Fig. 10 (b));
iii. the existence of PRC beams on a single direction of the structural system (see Fig. 10 (c));
iv. the non-existence of the longitudinal RC joint shear key and friction bars between PHC slab strips or the cast-in-place reinforced concrete topping. Basically, the PHC slab strips exhibit an independent, non-frictional (non-dissipative) behavior and thus do not act as a rigid diaphragm in the horizontal plane or as isolated frictional precast component (see Fig. 10 (d)).

**Table 1**
*General data regarding the existing situation of the structural system*

| | |
|---|---|
| Building type | *Hall* |
| Structural system type | *Prefabricated Reinforced Concrete (PRC) frame* |
| Year of building | *1990* |
| Location | *Focşani Municipality, Vrancea County (see Fig. 11)* |
| Existing structural elements | *PRC columns with a cross-section of 55x55 cm; PRC beams with a cross-section of 30x60 cm; PHC slabs with a total thickness of 15 cm which are pin supported on RC beams; insulated prefabricated glass-type foundations with general in-plane dimensions of 330x330 cm* |
| Concrete strength class | *C20/25* |
| Height regime | *GF+1F* |
| Storey height | *4.20 m* |
| Number of bay-span sectors | *16* |
| Number of spans | *2* |
| Inter-axle distance of spans | *6 m* |
| Number of bays | *8* |
| Inter-axle distance of bays | *6 m* |
| Foundation mark | *-2.95 m* |

a)

c)

b)

d)

Fig. 10 – The representation of: (a) PRC beams with pin configuration in marginal areas (without any monolithic concrete); (b) the one-way pin supported mode of PHC slab strips on existing PRC beams; (c) prefabricated RC beams on a single global direction of the structural system (along the span direction); (d) the non-existence of an additional top layer of concrete on the PHC slab strips.

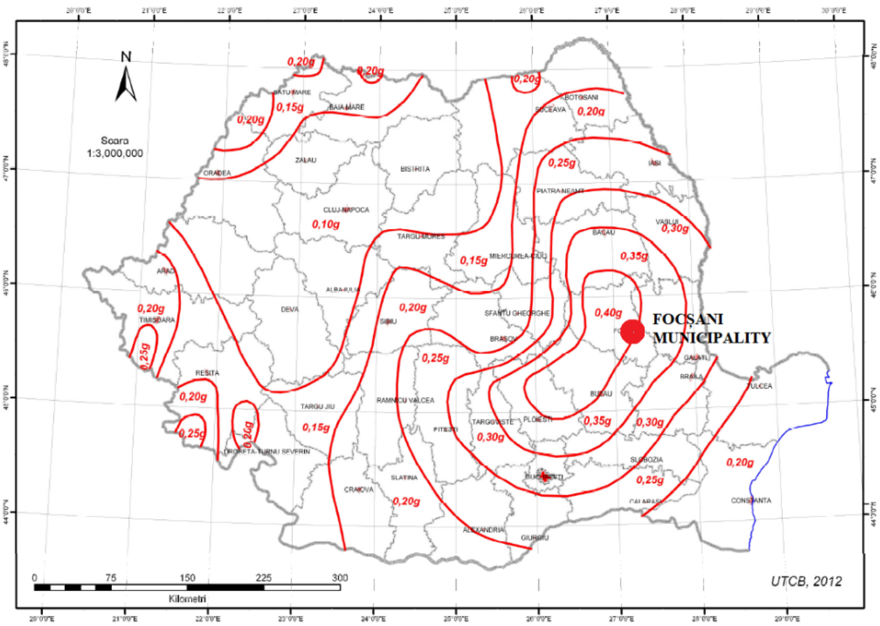



Fig. 11 – „Seismic zoning map of Romania (with the location of the analyzed structure – Focșani Municipality) in PGA design values „$a_g$” with IMR=225 years and 20% probability of exceedance in 50 years” (P100-1, 2013).

These structural deficiencies may lead to the following non-favorable effects, due to the severe seismic loads corresponding to the prefabricated structural system of reinforced concrete frame:

i. the loss of local stability of PRC columns/ beams and PHC slab strips;
ii. the reduction of the structure robustness” (Caldentey *et al.,* 2021);
iii. the brittle failure of PRC columns/ beams/ PHC slab strips” (Li *et al.,* 2016), (Ali *et al.,* 2022);
iv. the non-uniform loading in gravitational direction of the PRC columns;
v. the non-uniform loading in horizontal direction of the entire structural system, generating the possibility of the development of a „general torsion effect” (Ghayoumian & Emami, 2020);
vi. failure of poorly detailed precast concrete column (see Fig. 4);
vii. shear failure of precast concrete column at foundation socket (see Fig. 4);
viii. shear failure at base of precast concrete column (see Fig. 4);
ix. collapse of precast concrete hollow core floor (see Fig. 5);
x. failure of a poorly detailed beam-column connection (see Fig. 6);
xi. unseating of precast concrete beams off corbels of columns (see Fig. 6);
xii. beam-to-column connection failure (see Fig. 7);
xiii. brittle failure of beam-column pin connection at top of corner column (see Fig. 7) etc.

## 4. The dynamic analysis of the PRC frame system

The evaluation of the structural system was performed by employing the level 2 methodology, of medium complexity, such that the structural deficiencies, as well as the composition mode of the lateral system, do not require complex verifications.

Thus, in this first stage, „the dynamic analysis" (Budescu & Ciongradi, 2014), (Postelnicu, 2012), (Stratan, 2014) of the PRC frame system was performed, which includes the following subchapters (EC 2, 2006), (EC 8, 2004):

i. the evaluation of actions and the combinations of the effects of actions;
ii. the modal analysis with the specification of the eigenmodes of vibration, with the corresponding periods;
iii. the checking of the lateral displacements in the Ultimate Limit States (ULS), respecting the calculus relations specified in the Romanian norms for the seismic design of new structures P100-1 (P100-1, 2013) and for the seismic evaluation of existing structures P100-3 (P100-3, 2019).

### 4.1. The evaluation of actions

The evaluation of actions and the combinations of the effects of actions was made according to CR 0/ 2012, CR 1-1-3/2012 norms, SR EN 1991-1-1/NA (National Annex) standard and P100-1/ 2013. This evaluation of the actions was considered as a result of the requirements imposed by the investor. Basically, the existing structural system (with major structural deficiencies) is evaluated with the probable loads (established according to the future destination of the building).

Thus, there is no need to consider a special load case scenario, as this loading situation of the existing structure is the most unfavorable for the structural system.

Thew details regarding the loads considered on the top (master) storey slab, on the current storey slab and of the design lateral seismic action can be studied in Table 2, Table 3 and Table 4.

**Table 2**

*Loads considered on the top (master) storey slab*

| Load type | The value of surface load (kN/m$^2$) |
|---|---|
| *Hydro-insulation load* | *0.5* |
| *Slope concrete load* | *1.5* |
| *Plaster load* | *0.5* |
| *Live load* | *1* |
| *Snow load* | *2.5* |

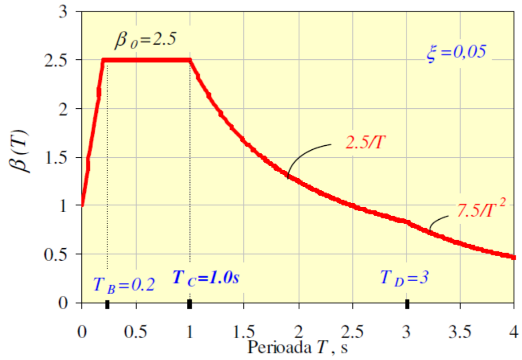
**Table 3**

*Loads considered on the current storey slab*

| Load type | The value of surface load (kN/m$^2$) |
|---|---|
| *Floor load* | *1* |
| *Partition wall load* | *1* |
| *Plaster load* | *0.5* |
| *Live load* | *2.5* |

**Table 4**

*Necessary data for the consideration of the seismic action in structural seismic design*

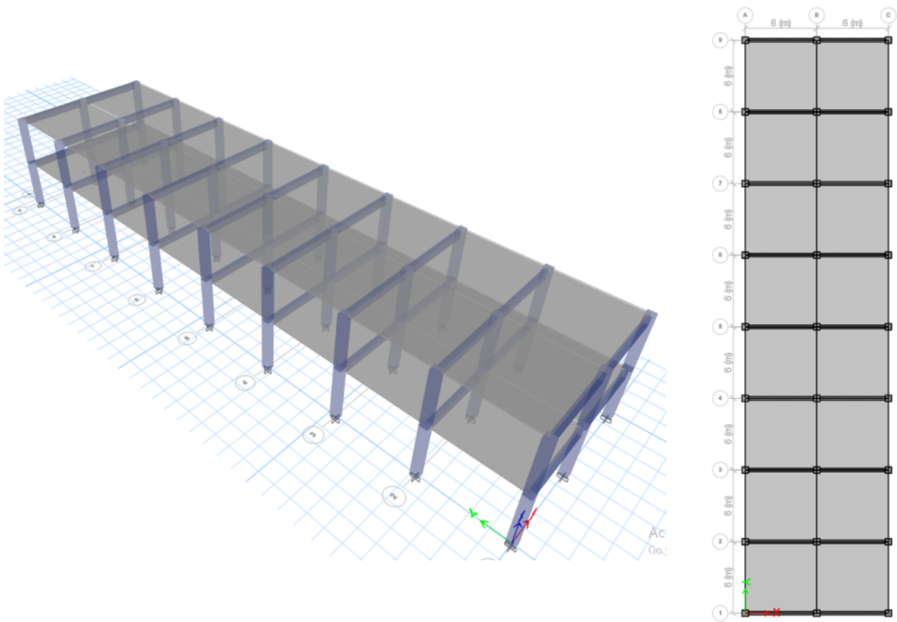
| Parameters | Specific elements |
|---|---|
| *Employed method* | *The modal method with response spectra* |
| *Considered limit state* | *ULS* |
| *The representation of the normalized response elastic spectrum in absolute accelerations for the horizontal components of motion in Vrancea (Focşani)* |  |
| *The representation of the design spectrum in absolute accelerations for the components of horizontal soil motions in Vrancea (Focşani)* |  |
| $T_C$ | *1.00 (s)* |
| $T_B$ | *0.20 (s)* |
| $T_D$ | *3.00 (s)* |
| $a_g$ | *0.4g* |
| $q$ | *3.5 (according to P100-3/2019)* |
| *Importance class* | *III* |
| $\gamma_{I,e}$ | *1* |
| $\beta_0$ | *2.5* |
| $\varepsilon$ | *0.05 (5%)* |
| *The combination of the effects of actions* | ***Modal:** 1P + 0.4L + 0.4S*<br>***Fundamental 1:** 1.35P + 1.5L + 1.05S* |

| | |
|---|---|
| | ***Fundamental 2:*** *1.35P + 1.05L + 1.5S*<br>***Seismic X:*** *1P + 0.4L + 0.4S + 1Seism_X*<br>***Seismic Y:*** *1P + 0.4L + 0.4S + 1Seism_Y*<br>*where "+" represents the combination of the effects of actions, not a summation*<br>*P –**P**ermanent; L –**L**ive; S –**S**now* |

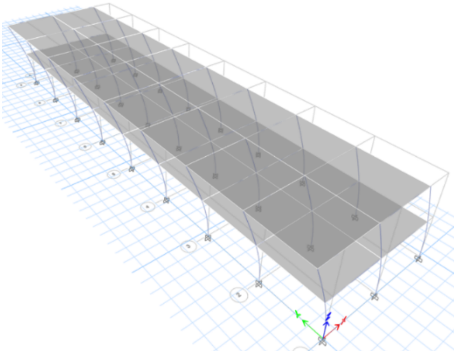
**4.2. Modal analysis**

In the modal analysis it was proposed to observe the first three eigenmodes of vibration of the structure (see Fig. 12 (a)) and the period values adjacent to them (see Table 5).

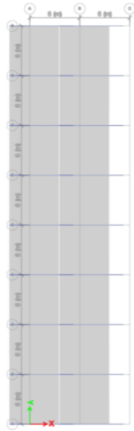
In these conditions, a translation motion on the global X direction was observed (see Fig. 12 (b), Fig. 12 (c)) as well as on the global Y direction (see Fig. 12 (d), Fig. 12 (e)) for the first two eigenmodes of vibration. For the third eigenmode of vibration, a general torsion motion was observed, for the structural system (see Fig. 12 (f), Fig. 12 (g)).



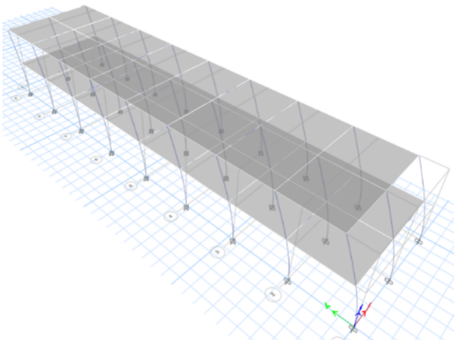
(a) (b)



(c)



(d)



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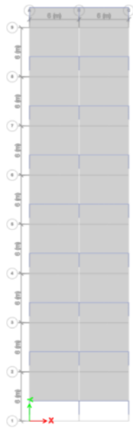
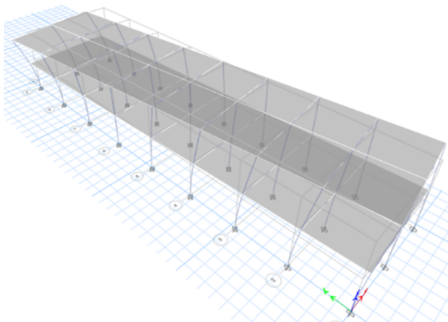
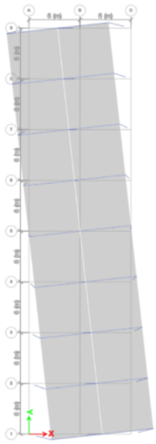


(f)



(g)



(h)

Fig. 12 –The representation of: (a) the tridimensional structural system; (b) plan dimensions of the structure with lateral resistant elements; (c), (d) the first eigenmode of vibration; (e), (f) the second eigenmode of vibration; (g), (h) the third eigenmode of vibration.

Furthermore, after the modal analysis was carried out, the behavior factor ”q” of the structure (see Table 4) was established, because the fundamental period of vibration of the structure is known (see Table 5), as well as the translation mode of the PRC structural system.

Prefabricated RC beams involve a pin configuration in marginal zones, so in the first and second vibration modes the period values is identical for a translation motion on the two global directions of the lateral system.

For the same reason, a major value of the fundamental vibration period of 0.868 (s) is registered.

Thus, the modal analytical study presents a preliminary conclusion regarding the seismic insecurity of the lateral structure, due to the absence of RC beam-column joints.

**Table 5**

*The results of the modal analysis*

| Vibration mode | Period (s) | Modal participating mass ratios | | Sum (X /Y) | |
|---|---|---|---|---|---|
| | | Global direction X | Global direction Y | Global direction X | Global direction X |
| *1* | *0.868* | *0.7883* | *0* | *0.7883* | *0* |
| *2* | *0.868* | *0* | *0.7883* | *0.7883* | *0.7883* |
| *3* | *0.772* | *0* | *0* | *0.7883* | *0.7883* |
| *4* | *0.135* | *0.2117* | *0* | *0.9986* | *0.7883* |
| *5* | *0.134* | *0* | *0.2117* | *0.9986* | *1* |
| *6* | *0.124* | *0* | *0* | *0.9986* | *1* |
| *7* | *0.108* | *4.734E-06* | *0* | *0.9986* | *1* |
| *8* | *0.082* | *0.0014* | *0* | *1* | *1* |
| *9* | *0.049* | *0* | *0* | *1* | *1* |
| *10* | *0.046* | *0* | *0* | *1* | *1* |
| *11* | *0.044* | *0* | *0* | *1* | *1* |
| *12* | *0.043* | *0* | *0* | *1* | *1* |

### 4.3. The verification of lateral displacements in ULS

In the specialized literature (Budescu & Ciongradi, 2014), (Paulay & Priestley, 1992) this verification of lateral rigidity of the structure is found as a form of verification and limitation of the deformation of the structure to prevent its general collapse. In these conditions, the verification of the lateral displacements of the structure also involves the verification of the spatial cooperation of the seismic-resistant elements that participate in the dissipation of the seismic energy induced by the earthquake. This spatial cooperation of structural elements is all the more important for PRC frame structure that "inadequate separation of non-structural elements" is found to be a system deficiency presented in chapter 2.

The verification of the lateral displacements is performed based on the calculus relations from P100-1(P100-1, 2013) norm and P100-3 (P100-3, 2019) norm.

Thus, the relative storey displacement „$d_r^{ULS}$" associated to the Ultimate Limit State (ULS), was computed for the PRC structural system according to Equation 1.

Equation 2 and Equation 3 represent the method of computing these lateral displacements on the global X and Y directions of the PRC hall.

$$d_r^{ULS}=c\cdot q\cdot d_{re}\leq d_{r,a}^{ULS} \tag{1}$$

$$d_{r,X}^{ULS}=c\cdot q\cdot d_{re}^{X}\leq d_{r,a}^{ULS} \tag{2}$$

$$d_{r,Y}^{ULS}=c\cdot q\cdot d_{re}^{Y}\leq d_{r,a}^{ULS} \tag{3}$$

here:
$d_r^{ULS}$ – the relative storey displacement under the seismic action associated to the Ultimate Limit State (ULS);
$d_{r,X}^{ULS}$ – the relative storey displacement under the seismic action associated to the Ultimate Limit State (ULS) for the global X direction of the structural system;
$d_{r,Y}^{ULS}$ – the relative storey displacement under the seismic action associated to the Ultimate Limit State (ULS) for the global Y direction of the structural system;
c – the amplifying factor of lateral displacements, computed according to Equation 4 and the fundamental vibration period of the structure;
q – the reduction factor of the seismic actions with the specific value for PRC frame system;
$d_{re}$ – the relative storey displacement determined in the seismic load combination;
$d_{re}^{X}$ – the relative storey displacement on the global X direction of the structural system, determined in the seismic load combination;
$d_{re}^{Y}$ – the relative storey displacement on the global Y direction of the structural system, determined in the seismic load combination;
$d_{r,a}^{ULS}$ – the admissible value of the relative storey displacement, determined in accordance to Equation 5.

The amplification factor "c" of the lateral displacements is determined (for the existing structure) in accordance with the P100-3 (P100-3, 2019) (see Equation 4). The admissible value of the relative storey displacement for both global directions of the structural system is computed according to Equation 5 (see P100-1 (P100-1, 2013) norm) and Equation 6 ((see (SR EN 1998-1, 2004), (SR EN 1998-1/NA, 2008)).

$$c(T_1)=\begin{cases} 4 \; if \; T_1 \le T_i \\ 4\text{-}3\cdot\dfrac{T_i\text{-}T_1}{T_i\text{-}T_s} \; if \; T_i<T_1<T_s \\ 1 \; if \; T_1 \ge T_s \end{cases} \tag{4}$$

where:
$c(T_1)$ – the dynamic amplification factor of the lateral displacements, corresponding to the fundamental vibration period of the structural system;
$T_1$ – the fundamental vibration period of the structure;
$T_i$ și $T_s$ –limit values established according to Table 6.

$$d_{r,a}^{ULS}=0.025h \tag{5}$$

where:
$d_{r,a}^{ULS}$ – the admissible value of the relative storey displacement;
h – the storey height.

$$d_r\vartheta\le 0.01\cdot h \tag{6}$$

where:
$d_r$ – the relative storey displacement;
h – the storey height;
$\upsilon$ – “the reduction factor which takes into account the smallest return period of the seismic action associated with requirements for limiting degradations” (Sococol *et al.,* 2022), (SR EN 1998-1, 2004), (SR EN 1998-1/NA, 2008).

**Table 6**
*$T_i$ and $T_s$ limit values for the determination of the amplification factor ”c” of lateral displacements (P100-3, 2019)*

| | Building construction period | | | | | |
|---|---|---|---|---|---|---|
| | Before 1963 | | 1963-1981 | | 1981-2005 | |
| $T_c$ (s) | $T_i$ (s) | $T_s$ (s) | $T_i$ (s) | $T_s$ (s) | $T_i$ (s) | $T_s$ (s) |
| 1.6 | 0.50 | 1.30 | 0.40 | 1.20 | 0.25 | 1.10 |
| 1 | 0.40 | 1.10 | 0.25 | 1.00 | 0.20 | 0.80 |
| 0.7 | 0.30 | 0.80 | 0.20 | 0.70 | 0.10 | 0.60 |

Thus, the necessary parameter values in order to determine the ULS lateral displacements, as well as their final values (of displacements) can be observed in a condensed form in Table 7.

**Table 7**
*The determination of lateral displacements at Ultimate Limit State (ULS)*

| Parameter | Value | Source |
|---|---|---|
| $T_1$ (s) | *0.868* | *Modal analysis* |
| $T_i$ (s) | *0.20* | *Table 6 (P100-3, 2019)* |
| $T_s$ (s) | *0.80* | *Table 6 (P100-3, 2019)* |
| $c(T_1)$ | *1* | *Equation 4 (P100-3, 2019)* |
| q | *3.5* | *(P100-3, 2019)* |
| υ | *0.4* | *(SR EN 1998-1/NA:2008)* |
| $d_{re}^{X}$ (mm) | *43.746* | *Seismic analysis* |
| $d_{re}^{Y}$ (mm) | *43.635* | *Seismic analysis* |
| $d_{r,X}^{ULS}$ (mm) | *153.111* | *Equation 2 (P100-1, 2013)* |
| $d_{r,Y}^{ULS}$ (mm) | *152.722* | *Equation 3(P100-1, 2013)* |
| $d_{r,a}^{ULS}$ (mm) | *105* | *Equation 5(P100-1, 2013)* |
| $d_{r,a}^{ULS}$ (mm) | *105* | *Equation 6 (SR EN 1998-1:2004)* |

In these conditions, the following two inequalities result (see Table 7, Equation 7 and Equation 8):

$$d_{r,X}^{ULS} > d_{r,a}^{ULS} \tag{7}$$

$$d_{r,Y}^{ULS} > d_{r,a}^{ULS} \tag{8}$$

Basically, the relative storey displacements for both X and Y principal directions of the structural system are beyond the admissible values computed according to P100-1 (P100-1, 2013). This verification has consequences upon the non-structural elements as well as on the spatial cooperation of seismic-resistant structural elements, "which may induce human loss due to their collapse in case of an earthquake with major intensity" (Mkrtychev & Busalova, 2016).

In the case of the existing structure, where PRC beams are pin supported to their marginal areas without "concrete in splice region" (Ali *et al.*, 2022) and the PHC slabs do not ensure an optimal transfer of the inertial forces, it can be considered that these lateral displacements will lead to both a local and global stability loss of the structure and the collapse of the entire structural system upon the occurrence of a major seismic event.

## 5. Proposal and listing of retrofitting and additional elevation (at the request of the owner) of the existing structural system

In these conditions, several "retrofitting solutions of the existing PRC frame system are proposed" (see Fig. 3) (Gkournelos *et al.*, 2021), (Spina *et al.,* 2021), (Markou, 2021), (Huang *et al.,* 2022), (Hu *et al.,* 2019), (Taheri *et al.,* 2020), (Lu *et al.,* 2021), (Lin *et al.,* 2019), (Zhang *et al.,* 2022), (Li & Gan, 2022), (Pang *et al.,* 2021), (Liu *et al.,* 2022) together with several additional elevation solutions, conditioned by the request of the owner (see Fig. 4).

The proposed retrofitting solutions are:

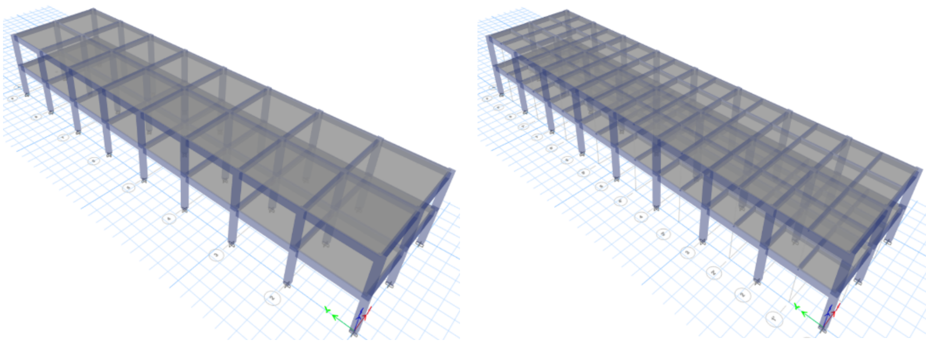
i. the positioning of the 30x60 cm cross-section PRC beams on the bays-direction of the structure, being fixed supported in the rigid RC beam-column joints (see Fig. 3 (a)), (Gulec *et al.,* 2021), (Lv *et al.,* 2021);

ii. the positioning of the 30x60 cm cross-section PRC beams on the bays-direction as well as the positioning of the secondary 25x45 cm cross-section RC beams on both global directions of the structure (see Fig. 3 (b)), (Lu *et al.,* 2021), (Lv *et al.,* 2021);

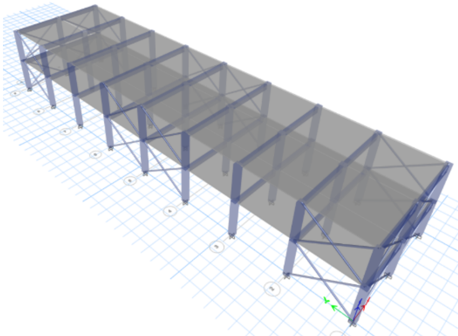
iii. the consideration of vertical "steel braced frames" (Imanpour *et al.,* 2022) made of circular pipe (ex.: Tubo-D159x4), in marginal and central areas of the structural system (see Fig. 3 (c)), (Wang *et al.,* 2021), (Kant *et al.*, 2022);

iv. the consideration of "wing RC shear walls" (Agha *et al.*, 2021), (Todea *et al.,* 2021) with a thickness of 15 cm in certain areas of the structure (see Fig. 3 (d));

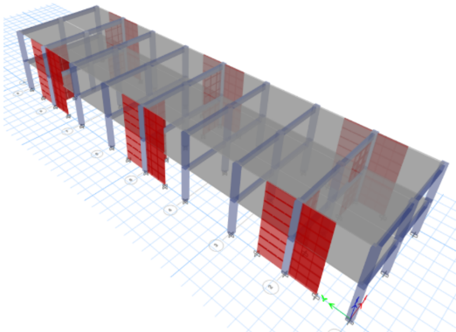
v. the consideration of steel beams (ex.: IPE 550) on the bay direction of the structure and the retrofitting of the PRC columns by confining them on all four external sides with steel plates (see Fig. 3 (e)), (Hassan & Farag, 2021), (Tian *et al.*, 2022), (Kant *et al.*, 2022), (Xu *et al.,* 2018).



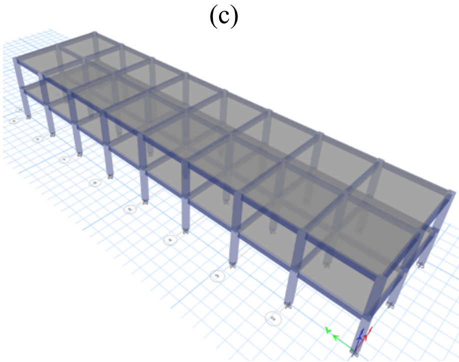
(a) (b)



(c)



(d)

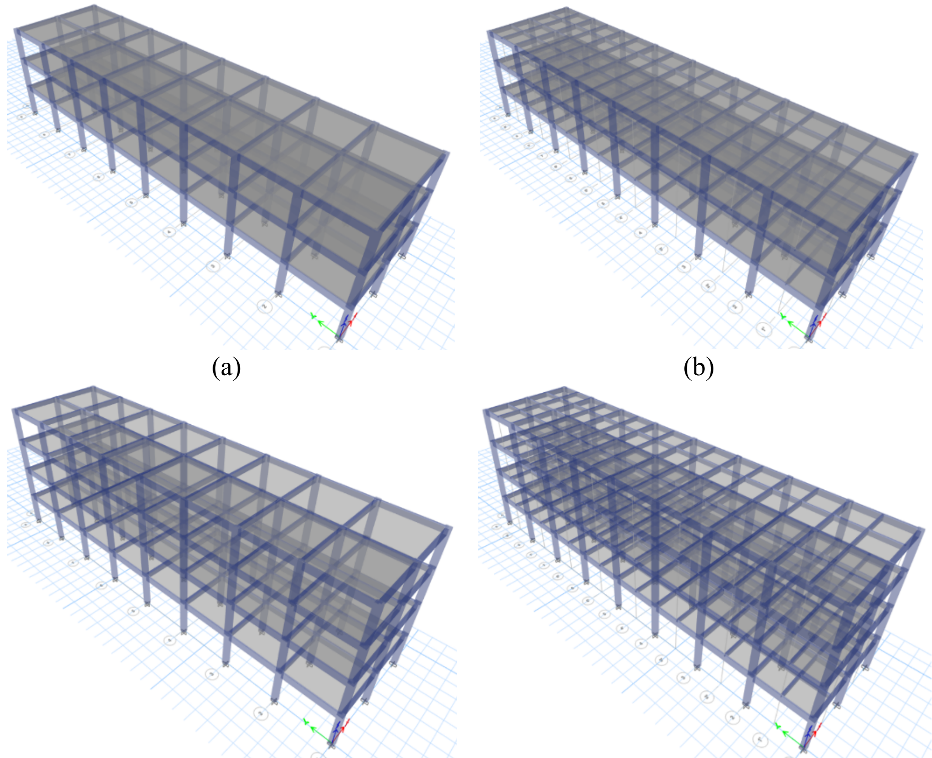


(e)

Fig. 13 –The representation of the retrofitted structural system: (a) with PRC beams on the bay direction of the structure (Model_2); (b) with a grid RC beams system (Model_3); (c) with "vertical steel braced frames" (Imanpour *et al.*, 2022) (Model_4); (d) with RC shear walls (Model_5); (e) with "steel-RC composite columns" (Kant *et al.*, 2022) and steel beams on the bay direction (Model_6). (Note: The existing PRC frame system is Model_1).

The elevation solutions were established in accordance with the owner requirements, as such:

i. elevation with an identical storey with the retrofitted structure storey with 30x60 cm cross-section PRC beams on the bays direction of the structural system (see Fig. 4 (a));
ii. elevation with an identical storey with the retrofitted structure storey with a grid system consisting in 30x60 cm cross-section PRC beams and 25x45 cm RC beams on both global directions of the structural system (see Fig. 4 (b));
iii. elevation with two identical floors with the retrofitted structure storey with 30x60 cm cross-section PRC beams on the bays direction of the structural system (see Fig. 4 (c));
iv. elevation with two identical floors with the retrofitted structure storey with a grid system consisting in 30x60 cm cross-section PRC beams and 25x45 cm RC beams on both global directions of the structural system (see Fig. 4 (d)).



(a) (b) (c) (d)

Fig. 14 –Representation of the structural system: (a) retrofitted with RC beams on the bay direction of the structure and elevated with one storey (Model_7); (b) retrofitted with grid RC beams system and elevated with one storey (Model_8); (c) retrofitted with RC beams on the bay direction of the structure and elevated with two stories (Model_9); (d) retrofitted with grid RC beams system and elevated with two stories (Model_10). (Note: The existing PRC frame system is Model_1).

The discussion of the results with a comparison between retrofitting solutions in terms of lateral displacements and modal analysis will be done in the second part of this research study.

## 6. Conclusions

The present research study highlights and presents the existence of several types of seismic-resistant PRC systems such as: precast moment resisting frames, precast structural wall systems, precast dual wall-frame systems and precast floor and roof diaphragms.

It also presents a classification of precast concrete moment-resisting systems according to the in-plane lateral force resisting mechanisms as well as the two categories of systems: “equivalent monolithic” systems and jointed” systems.

Increased attention has been directed towards lessons from previous earthquakes regarding the seismic response of poorly designed and detailed precast concrete frame buildings. Thus, the main deficiencies recorded by precast concrete frame buildings under seismic loading were observed, namely: improper design and detailing of ductile elements, inadequate diaphragm action, poor joint and connection details and inadequate separation of non-structural elements.

The analytical study regarding the dynamic analysis of the existing PRC frame system, which is a part of the quantitative seismic evaluation, highlights the importance of observing the main structural deficiencies that can damage the optimal seismic response of the structure.

In the current case, it was possible to observe major structural deficiencies in the areas where the PRC columns connect with PRC beams and RC beam-column joints does not exist. Further deficiencies were noted with the PHC slab strips, which require additional top concrete and concrete in splice regions.

In these conditions, an appropriate modal response was registered, having in the first and second eigenmodes of vibration, translation motions of the structural system in the two global (X and Y) directions of the structure.

Furthermore, computations which did not check out properly were made regarding the relative storey displacements of both main directions of the lateral system. These results confirm the necessity of intervening with retrofitting works for the existing structural system, such that the structural system will become earthquake resistant.

The retrofitting and elevation solutions were generally presented in the current paper, with the next articles presenting in detail the obtained results.

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# EVALUAREA SEISMICĂ CANTITATIVĂ A UNEI HALE PREFABRICATE TIP CADRU DE BETON ARMAT ȘI STABILIREA UNOR SOLUȚII DE RETROFITARE. PARTEA I – ANALIZA DINAMICĂ A SISTEMULUI STRUCTURAL EXISTENT

Structurile prefabricate tip cadru de beton armat proiectate și executate în perioada comunistă în România dețin un loc de frunte în lista clădirilor industriale/ comerciale/ sociale/ culturale. În acest studiu de cercetare se analizează o structură de acest tip. În partea introductivă a studiului se evidențiază tipurile de sisteme prefabricate tip cadru din beton armat specifice zonelor seismice cât și elemente componente ale acestora. În partea a doua se arată principalele deficiențe structurale ale sistemelor prefabricate solicitate la cutremure de pământ. În capitolele următoare se dezvoltă analiza dinamică aferentă structurii studiate cât și o verificare a rigidității laterale. Astfel, s-a constatat faptul că structura prefabricată tip cadru din beton armat dezvoltă deplasări laterale superioare limitei admisibile. În aceste condiții, se propun și câteva soluții de retrofitare a sistemului structural utilizând contravântuire metalice verticale, pereți structurali din beton armat, stâlpi armați cu armătură rigidă (din oțel laminat – beton armat) și grinzi metalice pe direcția traveelor.