***Translation from Romanian***

# Technical expertise

**on greenhouses and annex buildings located in the courtyard of the Librecht House**

within the project

**"Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House – (historical monument, LMI B-II-M-A-19107 code)"**

## TECHNICAL EXPERTISE IDENTIFICATION DATA

**Objective**: Building C7, C8, C10-C11, C12, C13, C15, C16, C17, C18, C20, C22, C24, C25, C26, C27, C28-C29 (according to the topographic plan)

**Study beneficiary**: Bucharest University

**Real estate location**: Bucharest Municipality, Dionisie Lupu Street, no. 46

**Internal project number:** 363/2024

**Phase:** technical expertise

**Date:** December 2024

**Revision:** **00**

**M.D.R.A.P.; M.C.C. CERTIFIED TECHNICAL EXPERT**
**ENG. IOAN ROTARESCU**

**M.D.R.A.P. CERTIFIED TECHNICAL EXPERT**
**ENG. DANIEL DIACONU**

ELABORATION TEAM

**Eng. NICULI Bogdan-Alin | Eng. NICULI Georgia**

# Contents

| Elaborated, **Technical expert:** | eng. IOAN ROTARESCU |
|---|---|
| **Signature:** | |

| Elaborated, **Technical expert:** | eng. DANIEL DIACONU |
|---|---|
| **Signature:** | |

Eng. Niculi Bogdan-Alin

**- 12.2024 -**

# 1. TECHNICAL EXPERTISE DATA

## 1.1 Titles and Signatures Page

### 1.1.1 Worksheet and signature list

Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)”

Address: Bucharest Municipality, Dionisie Lupu Street, no. 46

Design phase: technical expertise

## Signatures list

MLPAT Certified Technical Expert: Eng. Rotarescu Ioan ..........................................
Attestation certificate number: E328 of 29.03.1993 ..........................................

MCC Certified Expert: Eng. ROTARESCU IOAN ..........................................
Attestation certificate: 87E of 04.07.2006 ..........................................

Requirements: A1 ..........................................

MLPAT Certified Technical Expert: Eng. DIACONU DANIEL ..........................................
Attestation certificate number: E336 of 08.06.1993 ..........................................
Expertise number: ..........................................
Requirements: A2 ..........................................

### 1.1.2 Contributors

Eng. Niculi Bogdan-Alin ........ | Eng. Niculi Georgia ......
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## 1.2 SYNTHETIC REPORT – BUILDING C7

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| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | |
| Expertise date | 12.2024 | | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | |
| Importance category (G.D. 766/1997) | | | | C |
| Importance class and earthquake exposure (P100-1): | | | | III |
| Year of construction: | | | | 1852 |
| Building function: | guard building | | | |
| Total height above ground (m) | 4.77 | Number of floors: | | G |
| Built area (sq m): | 38.00 | Gross built up area (sq m): | | 38.00 |
| Structural system: | The building's resistance structure is made up of:<br>- foundations are made of brick masonry placed on a base of raw stone masonry in various heights and thicknesses.;<br>- structural walls made of solid ceramic brick masonry without reinforced concrete cores, without reinforced concrete belts above them;<br>- wooden floor slab over the ground floor;<br>- wooden structure frame; | | | |
| Non-structural components | - | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **44.00** | | | |
| Degree of structural damage R2: | **68.12** | | | |
| Degree of seismic structural insurance R3: | **53.00** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation): | **Yes** | No | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** |

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| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.3 SYNTHETIC REPORT – BUILDING C8

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | | | | |
| Expertise date | 12.2024 | | | | | | |
| Technical expert | Eng. Ioan Rotarescu | | Badge | E328 of 29.03.1993 | | | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | | | | |
| Importance category (G.D. 766/1997) | | | | | | C | |
| Importance class and earthquake exposure (P100-1): | | | | | | III | |
| Year of construction: | | | | | | 1971 | |
| Building function: | warehouse | | | | | | |
| Total height above ground (m) | 4.73 | | Number of floors: | | | G | |
| Built area (sq m): | 96.00 | | Gross built up area (sq m): | | | 96.00 | |
| Structural system: | The building's resistance structure is made up of: - continuous concrete foundations 45 cm thick;<br>- structural walls made of solid ceramic brick masonry without reinforced concrete cores, without reinforced concrete belts above them;<br>- wooden floor slab over the ground floor;<br>- wooden structure frame; | | | | | | |
| Non-structural components | - | | | | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | | 70 % | SLU | | 20 % | |
| Verification at the ultimate limit state: | 1 | | **2** | | 3 | | |
| Degree of fulfillment of the seismic design conditions, R1: | **41.00** | | | | | | |
| Degree of structural damage R2: | **57.50** | | | | | | |
| Degree of seismic structural insurance R3: | **54.00** | | | | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | | **II** | ~~III~~ | | ~~IV~~ | |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are not completed. | | | | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are not completed. | | | | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | | | | |
| Necessity of intervention works (consolidation): | **Yes** | | | No | | | |
| Seismic risk class after intervention works | ~~I~~ | | ~~II~~ | ~~III~~ | | **IV** | |

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|---|---|
| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.4 SYNTHETIC REPORT – BUILDING C10-C11

<table>
<tr><td colspan="8">Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)”</td></tr>
<tr><td colspan="8">Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures</td></tr>
<tr><td colspan="2">Expertise date</td><td colspan="6">12.2024</td></tr>
<tr><td colspan="2">Technical expert</td><td colspan="2">Eng. Daniel Diaconu</td><td colspan="2">Badge</td><td colspan="2">E336 of 08.06.1993</td></tr>
<tr><td colspan="2">Address:</td><td colspan="6">Bucharest Municipality, Dionisie Lupu Street, no. 46</td></tr>
<tr><td colspan="6">Importance category (G.D. 766/1997)</td><td colspan="2">C</td></tr>
<tr><td colspan="6">Importance class and earthquake exposure (P100-1):</td><td colspan="2">III</td></tr>
<tr><td colspan="6">Year of construction:</td><td colspan="2">1966 - 1971</td></tr>
<tr><td colspan="2">Building function:</td><td colspan="6">covered terrace</td></tr>
<tr><td colspan="2">Total height above ground (m)</td><td>4.61</td><td colspan="3">Number of floors:</td><td colspan="2">G</td></tr>
<tr><td colspan="2">Built area (sq m):</td><td>282</td><td colspan="3">Gross built up area (sq m):</td><td colspan="2">282</td></tr>
<tr><td>Structural system:</td><td colspan="7">The building's resistance structure is made up of:<br>- the poles are made of 4 L40x5 mm profiles, stiffened between them<br>- the beams are of the lattice beam type with lower and upper soles of 2 L40x5 mm profiles and diagonals of 8 mm bars<br>- the covering is made of corrugated sheet metal<br>- at the upper part of the lattice beams the structure is braced at the ends</td></tr>
<tr><td colspan="2">Non-structural components</td><td colspan="6">-</td></tr>
<tr><td colspan="4">Seismic action (probability of exceeding in 50 years)</td><td>SLS</td><td>70 %</td><td>SLU</td><td>20 %</td></tr>
<tr><td colspan="4">Verification at the ultimate limit state:</td><td colspan="2">1</td><td>**2**</td><td>3</td></tr>
<tr><td colspan="4">Degree of fulfillment of the seismic design conditions, R1:</td><td colspan="4">**58.00**</td></tr>
<tr><td colspan="4">Degree of structural damage R2:</td><td colspan="4">**58.00**</td></tr>
<tr><td colspan="4">Degree of seismic structural insurance R3:</td><td colspan="4">**55.00**</td></tr>
<tr><td colspan="4">Seismic risk class in which the construction was classified:</td><td>~~I~~</td><td>**II**</td><td>~~III~~</td><td>~~IV~~</td></tr>
<tr><td colspan="2">Description of the seismic risk class:</td><td colspan="6">Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake</td></tr>
<tr><td colspan="2">Verification at the serviceable limit state:</td><td colspan="6">Verifications of relative level displacements at SLS are not completed.</td></tr>
<tr><td colspan="2">Verification at the ultimate limit state:</td><td colspan="6">Verifications of relative level displacements at SLU are not completed.</td></tr>
<tr><td colspan="2">Conclusions:</td><td colspan="6">Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.**</td></tr>
<tr><td colspan="4">Necessity of intervention works (consolidation):</td><td colspan="2">**Yes**</td><td colspan="2">No</td></tr>
<tr><td colspan="4">Seismic risk class after intervention works</td><td>~~I~~</td><td>~~II~~</td><td>**III**</td><td>~~IV~~</td></tr>
</table>

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|---|---|
| Elaborated,<br>**Technical expert:** | ENG. DANIEL DIACONU |
| **Signature:** | |

## 1.5 SYNTHETIC REPORT – BUILDING C12

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|---|---|---|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | | | | |
| Expertise date | 12.2024 | | | | | | |
| Technical expert | Eng. Daniel Diaconu | | Badge | | E336 of 08.06.1993 | | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | | | | |
| Importance category (G.D. 766/1997) | | | | | C | | |
| Importance class and earthquake exposure (P100-1): | | | | | III | | |
| Year of construction: | | | | | > 1991 | | |
| Building function: | repository | | | | | | |
| Total height above ground (m) | 2.88 | | Number of floors: | | G | | |
| Built area (sq m): | 43 | | Gross built up area (sq m): | | 43 | | |
| Structural system: | The building's resistance structure is made up of:<br>- the building has no foundations, the metal structure of the warehouse is unloaded through a concrete slab;<br>- metal frame made of RHS 60x30 profiles | | | | | | |
| Non-structural components | - | | | | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % | | | |
| Verification at the ultimate limit state: | 1 | **2** | 3 | | | | |
| Degree of fulfillment of the seismic design conditions, R1: | **49.00** | | | | | | |
| Degree of structural damage R2: | **41.00** | | | | | | |
| Degree of seismic structural insurance R3: | - | | | | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ | | | |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | | | | |
| Necessity of intervention works (consolidation): | **Yes** | | No | | | | |
| Seismic risk class after intervention works**(new building)** | ~~I~~ | ~~II~~ | ~~III~~ | **IV** | | | |

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|---|---|
| Elaborated,<br>**Technical expert:** | ENG. DANIEL DIACONU |
| **Signature:** | |

## 1.6 SYNTHETIC REPORT – BUILDING C13

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| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | |
| Expertise date | 12.2024 | | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | |
| Importance category (G.D. 766/1997) | | | | C |
| Importance class and earthquake exposure (P100-1): | | | | III |
| Year of construction: | | | | 1968-1971 |
| Building function: | kitchen | | | |
| Total height above ground (m) | 4.90 | Number of floors: | | G |
| Built area (sq m): | 256 | Gross built up area (sq m): | | 256 |
| Structural system: | The building's resistance structure is made up of: - isolated foundations with balancing beams;<br>- structural walls made of ceramic brick masonry filled with reinforced concrete cores and reinforced concrete belts above them;<br>- the floor above the ground floor is made of reinforced concrete;<br>- non-circulatory terrace; | | | |
| Non-structural components | - | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **72.00** | | | |
| Degree of structural damage R2: | **81.00** | | | |
| Degree of seismic structural insurance R3: | **86.00** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | ~~II~~ | **III** | ~~IV~~ |
| Description of the seismic risk class: | Building where structural degradation is expected that does not significantly affect structural safety, but where degradation of non-structural elements may be important | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs III** seismic risk class. **No intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation): | Yes | **No** | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | **III** | ~~IV~~ |

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| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.7 SYNTHETIC REPORT – BUILDING C15

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| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | |
| Expertise date | 12.2024 | | | |
| Technical expert | Eng. Daniel Diaconu | Badge | E336 of 08.06.1993 | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | |
| Importance category (G.D. 766/1997) | | | | C |
| Importance class and earthquake exposure (P100-1): | | | | III |
| Year of construction: | | | | 1911 - 1971 |
| Building function: | basin | | | |
| Total height above ground (m) | 0.83 | Number of floors: | | G |
| Built area (sq m): | 28 | Gross built up area (sq m): | | 28 |
| Structural system: | The building's resistance structure is made up of: - concrete foundations and brick masonry<br>- masonry basin walls – $h_{med}$ = 85-90 cm | | | |
| Non-structural components | - | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **41.00** | | | |
| Degree of structural damage R2: | **72.00** | | | |
| Degree of seismic structural insurance R3: | **-** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | ~~II~~ | **III** | ~~IV~~ |
| Description of the seismic risk class: | Building where structural degradation is expected that does not significantly affect structural safety, but where degradation of non-structural elements may be important | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs III** seismic risk class. **No intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation): | Yes | **No** | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | **III** | ~~IV~~ |

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| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.8 SYNTHETIC REPORT – BUILDING C16

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| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | |
| Expertise date | 12.2024 | | | |
| Technical expert | Eng. Daniel Diaconu | Badge | E336 of 08.06.1993 | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | |
| Importance category (G.D. 766/1997) | | | C | |
| Importance class and earthquake exposure (P100-1): | | | III | |
| Year of construction: | | | 1911 - 1971 | |
| Building function: | gazebo | | | |
| Total height above ground (m) | 3.70 | Number of floors: | G | |
| Built area (sq m): | 12 | Gross built up area (sq m): | 12 | |
| Structural system: | - metal structure made of 8 mm bars | | | |
| Non-structural components | - | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **67.00** | | | |
| Degree of structural damage R2: | **48.00** | | | |
| Degree of seismic structural insurance R3: | **-** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | ~~II~~ | **III** | ~~IV~~ |
| Description of the seismic risk class: | Building where structural degradation is expected that does not significantly affect structural safety, but where degradation of non-structural elements may be important | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs III** seismic risk class. **No intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation): | Yes | **No** | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | **III** | ~~IV~~ |

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|---|---|
| Elaborated,<br>**Technical expert:** | ENG. DANIEL DIACONU |
| **Signature:** | |

## 1.9 SYNTHETIC REPORT – BUILDING C17

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| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | |
| Expertise date | 12.2024 | | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | |
| Importance category (G.D. 766/1997) | C | | | |
| Importance class and earthquake exposure (P100-1): | III | | | |
| Year of construction: | 1911 - 1964 | | | |
| Building function: | kitchen | | | |
| Total height above ground (m) | 6.55 | Number of floors: | G | |
| Built area (sq m): | 378 | Gross built up area (sq m): | 378 | |
| Structural system: | The building's resistance structure is made up of: - continuous concrete foundations;<br>- structural walls made of solid ceramic brick masonry without reinforced concrete cores, with reinforced concrete belts above them;<br>- wooden floor slab over the ground floor;<br>- wooden structure frame; | | | |
| Non-structural components | - | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **46.00** | | | |
| Degree of structural damage R2: | **79.00** | | | |
| Degree of seismic structural insurance R3: | **45.00** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation): | **Yes** | No | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** |

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|---|---|
| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.10 SYNTHETIC REPORT – BUILDING C18

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|---|---|---|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | | | | |
| Expertise date | 12.2024 | | | | | | |
| Technical expert | Eng. Daniel Diaconu | | Badge | E336 of 08.06.1993 | | | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | | | | |
| Importance category (G.D. 766/1997) | | | | C | | | |
| Importance class and earthquake exposure (P100-1): | | | | III | | | |
| Year of construction: | | | | > 1974 | | | |
| Building function: | Shed | | | | | | |
| Total height above ground (m) | - | | Number of floors: | G | | | |
| Built area (sq m): | 99 | | Gross built up area (sq m): | 99 | | | |
| Structural system: | The building's resistance structure is made up of:<br>- metal poles made of 120 mm circular pipe and main beams made of U profiles - secondary metal beams made of C18 wood with a section of 10x10 cm<br>The C18 building was built after the C17 and C20 buildings, to facilitate the connection between the two buildings, this is of a provisional nature. | | | | | | |
| Non-structural components | - | | | | | | |
| Seismic action (probability of exceeding in 50 years) | | | SLS | 70 % | SLU | 20 % | |
| Verification at the ultimate limit state: | | | **1** | 2 | 3 | | |
| Degree of fulfillment of the seismic design conditions, R1: | | | 25 | | | | |
| Degree of structural damage R2: | | | 35 | | | | |
| Degree of seismic structural insurance R3: | | | - | | | | |
| Seismic risk class in which the construction was classified: | | | I | ~~II~~ | ~~III~~ | ~~IV~~ | |
| Description of the seismic risk class: | Building with high risk of collapse during earthquakes with intensities corresponding to the seismic calculation areas (design earthquake) | | | | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs I** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | | | | |
| Necessity of intervention works (consolidation): | | | **Yes** | | No | | |
| Seismic risk class after intervention works | | | ~~I~~ | ~~II~~ | ~~III~~ | **IV** | |

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|---|---|
| Elaborated,<br>**Technical expert:** | ENG. DANIEL DIACONU |
| **Signature:** | |

## 1.11 SYNTHETIC REPORT – BUILDING C20

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|---|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | | |
| Expertise date | 12.2024 | | | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 | | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | | |
| Importance category (G.D. 766/1997) | | | | C | |
| Importance class and earthquake exposure (P100-1): | | | | III | |
| Year of construction: | | | | > 1974 | |
| Building function: | kitchen | | | | |
| Total height above ground (m) | 5.35 | Number of floors: | G | | |
| Built area (sq m): | 282 | Gross built up area (sq m): | 282 | | |
| Structural system: | The building's resistance structure is made up of: - plain concrete foundations<br>- solid brick masonry walls, the building extension is made of vaults<br>- the poles and belts were discovered only on the initial body, made of solid brick<br>- the floor and frame are made of wood | | | | |
| Non-structural components | - | | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % | |
| Verification at the ultimate limit state: | 1 | **2** | 3 | | |
| Degree of fulfillment of the seismic design conditions, R1: | **50.00** | | | | |
| Degree of structural damage R2: | **79.30** | | | | |
| Degree of seismic structural insurance R3: | **53.00** | | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ | |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | | |
| Necessity of intervention works (consolidation): | **Yes** | No | | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** | |

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| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.12 SYNTHETIC REPORT – BUILDING C22

| | | | | |
|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | |
| Expertise date | 12.2024 | | | |
| Technical expert | Eng. Daniel Diaconu | Badge | E336 of 08.06.1993 | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | |
| Importance category (G.D. 766/1997) | | | | C |
| Importance class and earthquake exposure (P100-1): | | | | III |
| Year of construction: | | | | > 1991 |
| Building function: | repository | | | |
| Total height above ground (m) | 2.66 | Number of floors: | | G |
| Built area (sq m): | 83 | Gross built up area (sq m): | | 83 |
| Structural system: | The building's resistance structure is made up of:<br>- the building has no foundations, the metal structure of the warehouse is unloaded through a concrete slab;<br>- metal frame made of RHS 30x30 profiles | | | |
| Non-structural components | - | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | | **1** | 2 | 3 |
| Degree of fulfillment of the seismic design conditions, R1: | **41.00** | | | |
| Degree of structural damage R2: | **-** | | | |
| Degree of seismic structural insurance R3: | **-** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ |
| Description of the seismic risk class: | Building with high risk of collapse during the design earthquake corresponding to the ultimate limit state; | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are not completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are not completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation):<br>**The building is proposed to be demolished.** | **Yes** | | No | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** |

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|---|---|
| Elaborated,<br>**Technical expert:** | ENG. DANIEL DIACONU |
| **Signature:** | |

## 1.13 SYNTHETIC REPORT – BUILDING C24

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|---|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | | |
| Expertise date | 12.2024 | | | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 | | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | | |
| Importance category (G.D. 766/1997) | | | | C | |
| Importance class and earthquake exposure (P100-1): | | | | III | |
| Year of construction: | | | | 1911 - 1964 | |
| Building function: | Annex greenhouse | | | | |
| Total height above ground (m) | 2.66 | Number of floors: | | G | |
| Built area (sq m): | 83 | Gross built up area (sq m): | | 83 | |
| Structural system: | The building's resistance structure is made up of:<br>- continuous foundations made of stone and brick masonry. Foundations are made of brick masonry placed on a base of raw stone masonry in various heights and thicknesses.;<br>- structural walls made of solid ceramic brick masonry without reinforced concrete cores, without reinforced concrete belts above them;<br>- the floor is made of wood and metal; | | | | |
| Non-structural components | - | | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % | |
| Verification at the ultimate limit state: | 1 | **2** | 3 | | |
| Degree of fulfillment of the seismic design conditions, R1: | **66.00** | | | | |
| Degree of structural damage R2: | **68.24** | | | | |
| Degree of seismic structural insurance R3: | **61** | | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ | |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | | |
| Necessity of intervention works (consolidation): | **Yes** | | No | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** | |

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| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.14 SYNTHETIC REPORT – BUILDING C25

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|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | |
| Expertise date | 12.2024 | | | |
| Technical expert | Eng. Daniel Diaconu | Badge | E336 of 08.06.1993 | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | |
| Importance category (G.D. 766/1997) | C | | | |
| Importance class and earthquake exposure (P100-1): | III | | | |
| Year of construction: | 1991 - 2023 | | | |
| Building function: | greenhouse | | | |
| Total height above ground (m) | 4.77 | Number of floors: | G | |
| Built area (sq m): | 441 | Gross built up area (sq m): | 441 | |
| Structural system: | The building's resistance structure is made up of: - 60x30x4 rectangular poles<br>- 30x30x3 bracing<br>- 20x20x3 purlin | | | |
| Non-structural components | - | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **63.00** | | | |
| Degree of structural damage R2: | **54.00** | | | |
| Degree of seismic structural insurance R3: | **60.00** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are not completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are not completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation): | **Yes** | | No | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** |

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|---|---|
| Elaborated,<br>**Technical expert:** | ENG. DANIEL DIACONU |
| **Signature:** | |

## 1.15 SYNTHETIC REPORT – BUILDING C26

| | | | |
|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | |
| Expertise date | 12.2024 | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | |
| Importance category (G.D. 766/1997) | | | C |
| Importance class and earthquake exposure (P100-1): | | | III |
| Year of construction: | | | 1911 - 1964 |
| Building function: | Annex greenhouse | | |
| Total height above ground (m) | 3.20 | Number of floors: | G |
| Built area (sq m): | 85 | Gross built up area (sq m): | 85 |
| Structural system: | The building's resistance structure is made up of:<br>- continuous foundations made of stone and brick masonry. Foundations are made of brick masonry placed on a base of raw stone masonry in various heights and thicknesses.;<br>- structural walls made of solid ceramic brick masonry without reinforced concrete cores, without reinforced concrete belts above them;<br>- the floor is made of wood and metal; | | |
| Non-structural components | - | | |

| | | | | |
|---|---|---|---|---|
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **65.00** | | | |
| Degree of structural damage R2: | **54.67** | | | |
| Degree of seismic structural insurance R3: | **61** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | |
| Necessity of intervention works (consolidation): | **Yes** | | No | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** |

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|---|---|
| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.16 SYNTHETIC REPORT – BUILDING C27

| | | | | | |
|---|---|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | | | |
| Expertise date | 12.2024 | | | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 | | |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | | | |
| Importance category (G.D. 766/1997) | | | | C | |
| Importance class and earthquake exposure (P100-1): | | | | III | |
| Year of construction: | | | | 1911 - 1964 | |
| Building function: | Annex greenhouse | | | | |
| Total height above ground (m) | 1.34 | Number of floors: | | G | |
| Built area (sq m): | 45 | Gross built up area (sq m): | | 45 | |
| Structural system: | The building's resistance structure is made up of:<br>- continuous foundations made of stone and brick masonry. Foundations are made of brick masonry placed on a base of raw stone masonry in various heights and thicknesses.;<br>- structural walls made of solid ceramic brick masonry without reinforced concrete cores, without reinforced concrete belts above them;<br>- the floor is made of wood and metal; | | | | |
| Non-structural components | - | | | | |
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % | |
| Verification at the ultimate limit state: | 1 | **2** | 3 | | |
| Degree of fulfillment of the seismic design conditions, R1: | **66.00** | | | | |
| Degree of structural damage R2: | **68.12** | | | | |
| Degree of seismic structural insurance R3: | **61** | | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | **II** | ~~III~~ | ~~IV~~ | |
| Description of the seismic risk class: | Building for which the probability of collapse is reduced, but for which major structural degradation is expected upon the incidence of the design earthquake | | | | |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are completed. | | | | |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are completed. | | | | |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs II** seismic risk class. **Intervention works are required to consolidate the buildings.** | | | | |
| Necessity of intervention works (consolidation): | **Yes** | | No | | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** | |

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|---|---|
| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

## 1.17 SYNTHETIC REPORT – BUILDING C28-C29

| | | | |
|---|---|---|---|
| Name of the work: ”Services for preparing a technical expertise of the annexes and the greenhouse located in the courtyard of the Librecht - Filipescu House, now University House (historical monument, LMI B-II-M-A-19107 code)” | | | |
| Purpose of the expertise: Seismic assessment of the building and arrangement of intervention measures | | | |
| Expertise date | 12.2024 | | |
| Technical expert | Eng. Ioan Rotarescu | Badge | E328 of 29.03.1993 |
| Address: | Bucharest Municipality, Dionisie Lupu Street, no. 46 | | |
| Importance category (G.D. 766/1997) | | | C |
| Importance class and earthquake exposure (P100-1): | | | III |
| Year of construction: | | | 1964 - 1971 |
| Building function: | repository | | |
| Total height above ground (m) | 4,05 \| 5,60 | Number of floors: | G |
| Built area (sq m): | 145 \| 21 | Gross built up area (sq m): | 145 \| 21 |
| Structural system: | The building's resistance structure is made up of: - continuous foundations made of masonry and concrete;<br>- structural walls made of solid ceramic brick masonry with reinforced concrete cores and (partial) reinforced concrete belts above them;<br>- floor above the ground floor;<br>- wooden structure frame; | | |
| Non-structural components | - | | |

| | | | | |
|---|---|---|---|---|
| Seismic action (probability of exceeding in 50 years) | SLS | 70 % | SLU | 20 % |
| Verification at the ultimate limit state: | 1 | **2** | 3 | |
| Degree of fulfillment of the seismic design conditions, R1: | **62.00** | | | |
| Degree of structural damage R2: | **83.00** | | | |
| Degree of seismic structural insurance R3: | **66.00** | | | |
| Seismic risk class in which the construction was classified: | ~~I~~ | II | **III** | ~~IV~~ |

| | |
|---|---|
| Description of the seismic risk class: | Building where structural degradation is expected that does not significantly affect structural safety, but where degradation of non-structural elements may be important |
| Verification at the serviceable limit state: | Verifications of relative level displacements at SLS are not completed. |
| Verification at the ultimate limit state: | Verifications of relative level displacements at SLU are not completed. |
| Conclusions: | Based on the results of the qualitative and quantitative assessment, the resistance structure falls into the **Rs III** seismic risk class. **No intervention works are required to consolidate the buildings.** |

| | | | | |
|---|---|---|---|---|
| Necessity of intervention works (consolidation):<br>**Seismic compliance interventions are needed** | **Yes** | | No | |
| Seismic risk class after intervention works | ~~I~~ | ~~II~~ | ~~III~~ | **IV** |

| | |
|---|---|
| Elaborated,<br>**Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

# 2. EVALUATION REPORT 2.1

## Purpose of the expertise

In accordance with the contract concluded between the Bucharest University and SC GEOSTRUCT SRL through Eng. Ioan Rotarescu, MLPAT expert no. E328 of 29.03.1993, MC expert no. 87E of 04.07.2006 and expert Daniel Diaconu, MLPAT expert no. E336 of 08.06.1993, carried out the technical expertise on the resistance structure of buildings C7, C8, C10-C11, C12, C13, C15, C16, C17, C18, C20, C22, C24, C25, C26, C27, C28-C29 (according to the topographic plan) within the premises of the Librecht – Filipescu House, today the University House – LMI B-II-M-A-19107, in order to establish:

- **- the degree of fulfillment of the structural compliance conditions** and composition of the load-bearing elements and of the construction rules for structures that take on the effect of seismic action;

**- the degree of structural damage** as a measure of the degradation produced by seismic action or other cause;

**- the resistance and deformability capacity of the** building's resistance structure, analytically determined at the base level of the structure in relation to the seismic requirements;

**- classification in a seismic risk class and decision to intervene** on the resistance structure**.**

The analyzed buildings are not included in the list of historical monuments in Bucharest, but are located in the protection area of historical monuments, LMI B-II-M-A-19107 code.

The activity carried out to evaluate the building, the results of the examination and studies carried out for the evaluation, the conclusions regarding the seismic safety of the building, the need for intervention works and, where applicable, the nature and proportions of these works, are presented in this seismic evaluation report of the construction, part of the technical expertise.

The seismic assessment of the building is done for the fundamental performance requirements, defined for new buildings in the code P 100-1/2013. The value considered for the IMR is 225 years (20% probability of exceedance in 50 years) for checks at the Ultimate Limit State and 40 years (20% probability of exceedance in 10 years) for checks at the Serviceability Limit State.

The technical expertise refers to the strength structure of the building and the report is prepared in accordance with the legislation and technical regulations in force.

## 2.2 Technical regulations

### 2.2.1 Technical regulations in force at the time of construction of the building

The evolution of anti-seismic design standards in Romania:

- Instructions for preventing damage to buildings due to earthquakes and for restoring damaged ones were developed after the 1940 earthquake, the official edition coming into force in 1945.
- Conditional standard for the design of civil and industrial buildings in seismic regions P13-63, with seismic zoning map STAS 2923-63;
- Standard for the design of civil and industrial buildings in seismic regions P 13-70, with seismic zoning map STAS 2923-63;
- Standard for the anti-seismic design of housing, social-cultural, agro-zootechnical and industrial buildings P100-78 (81), with seismic zoning map STAS 11100/1-77;
- Standards for seismic design of residential, social-cultural, agricultural and industrial buildings, Indicative P 100-92 (96), with their own zoning maps;

- Seismic design codes P 100-1/2004 (2006), with their own zoning maps; - Seismic design code P 100-1/2013, with their own zoning map.

Evolution of design standards for masonry structures:

- Technical instructions regarding construction measures for buildings with load-bearing walls made of brick masonry, located in seismic zones, Indicative P2-1962;
- Norms regarding the composition, calculation and execution of masonry structures, Indicative P2-1975;
- Norms regarding the composition, calculation and execution of masonry structures, Indicative P2-1985;

- Design code for masonry structures, Indicative CR6-2006; - Design code for masonry structures, Indicative CR6-2013;

Evolution of norms for the evaluation of existing buildings:

- Standard for the anti-seismic design of housing, social cultural, agricultural and industrial buildings, indicative P100-92 (Chapter 11 – Provisions regarding the evaluation of the level of protection of existing buildings, Chapter 12 – Provisions regarding intervention measures on existing buildings);
- Seismic Design Code – Part III – Provisions for the seismic assessment of existing buildings, Indicative P100-3/2008;
- Seismic Design Code – Part III – Provisions for the seismic assessment of existing buildings, Indicative P100-3/2019;

At the time of the construction of the expertized building (1828), there were no norms or technical regulations, these buildings being executed by craftsmen with empirical knowledge about constructions.

### 2.2.2 Technical prescriptions in force used in the expertise

| Technical regulation indicator – standard | Name of the technical regulation | Approval order | The publication in which it appeared |
|---|---|---|---|
| 1 | 2 | 3 | 4 |
| **Load assessment** | | | |
| **- CR 0-2012** | Design code. Fundamentals of building design | O.M.D.R.T. no. 1.530/23.08.2012 | Official Gazette, p I, no. 647bis/11.09.2012 |
| **- SR EN 1990:2004** | Fundamentals of structural design | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |
| **- SR EN 1990:2004 /NA:2006** | Fundamentals of structural design. National annex | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |
| **- SR EN 1991-1-1/2004** | Specific weights, dead weights, useful loads for buildings | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |
| **- SR EN 1991-1-1:2004 /NA:2006** | Specific weights, dead weights, useful loads for buildings. National Annex | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |
| **- SR EN 1991-1-3:2005** | Snow loads | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |
| **- CR 1-1-3/2012** | Design code for evaluating snow action on structures | O.M.D.R.A.P. no. 1530/3.08.2012 | Official Gazette, p I, no. 704bis/15.10.2012 |
| **- CR 1-1-3/2012 completion** | Design code for evaluating snow action on structures | O.M.D.R.A.P. no. 2.414/01.08.2013* | Official Gazette, p I, no. 555bis/02.09.2013 |
| **- SR EN 1991-1-3:2005 /NA:2006** | Snow data loading. National Annex | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |
| **- SR EN 1991-1-4:2006** | Wind actions | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |
| **- CR 1-1-4/2012** | Design code for the assessment of wind action on structures | O.M.D.R.A.P. no. 1751/21.09.2012 | Official Gazette, p I, no. 704bis/15.10.2012 |
| **- SR EN 1991-1-4:2006 /NB:2007** | Wind actions. National Annex | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 | - |

| | | | |
|---|---|---|---|
| **Technical regulations regarding the design and execution of structural elements – foundations** | | | |
| **- NP 126:2010** | Code of practice for the design and execution of constructions founded on soils with large swellings and contractions (P.U.C.M) | O.M.D.R.T. no. 115/31.05.2012 | Official Gazette, p I, no. 397bis/13.06.2012 |
| **- NP 112-2004** | Normative on the design of direct foundation structures | OMTCT – 275/2005 | M. Of. no. 451- |
| **Technical regulations regarding the design and execution of structural elements – concrete, reinforced concrete and prestressed concrete** | | | |
| **- ST 009-2011** | Technical specification for steel products used as reinforcement: requirements and performance criteria | OMTCT – 180/2005 | M.Of. no. 426-bis/19.05.2005 |
| **- SR EN 1992-1-1:2004** | Eurocode 2: Design of concrete structures. Part 1- 1: General rules and building regulations | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. | - |
| **- SR EN 1992-1-1:2004/NB:2008** | Eurocode 2: Design of concrete structures. Part 1- 1: General rules and building rules. National Annex | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. | - |
| **- SR EN 1992-1-1:2004/AC:2012** | Eurocode 2: Design of concrete structures. Part 1- 1: General rules and building rules. National Annex | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. | - |
| **- SR EN 1992-1-2:2006** | Eurocode 2: Design of concrete structures. Part 1-2: General rules. Calculation of fire behaviour | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. | - |
| **- SR EN 1992-1-2:2006/NA:2009** | Eurocode 2: Design of concrete structures. Part 1-2: General rules. Calculation of fire behaviour. National annex | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. | - |
| **- NE 012/1-2007** | Standard for the production of concrete and the execution of works made of concrete, reinforced concrete and prestressed concrete | M.D.L.P.L. no. 577 / 29.04.2008 | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. 2 /2008 |
| **- NE 012/2-2010** | Standard for the production and execution of concrete, reinforced concrete and prestressed concrete works – Part 2: Execution of concrete works | O.M.D.R.T. no. 2.514/22.11.2010 | Official Gazette, p I, no. 853bis/20.12.2010 |
| **Technical regulations regarding the design and execution of structural elements – concrete, reinforced concrete and prestressed concrete** | | | |
| **P 100-1/2013** | Norm for the anti-seismic design of housing, social-cultural, agricultural and industrial buildings | O.M.D.R.A.P. no. 2.465/08.08.2013 | Official Gazette, p I, no. 558bis/03.09.2013 |
| **P100-1/2013 addition** | Norm for the anti-seismic design of housing, social-cultural, agricultural and industrial buildings | O.M.D.R.A.P. no. 2.956/18.11.2019 | Official Gazette, p I, no. 928/18.11.2019 |
| **P 100-3/2019** | Seismic Design Code, Part III, Provisions for Seismic Assessment of Existing Buildings | O.M.D.R.A.P. no. 2834/2019 | Official Gazette, p I, no. 1003 bis/ 13.12.2019 |
| **Technical regulations regarding the design and execution of structural elements – masonry** | | | |
| **- CR 6-2013** | Design code for masonry structures | O.M.D.R.A.P. no. 2.464/08.08.2013 | Official Gazette, p I, no. 582bis/13.09.2013 |
| **- SR EN 1996** | Design of masonry structures (with National Annex) | Official Gazette, p I , no. 374 of 16/05/2008. B.C.no. | - |

In addition to the standards in force, regulations and specialized literature, the technical expertise is based on the following elements (according to points 2.3, 2.4, 2.5, 2.6, 2.7, 2.8).

### 2.2.3 Legislative regulations in force

When preparing the technical expertise, the following legislative framework was taken into account:

- Law 177/2015 amending and supplementing Law 10/1995 on quality in construction provides that repair works are carried out only on the basis of a technical expertise prepared by a certified technical expert, if they constitute interventions on existing constructions, defined as reconstruction, consolidation, transformation, extension, partial demolition works, as well as repair works.

- Government Ordinance no. 20 of January 27, 1994 on the safety of the existing built stock provides that building owners should act to identify buildings under their ownership or management that present insufficient levels of protection against seismic actions, degradation or damage, to order the technical expertise of the buildings by certified technical experts, in accordance with the technical regulations, to adopt the intervention decision and then to continue the rehabilitation actions based on the conclusions substantiated in the technical expertise report.

- GD 486/93 on increasing the operational safety of constructions and installations that represent high-risk sources, provides for the inventory and hierarchy of constructions depending on the size of the potential danger of damage they represent, and then they will be technically expertized. In a later stage, based on the expert opinions prepared, the technical and economic documentation necessary for the execution of repair, consolidation or modernization works is developed and the execution of the stipulated works is carried out.

- G.D.R. no. 644/1990 on reducing the risk of damage to constructions, which provides for the obligation of owners to request an analysis of the condition of all heritage constructions.
- Decision no. 742/2018 on amending Government Decision no. 925/1995 for the approval of the Regulation on the verification and technical expertise of the quality of projects, the execution of works and constructions
- Government Decision no. 766 of 21 November 1997 for the approval of regulations on the quality of constructions. Regulation on the establishment of the category of importance of constructions.
- Law no. 422/2001 on the protection of historical monuments.

## 2.3 Activities carried out to prepare the expertise

### 2.3.1 Motivation for carrying out the technical expertise

To prepare the expertise, a visual inspection, geometric survey, photographic survey and non-destructive tests were carried out on the masonry and reinforced concrete resistance elements. Also, the structural calculation was performed and the results were interpreted in accordance with the regulatory provisions, in order to formulate the conclusions of the expertise.

### 2.3.2 Study of the technical book of the building

The beneficiary did not provide the Technical Book of the buildings or documentation regarding possible structural interventions.

### 2.3.3 Visit to the building

The technical expert's visit to the site was carried out in August-September 2024. During the visit, the beneficiary allowed access to the analyzed buildings, conducting a visual assessment of the building, the constructive elements and a photographic survey of the degradation (***Annex B – Photographic survey – page 90***).

## 2.4 Data that formed the basis of the technical expertise

This expertise was carried out based on the following information:

- architectural survey
  (Annex A – **Geometric survey, page 88**); -
photographic survey of degradations
  (***Annex B – Photographic survey – page 90);*** -
calculation notes
  (***Anexa C – Calculation summary – page 327***); -
Geotechnical study
  (***ANNEX D – GEOTECHNICAL STUDY – page 402***); -
in situ inspections, structural strength surveys
  (*Error! Reference source not found.* ***– page*** Error! Bookmark not defined.);

## 2.5 Site characteristics

### 2.5.1 Geotechnical conditions of the site

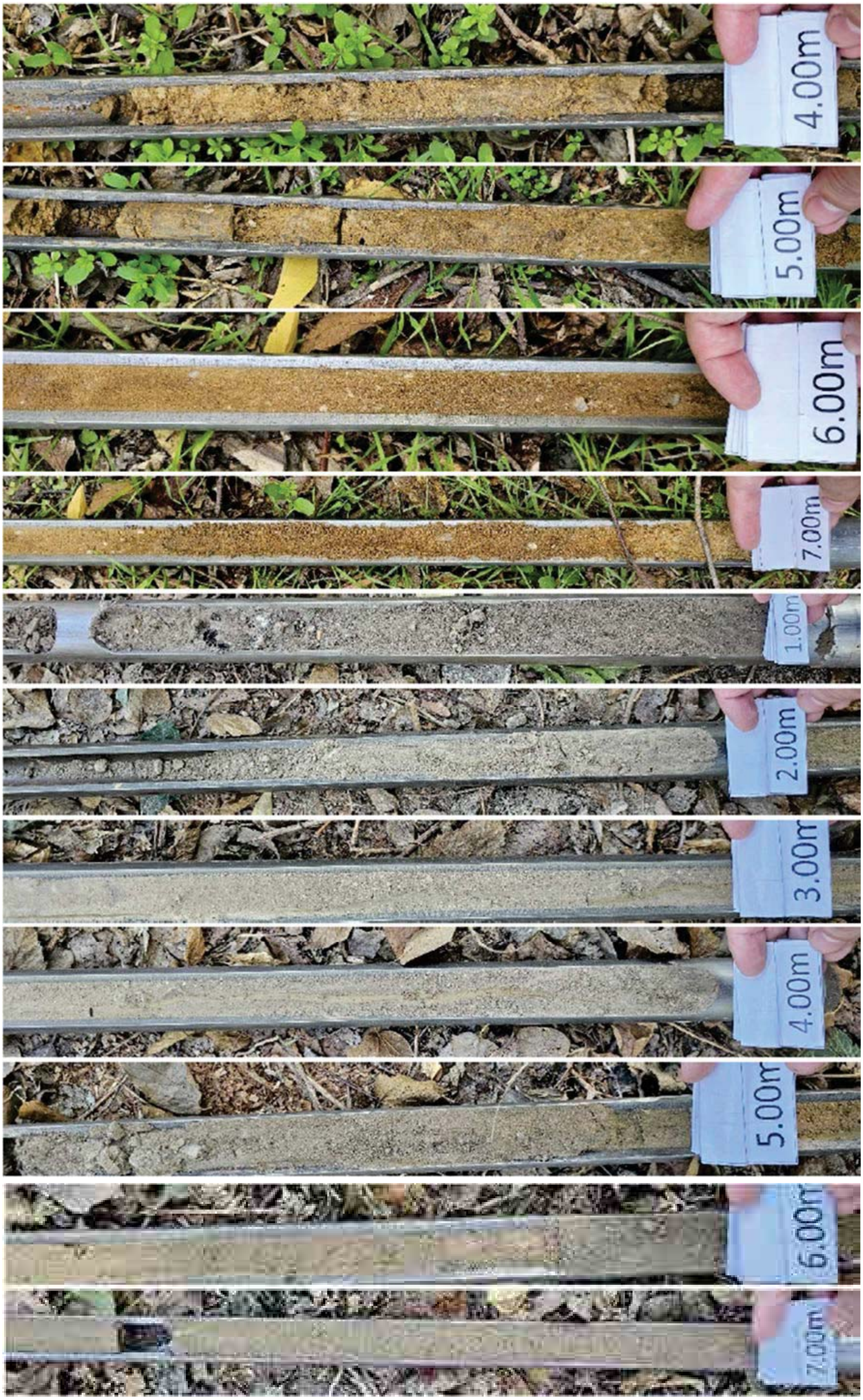
The technical expertise was also based on geotechnical data obtained in September 2024.

***After the start of rehabilitation works, additional investigations will be carried out on the foundation system and the nature of the terrain.***

Disturbed and undisturbed samples were taken from the geotechnical drillings, which were analyzed in the laboratory, highlighting the following stratification:

| Geotechnical investigation | Layer | Layer depth | Layer thicknes | Lithological description |
|---|---|---|---|---|
| **Drilling F01** | 1 | - 1,00 m | 1.00 m | Fillers from construction waste and topsoil |
| | 2 | - 3,70 m | 2.70 m | Brownish sandy silty clay with medium plasticity, soft to hard plastic, with medium compressibility |
| | 3 | - 7,00 m | 3.30 m | Medium gravel with dense coarse sand |
| **Drilling F02** | 1 | - 1,10 m | 1.10 m | Fillers from construction waste and topsoil |
| | 2 | - 7,00 m | 5.90 m | Brownish sandy silty clay with medium plasticity, soft to hard plastic, with medium compressibility |

*Figura 18. Realizare foraje geotehnice*



4.00m
5.00m
6.00m
7.00m
1.00m
2.00m
3.00m
4.00m
5.00m
6.00m
7.00m

**SURVEY S01** (The reference level of the elevations and the depth of the survey were reported to the C.T.A. - considered to be depth 0.00)

**SURVEY S02** (The reference level of the elevations and the depth of the survey were reported to the C.T.A. - considered to be depth 0.00)

**SURVEY S03** (The reference level of the elevations and the depth of the survey were reported to the C.T.A. - considered to be depth 0.00)




Following the analysis of the nature of the samples taken and the primary drilling logs drawn up during the geotechnical prospecting drilling, a heterogeneous lithology was identified for the analyzed site, consisting of the following soil layers:

**Drilling F01-F02**

Under the layer of fills made of construction material debris and topsoil with a thickness varying between 1.00 m-1.30 m, the following stratification was intercepted:

**Layer 1: Brownish sandy silty clay with medium plasticity, soft to hard plastic, medium compressibility with intercalations of organic matter** with a thickness varying between 2.70m-5.70m;

| item no. | Name | Symbol | MU | Values | |
|---|---|---|---|---|---|
| | | | | | |
| 1 | Clay particle size fraction | Cl | % | 26.21 | 28.01 |
| 2 | Dust particle size fraction | Si | % | 50.37 | 51.60 |
| 3 | Sand particle size fraction | S1 | % | 21.55 | 22.45 |
| 4 | Natural moisture | w | % | 17.79 | 221.81 |
| 5 | Upper plasticity limit | $w_L$ | % | 39.64 | 42.53 |
| 6 | Lower plasticity limit | $w_p$ | % | 18.14 | 18.93 |
| 7 | Plasticity index | $I_p$ | % | 21.50 | 23.69 |
| 8 | Consistency index | $I_c$ | - | 0.87 | 1.02 |
| 9 | Density | $\rho$ | g/cm$^3$ | 1.84 | 1.88 |
| 10 | Dry density | $\rho_d$ | g/cm$^3$ | 1.51 | 1.55 |
| 11 | Porosity | n | % | 44.58 | 46.11 |
| 12 | Porosity indices | e | - | 0.80 | 0.86 |
| 13 | Moisture content | $S_r$ | - | 0.70 | 0.71 |
| 14 | Oedometric modulus | $M_{2-3}$ | kPa | 10399 | 10417 |
| 15 | Flooded modulus | $M_{2-3}$ | kPa | - | - |

| | | | | | |
|---|---|---|---|---|---|
| 16 | Specific settlement at 200 kPa | $\varepsilon_{p2}$ | % | 3.8 | 3.8 |
| 17 | Additional settlement | $Im_{300}$ | % | - | - |
| 18 | Inflation pressure | $p_u$ | kPa | - | - |
| 19 | Internal friction angle | $\phi$ | - | 15.6 | 15.9 |
| 20 | Cohesion | c | kPa | 29.7 | 30.4 |

Layer 2: Medium gravel with coarse sand packed with a thickness of 3.30m;

| item no. | Name | Symbol | MU | Values | |
|---|---|---|---|---|---|
| | | | | | |
| 1 | Dust particle size fraction | Si | % | 5.63 | 7.56 |
| 2 | Sand particle size fraction | Sa | % | 22.67 | 32.36 |
| 3 | Gravel particle size fraction | Gr | % | 62.01 | 69.77 |
| 4 | Natural moisture | w | % | 4.23 | 4.94 |

In the content of the geotechnical project, depending on the designed geotechnical structure and the analyzed limit states, the appropriate characteristic and calculation values of the geotechnical parameters will be determined and used, in accordance with the NP 122:2010 standard, based on the results presented previously.

Following the analysis of the land on the site, it can be concluded that the foundation soil consists of:

- *Brownish sandy loam with medium plasticity, soft to hard plasticity, with medium compressibility is a good soil; it falls into the category of fine soils with IP>20%: sandy loams, silty loams and clays, having e< 1.1 and IC >0.75, under conditions of practically uniform and horizontal stratification;*
- *Medium gravel with dense coarse sand is a good soil; it falls into the category of Blocks, boulders and gravels, containing less than 40% sand and less than 30% clay, under conditions of practically uniform and horizontal stratification (having a slope of less than 10%).*

### 2.5.2 Seismic area classification

The studied buildings are located in an area corresponding to a ground acceleration of **ag = 0.30g** (IMR 225 years), with a seismic spectrum corner period **Tc = 1.60 sec**, in accordance with the maps3.1 and 3.2 of P100.

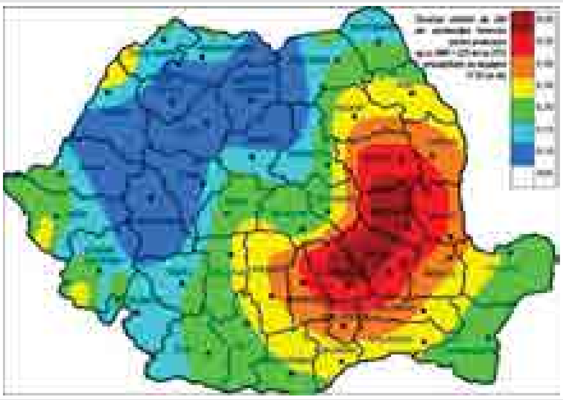


*Figure no. 1 - Zoning of the Romanian territory in terms of peak acceleration values for ag design for earthquakes with IMR = 225 years*


*Figure no. 2 - Zoning of the Romanian territory in terms of the control period Tc of the response spectrum*

### 2.5.3 Positioning in the wind action area

From the point of view of wind loads, according to CR 1-1-4/2012, the location corresponds to a reference wind pressure **$q_b$ = 0,50 kN/m2**.

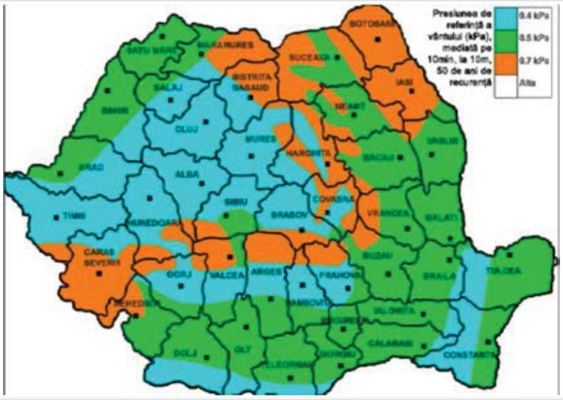



*Figure 3 - Zoning map of load values and characteristics and wind data (IMR = 50 years)*

### 2.5.4 Snow area classification

From the point of view of snow loads, according to CR 1-1-3/2012, the location corresponds to a characteristic value of snow loads on the ground **$S_{0k}$ = 2,00 kN/m2.**

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| **Loading from packed and unpacked snow - $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$** | |
|---|---|
| γ – importance-exposure factor for snow action acc.to. CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to. CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to. CR1-1-3/2012 | 1 |
| $C_t$ – thermal coefficient acc.to. CR1-1-3/2012 | 1 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to. CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$S_k$ = 1.60 kN/m2** |

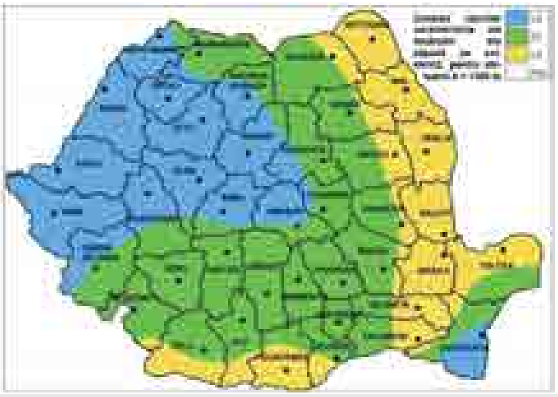


Figure no. 4 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

### 2.5.5 Frost depth

The maximum freezing depth in the site area is 80-90 cm from the ground surface, according to STAS 6054-77.

Figure no. 5 - Zoning map of maximum freezing depth

### 2.5.6 Non-destructive testing

Non-destructive tests were performed on the reinforced concrete and masonry elements that make up the load-bearing structures.

### 2.5.7 In situ and laboratory testing

#### 2.5.7.1 Surveys and measurements of structural elements

In order to reveal the characteristics of the structural elements, surveys and measurements were carried out on the elements of the resistance structure.

#### 2.5.7.2 Determination of the resistance of concrete elements (synthesis)

Detailed test results are presented in (*Error! Reference source not found.*)

Photo 01

Photo 02

| Photo 03 | Photo 04 |
|---|---|

Photo 05

Photo 06

Photo 07

Photo 08

Photo 09

Data processing and interpretation according to NP 137 - 2014

## Data processing and interpretation according to NP 137 - 2014

| Încercarea betonului prin metoda combinată | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Element | Pct. | Citiri sclerometru | | | | | | | | | | | Citiri ultrasunete | | | | Rezistența | |
| | | ΔN = Nmax – Nmin ≤ 5 | | | | | | | | | | | Viteza (m/s) | | | $V_{med}$ (m/s) | Valoarea de referință | Valoarea efectivă |
| | | Citiri valabile [citiri eliminate] | | | | | $N_{max}$ | $N_{min}$ | ΔN | $N_{med}$ | Corecția | $N_{med}$ | | | | | $f_{c,ref}$ [N/mm²] | $f_{c,ef} = 0,9 \cdot c_{,ref} \cdot C_t$ |
| Stalp ST02 | 1 | [47] | 49.0 | 50.0 | 51.0 | [55] | 51.0 | 49.0 | 2.0 | 50.0 | 0.0 | 50.0 | 3530 | 3540 | 3520 | 3530 | 35.3 | 31.8 |
| | 2 | [35] | 48.0 | 49.0 | 50.0 | [51] | 50.0 | 48.0 | 2.0 | 49.0 | 0.0 | 49.0 | 3510 | 3540 | 3540 | 3530 | 33.7 | 30.3 |
| | 3 | [45] | 47.0 | 49.0 | 51.0 | [52] | 51.0 | 47.0 | 4.0 | 49.0 | 0.0 | 49.0 | 3520 | 3530 | 3530 | 3527 | 33.7 | 30.3 |
| | | | | | | | | | | | | | | | | | Valoare mediata | **30.8** |

**Data processing and interpretation according to NP 137 - 2014**

| Încercarea betonului prin metoda combinată | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Element | Pct. | Citiri sclerometru – ΔN = Nmax – Nmin ≤ 5 – Citiri valabile (citiri eliminate) | | | | | $N_{min}$ | $N_{max}$ | ΔN | $N_{med}$ | Corecția | $N_{med}$ | Citiri ultrasunete – Viteza (m/s) | | | $V_{med}$ (m/s) | Rezistența – Valoarea de referință $f_{c,is}$ (N/mm²) | Valoarea efectivă $f_{ck}$=0.9 $f_{c,is}$ |
| Planseu (P01) | 1 | (38) | 40.0 | 41.0 | 42.0 | (45) | 42.0 | 40.0 | 2.0 | 41.0 | 0.0 | 41.0 | 2560 | 2530 | 2530 | 2540 | 18.5 | 16.7 |
| | 2 | (35) | 36.0 | 37.0 | 37.0 | (38) | 37.0 | 36.0 | 1.0 | 36.7 | 0.0 | 36.7 | 1820 | 1930 | 2050 | 1933 | 14.3 | 12.9 |
| | 3 | (34) | 35.0 | 35.0 | 36.0 | (45) | 36.0 | 35.0 | 1.0 | 35.3 | 0.0 | 35.3 | 2120 | 2150 | 2140 | 2137 | 12.9 | 11.6 |
| | | | | | | | | | | | | | | | | | Valoare medială | **13.7** |

**Data processing and interpretation according to NP 137 - 2014**

| Încercarea betonului prin metoda combinată | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Element | Pct. | Citiri sclerometru – ΔN = Nmax – Nmin ≤ 5 – Citiri valabile (citiri eliminate) | | | | | $N_{max}$ | $N_{min}$ | ΔN | $N_{med}$ | Corecția | $N_{med}$ | Citiri ultrasunete – Viteza (m/s) | | | $V_{med}$ (m/s) | Rezistența – Valoarea de referință $f_{c,is}$ (N/mm²) | Valoarea efectivă $f_{ck}$=0.9 $f_{c,is}$ |
| Grinda (GR01) | 1 | (38) | 39.0 | 40.0 | 41.0 | (43) | 41.0 | 39.0 | 2.0 | 40.0 | 0.0 | 40.0 | 3410 | 3400 | 3400 | 3403 | 17.3 | 15.6 |
| | 2 | (32) | 35.0 | 35.0 | 36.0 | (48) | 36.0 | 35.0 | 1.0 | 35.3 | 0.0 | 35.3 | 3440 | 3450 | 3470 | 3453 | 14.8 | 13.3 |
| | 3 | (35) | 37.0 | 37.0 | 38.0 | (39) | 38.0 | 37.0 | 1.0 | 37.3 | 0.0 | 37.3 | 3560 | 3420 | 3510 | 3497 | 16.3 | 14.7 |
| | | | | | | | | | | | | | | | | | Valoare medială | **14.5** |

### 2.5.7.3 Determination of the moisture level in walls (Building C7)

| Relative humidity–Walls (%) | | | | | |
|---|---|---|---|---|---|
| **Measurement point** | **Humidity value** | **Measurement point** | **Humidity value** | **Measurement point** | **Humidity value** |
| 1 | 70 | 8 | 59 | 15 | 26 |
| 2 | 72 | 9 | 16 | 16 | 35 |
| 3 | 83 | 10 | 56 | 17 | 62 |
| 4 | 54 | 11 | 52 | 18 | 21 |
| 5 | 79 | 12 | 50 | 19 | 66 |
| 6 | 32 | 13 | 100 | 20 | 14 |
| 7 | 27 | 14 | 74 | 21 | 18 |

Findings:

- *on exterior walls the humidity is much higher, especially where the brick is exposed at the facade level;*
- *The humidity at the base level is also higher compared to areas located at a higher altitude and where the sun's heat has access, so the effect of rainwater seepage, of rainwater stagnation near the building due to improper drainage, is diminished, the recorded values are approx. 50% U;*

## 2.6 Buildings description

### 2.6.1 Structural system – existing situation

**Building C7, C8**

The infrastructure of the building C7 and C8 is made of continuous stone foundations bound with hydraulic mortar and solid brick masonry, with a 10 cm flare from the structural wall which is 25 cm thick.

Building C7 and C8, have a ground floor height regime, with a rectangular shape, both buildings are attached to other buildings that are owned by the beneficiary of the study.

Following the surveys carried out, the resistance structure was determined, on which occasion it was noted that the building does not have reinforced concrete lintels, the masonry being simple without reinforced concrete confinement elements.

The floors above the ground floor are made of flexible wooden beam floors. The frames are made of wood

**Building C10, C11**

The building's resistance structure is made up of:

- the poles are made of 4 L40x5 mm profiles, stiffened between them - the beams are of the lattice beam type with lower and upper soles of 2 L40x5 mm profiles and diagonals of 8 mm bars

- the covering is made of corrugated sheet metal

- at the upper part of the lattice beams the structure is braced at the ends

**Building C13**

The foundations of building C13 are of the isolated surface type, and continuous beams under the masonry walls. The superstructure is of frame type (reinforced concrete pillars and beams), during the surveys carried out it was found that the floors are of 2 types (monolithic and prefabricated). The poles have a section of 30x35 cm and beams with a section of 30x30 cm.

The floor is 13-15 cm thick, over which the waterproofing layers of the non-traffic terrace are arranged.

**Building C15** functions as a pool and was built between 1911 and 1971

**Building C16**, is a metal gazebo, which discharges through a concrete platform. The main elements are made of 60 mm diameter pipe and the metal elements are made of 8 mm diameter bars.

**Building C17** and **building C20**, according to the land registry, have the function of a kitchen. During the surveys for building C17, reinforced concrete pillars were found, only reinforced concrete belts and beams.

The resistance structure is made of solid brick masonry, after the construction of the two buildings, at the time of the construction of the structure of building C18, the transition between the two sections was also made, with the pillars made of 120 mm circular pipe, the beams being made of U profiles and wooden rafters.

**Building C28** and **building C29**.

Buildings C28 and C29 have a masonry structure over which, in part, reinforced concrete belts were made, with a section of 15x25 cm, locally reinforced concrete ppolesillars were also made. The building has a rectangular shape in plan. Since the construction of the building bodies, interventions and modifications have been made that could not be dated accurately.

The frame is made of wood.

### 2.6.2 Damage, degradation, non-conformities

The photographic survey of the degradation is presented in Annex B – Photographic survey, the current state of the construction being revealed visually.

The degradations reported above were caused by:

**a) the repeated seismic actions**[1] suffered by the construction, which caused degradation in the form of cracks in the structural elements;

**b) the action of the weather** in the form of moisture infiltration, temperature variations and wind action caused damage to the roof, degradation of the plaster and paint layers and degradation of the mortar on the foundations;

**c) functional changes** (successive stages of expansion), led to changes in the loading regime, local and overall stiffness and resistance, causing the weakening of the resistance structure and the reduction of the quality of the structural response to current gravitational and seismic action;

**d) lack of maintenance of the construction**, led to its continuous degradation;

**e) masonry design error**, including the lack of reinforced concrete foundations, the lack of reinforced concrete confinement elements, the lack of slabs capable of fulfilling the role of a rigid washer in plan, the erection of gables, high gables in simple masonry with high sensitivity to overturning under horizontal actions, inadequate anchoring of the framework, non-conforming framework and with undersized elements, structural irregularity in plan, different wall areas in the two directions, large distances between walls, the existence of arches, vaults, domes, frameworks without elements that limit lateral thrusts;

**f) aging of the material over time;**

**g) uncontrolled interventions** in the form of breaking and opening for the installation of installations;

#### 2.6.2.1 Structural deterioration:

The photographic survey of the deterioration is presented in **Annex B**, the current state of the constructions being visually revealed. In principle, they refer to:

- cracks in the wooden floors;

**- most of the walls, of all solid brick masonry structures, show damage in the form of cracks, fissures-dislocations in the door and window openings;**

**- extensive portions of the wooden floor with multiple cracks in its plane, portions of plaster detached and fallen together with the existing layers are observed;**

- moisture infiltrations at the level of the walls and floors;

- damaged plaster on the outside;

[1] Strong Vrancea earthquakes (with intensity I≥VI) starting from the possible date of lifting the building (acc. The earthquake in Romania on March 4, 1977, coordinators acad. Stefan Balan, eng. Valeriu Cristea, dr. eng. Ilon Cornea, Publishing House of the Academy of the RSR (Socialist Republic of Romania), Bucharest 1982, page 78−79 and 84−85).

| Date (year/month/day) | Time (h:m:s) | Intensity | Magnitude |
|---|---|---|---|
| 1940/X/22 | 06 : 37 | VII | 6.1 |
| ~~1940/XI/10~~ | 01 : 39 | IX | 7.2 |
| 1945/IX/7 | 15 : 48 | VII − VIII | 6.4 |
| 1945/XII/09 | 06 : 08 | VII | 6.1 |
| 1948/V/29 | 04 : 49 | VI − VII | 5.8 |
| 1977/III/04 | 21 : 22 | IX | 7.2 |
| 1986/08/31 | 00 : 30 | VIII | 6.5 |
| 1990/05/30 | 13 : 30 | VI | 5.8 |
| 1990/05/31 | 04 : 00 | VI | 5.6 |
| 2004/10/27 | - | VII | 6.0 |
| 2014/11/22 | - | VI − VII | 5.7 |
| 2018/10/28 | - | VI − VII | 5.8 |

- existing chimneys and gables with sensitivity to overturning under horizontal actions; - friable masonry at the level of the land elevation;

### 2.6.3 Intervention

No documents were made available to the expert indicating any interventions on the existing construction.

### 2.6.4 Materials

In order to reveal the characteristics of the structural elements, surveys and measurements were carried out on the elements of the resistance structure.

### 2.6.5 Neighboring buildings

Building C13, C7, C8, C28 are attached to the ridge, on one side of other buildings that are not in the study analysis (they are not owned by the beneficiary) and are not subject to the contract.

### 2.6.6 Behavior during previous earthquakes and land settlements

No information is available regarding the behavior of the building during earthquakes, cracks or deformations were observed.

## 2.7 Neighboring buildings

In order to select the calculation method and the appropriate values of the confidence factors, according to P100-3/2019 (chap. 4.3.1), the following knowledge levels are defined:

- KL1: Limited knowledge; - KL2: Normal knowledge; - KL3: Complete knowledge.

### 2.7.1 Establishing the level of knowledge in which the construction falls

| item | Criterion | Level of accomplishment |
|---|---|---|
| **1.** | Inspection level | x **Limited inspection**<br>Extended inspection<br>Appropriate inspection |
| **2.** | Building geometry | Initial design<br>x **Complete building survey** |
| **3.** | Detailed composition of the building from the original technical design documentation | Yes<br>**x No** |
| **4.** | Mechanical properties of materials | Technical design documentation and reports Technical design documentation and limited tests Extended tests |

The level of knowledge achieved requires the permitted calculation method and the values of the confidence factors and is determined according to the following factors which, for the expertized building, are fulfilled as follows:

- geometry of the structure: is known from the geometric survey (in situ measurements)
- composition of structural and non-structural elements: the composition of the structure was revealed by direct viewing;

### 2.7.2 Establishing the CF confidence factors

The value of the confidence factor CF is used to determine the final values of the resistances of the materials used in the structure.

| Level of knowledge achieved | Building geometry | Detail composition | Mechanical properties of materials | Calculation | C.F. |
|---|---|---|---|---|---|
| **KL1** | **From the original overall design and visual inspection by field survey<br><br>or<br><br>from a complete building survey** | **Based on simulated design in accordance with practice at the time of construction and based on a limited field inspection** | **Values established based on standards valid at the time of construction and from limited field tests** | **LF-MRS** | **CF = 1,35** |
| KL2 | | From the original incomplete design and from an extensive limited field inspection | From the original design specifications and from limited field tests or from an extensive field material quality test | Any cf. method P100-1/2013 | CF = 1,20 |
| KL3 | | From the original complete design and from a limited field inspection or from a comprehensive field inspection | From original reports on the quality of materials in the work and from limited field tests or from a comprehensive test | Any cf. method P100-1/2013 | CF = 1,00 |

Under these conditions, the level of knowledge, from the point of view of the code P100-3/2019, point 4.3.1 the analyzed construction falls into **KL3 – limited knowledge** (acc. to point 2.7.2).

According to P100-3/2019, chap. 4.3.1(5), "The expert can complete the initial investigation of the construction after the structure has been uncovered, once the building's exploitation has been discontinued and intervention works have begun. Based on the new information obtained, the conclusions of the expertise and the intervention solution can be revised."

## 2.8 Evaluation methodology

### 2.8.1 Fundamental performance requirements and limit states

The fundamental requirements defined by the P100-1/2013 code in the case of designing structures for seismic actions are:

- life safety requirement (the structure will be designed to respond to the seismic action with IMR 225 years with a sufficient safety margin compared to the deformation level at which local or general collapse occurs so that people's lives are protected);
- degradation limitation requirement (the structure will be designed to respond to the seismic action with IMR 40 years without degradation or decommissioning whose costs are excessively high compared to the cost of the structure);

The fulfillment of the fundamental requirements is controlled by checking two categories of limit states:

- ultimate limit state, ULS, associated with the rupture of structural elements and other forms of failure that may endanger the safety of people's lives;
- the service limit state, SLS, which takes into account the development of degradations to a level beyond which the specific operating requirements are no longer met;

### 2.8.2 Seismic hazard

The seismic assessment of the building is made for the fundamental performance requirements, defined for new buildings in the code P 100-1/2013. The value considered for the IMR is 225 years (20% probability of exceedance in 50 years) for checks at the Ultimate Limit State and 40 years (20% probability of exceedance in 10 years) for checks at the Serviceability Limit State.

According to P100-3/2019, Annex A, in the case of designing intervention works for fundamental requirements, the seismic hazard defined as follows is taken into account:

- **When designing partial intervention works, aiming to be classified in risk class RsII, the ground acceleration values** are used for design with an average recurrence interval of 40 years, for verification at the Ultimate Limit State..
- **when designing intervention works with the aim of classifying them in the seismic risk class RsIII,** the ground acceleration values are used for design with an average recurrence interval of 100 years, for checks at the Ultimate Limit State, and 30 years, for checks at the Serviceability Limit State**.**.
- **when designing intervention works with the aim of classifying them in the seismic risk class RsIV,** the ground acceleration values are used for design with an average recurrence interval of 225 years, for checks at the Ultimate Limit State, and 40 years, for checks at the Serviceability Limit State**.**.

The peak values of the horizontal seismic acceleration corresponding to the average recurrence intervals mentioned above are determined based on the ag values established according to the zoning provided by P 100-1/2013, for the average recurrence interval of 225 years, by multiplication with the scaling factors in table A.1.

Table no. 1 - Scaling factors for determining the values of horizontal seismic accelerations according to Annex A, P100-3/2019

| County | $a_{g30}/a_{g225}$ | $a_{g40}/a_{g225}$ | $a_{g100}/a_{g225}$ | $a_{g225}/a_{g225}$ | $a_{g475}/a_{g225}$ |
|---|---|---|---|---|---|
| Arges, Bacau, Botosani, Braila, Bucuresti, Buzau, Calarasi, Constanta, Covasna, Dambovita, Dolj, Galati, Giurgiu, Gorj, Harghita, Ialomita, Iasi, Mehedinti, Neamt, Olt, Prahova, Suceava, Teleorman, Tulcea, Valcea, Vaslui, Vrancea | 0.40 | 0.45 | 0.80 | 1.00 | 1.25 |
| Alba, Arad, Bihor, Bistrita Nasaud, Brasov, Caras Severin, Cluj, Hunedoara, Maramures, Mures, Salaj, Satu Mare, Sibiu, Timis | 0.35 | 0.40 | 0.80 | 1.00 | 1.35 |

where: ag30 peak value of horizontal seismic acceleration with IMR = 30 years; ag40 peak value of horizontal seismic acceleration with IMR = 40 years; ag100 peak value of horizontal seismic acceleration with IMR = 100 years; ag225 peak value of horizontal seismic acceleration with IMR = 225 years; ag475 peak value of horizontal seismic acceleration with IMR = 475 years.

### 2.8.3 Compliance with the minimum density condition of structural masonry walls according to P100-1/2013

According to P100-1/2013, chapter 8.3.2, the minimum density of structural walls (p%) for buildings with structural walls made of simple masonry and reinforced masonry are the following:

*Table no. 1 -*

| **Number of floors above the embedment section and minimum density of structural walls (p%) for buildings with structural walls in ZNA** | | | | | | |
|---|---|---|---|---|---|---|
| N floors | Design Ground Acceleration ($a_g$) | | | | | |
| | 0.10g and 0.15g | | 0.20g and 0.25g | | 0.30g and 0.40g | |
| | Burnt clay deg. 1 and 2 | Burnt clay deg. 2S and AAC | Burnt clay deg. 1 and 2 | Burnt clay deg. 2S and AAC | Burnt clay deg. 1 and 2 | Burnt clay deg. 2S and AAC |
| 1 (P) | ≥4,0% | ≥4,5% | ≥5,0% | ≥5,5% | NA | NA |
| 2 (P+1) | ≥4,5% | ≥5,0% | ≥5,5% | ≥6,0% | | |
| 3 (P+2) | ≥5,0% | ≥5,5% | NA | NA | | |
| NA - the use of unreinforced masonry (ZNA) is not accepted | | | | | | |

**In the case of the buildings analyzed, structures with simple masonry structural walls are not accepted.**

### 2.8.4 Classification into the category of structures with dense or sparse walls according to CR6/2013

According to the CR6-2013 guideline – Design code for masonry structures, chapter 5.2, masonry resistance structures can be defined as structures with dense walls (honeycomb system) or structures with sparse walls (cellular system) according to the following criteria:

| **Masonry structure type acc.to. Chap. 5.2/CR6-2013** | | | |
|---|---|---|---|
| **Height level** | **Maximum distances between walls, in the two main directions** | **Cell area formed by the walls in the two main directions** | **Structure type** |
| **≤3.25m** | ≤5.00m | ≤25.00m2 | **Structure with dense walls** (honeycomb system) |
| **≤4.00m** | ≤9.00m | ≤75.00m2 | **Structure with sparse walls** (cellular system) |

### 2.8.5 Evaluation methodology

The seismic evaluation of existing individual building structures is carried out according to P100-3/2019 standard and consists of a set of operations that must establish the vulnerability and the introduction of the construction into a seismic risk class.

The P100/3-2019 code provides for three building evaluation methodologies, different in terms of complexity, defined by the conceptual basis, the level of refinement of the calculation methods and the level of detail of the verification operations:

- level 1 methodology (**low complexity**);
- level 2 methodology (**average complexity**);
- level 3 methodology (**high complexity**).

For the seismic assessment of a building, the choice of assessment methodologies is made depending on: the technical knowledge from the period of the design and execution of the building, the complexity of the building (especially from a structural point of view), the data available for preparing the assessment (level of knowledge), the function, importance and value of the building, the conditions regarding the seismic hazard of the site, the seismic acceleration values for the design and the local ground conditions, the type of structural system, the fundamental requirements established for the building, the purpose of the technical expertise and other conditions relevant to the building being assessed.

When preparing the expertise, the **level 2 methodology** was used, which can be applied to buildings with any type of structure, belonging to any class of importance-earthquake exposure. (according to P100-3/2019, chap. 2.3.2, art.1) and consists of:

- **qualitative assessment** of the building based on the criteria of conformity, composition and detailing of the constructions and the level of degradation and
- **quantitative evaluation** based on a linear static structural calculation and behavior factors. It is not necessary to apply the level 3 methodology.

## 2.9 Indicator R1 - Degree of fulfillment of seismic composition conditions

The qualitative evaluation of the structural conformation, the composition of the load-bearing elements and the construction rules for structures that take on the effect of seismic action is represented by the R1 indicator which is calculated in the following table, according to the criteria in the list specific to the type of construction according to. P100-3/2019.

| **R1 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.1) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | **III** | IV |
| **Values $R_1$** | | | |
| <30 | **30-60** | **60-90** | 90-100 |

### 2.9.1 Building C7

| **Fulfilling the conditions for masonry structures in the level 2 methodology. R1 indicator** | | | | |
|---|---|---|---|---|
| **Criterion** | **Fulfilled** | **Minor unfulfillment** | **Moderate unfulfillment** | **Major unfulfillment** |
| Scoring intervals | 10 | 8…9 | 4…7 | 0…3 |
| 1. **Structural system quality: spatial collaboration efficiency** | | | | |
| The nature and quality of the connections between the walls on the orthogonal directions | | | | 3 |
| Nature and quality of connections between walls and floors | | | | 3 |
| Approximately equal masonry areas in both directions | | | | 3 |
| Criterion average | **[3.00]** | | | |
| 2. **Masonry quality** | | | | |
| Elements quality | | | 5 | |
| Structure homogeneity | | | 5 | |
| Joints regularity | | | 5 | |
| Degree of mortar filling | | | 5 | |
| Existence of weakened areas | | | 5 | |
| Criterion average | **[5.00]** | | | |
| 3. **Type of floors** | | | | |
| Horizontal rigidity | | | | 3 |
| Efficiency of connections with walls | | | | 3 |
| Criterion average | **[3.00]** | | | |
| 4. **Plan configuration** | | | | |
| Compactness and geometric symmetry and structure in plan | | | 5 | |
| Criterion average | **[5.00]** | | | |
| 5. **Configuration in elevation** | | | | |
| Geometric and structural uniformity in elevation | | | 7 | |
| Criterion average | **[7.00]** | | | |
| 6. **Distance between walls** | | | | |
| Distances between structural walls, in each direction | | | 5 | |
| Criterion average | **[5.00]** | | | |
| 7. **Elements that give lateral thrust** | | | | |
| Existence of arches, vaults, domes, trusses with or without elements that limit the effects of thrusts | | | 5 | |
| Criterion average | **[5.00]** | | | |
| 8. **Type of foundation ground** | | | | |
| Nature of the ground foundation | | | 5 | |

| | | | | |
|---|---|---|---|---|
| The capacity of the foundations to take over and transmit to the ground vertical loads, efforts from differentiated settlements and | | | 5 | |
| Criterion average | [5.00] | | | |
| 9. **Possible interactions with adjacent buildings** | | | | |
| Existence of the risk of collision with adjacent buildings, the heights of neighboring buildings, the risk of falling of some components of neighboring buildings | | | 5 | |
| Criterion average | [5.00] | | | |
| 10. **Non-structural elements** | | | | |
| Existence of major masonry elements (sills, pediments, tympani), heavy cladding, important decorative elements | | | | 1 |
| Criterion average | [1.00] | | | |
| **Total** | [44.00] | | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C7 (1852): R1= 44.00**, placing the construction in class II of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. to chapter D.3.2.2 of P100/3-2019);

### 2.9.2 Building C8

| Fulfilling the conditions for masonry structures in the level 2 methodology. R1 indicator | | | | |
|---|---|---|---|---|
| **Criterion** | **Fulfilled** | **Minor unfulfillment** | **Moderate unfulfillment** | **Major unfulfillment** |
| Scoring intervals | 10 | 8…9 | 4…7 | 0…3 |
| 1. **Structural system quality: spatial collaboration efficiency** | | | | |
| The nature and quality of the connections between the walls on the orthogonal directions | | | | 3 |
| Nature and quality of connections between walls and floors | | | | 3 |
| Approximately equal masonry areas in both directions | | | | 3 |
| Criterion average | [3.00] | | | |
| 2. **Masonry quality** | | | | |
| Elements quality | | | 5 | |
| Structure homogeneity | | | 5 | |
| Joints regularity | | | 5 | |
| Degree of mortar filling | | | 5 | |
| Existence of weakened areas | | | 5 | |
| Criterion average | [5.00] | | | |
| 3. **Type of floors** | | | | |
| Horizontal rigidity | | | | 3 |
| Efficiency of connections with walls | | | | 3 |
| Criterion average | [3.00] | | | |
| 4. **Plan configuration** | | | | |
| Compactness and geometric symmetry and structure in plan | | | | 3 |
| Criterion average | [3.00] | | | |
| 5. **Configuration in elevation** | | | | |
| Geometric and structural uniformity in elevation | | | 7 | |
| Criterion average | [7.00] | | | |
| 6. **Distance between walls** | | | | |
| Distances between structural walls, in each direction | | | | 4 |
| Criterion average | [4.00] | | | |
| 7. **Elements that give lateral thrust** | | | | |
| Existence of arches, vaults, domes, trusses with or without elements that limit the effects of thrusts | | | 5 | |
| Criterion average | [5.00] | | | |
| 8. **Type of foundation ground** | | | | |
| Nature of the ground foundation | | | 5 | |
| The capacity of the foundations to take over and transmit to the ground vertical loads, efforts from differentiated settlements and | | | 5 | |
| Criterion average | [5.00] | | | |
| 9. **Possible interactions with adjacent buildings** | | | | |

| | | | | |
|---|---|---|---|---|
| Existence of the risk of collision with adjacent buildings, the heights of neighboring buildings, the risk of falling of some components of neighboring buildings | | | 5 | |
| Criterion average | [5.00] | | | |
| 10. **Non-structural elements** | | | | |
| Existence of major masonry elements (sills, pediments, tympani), heavy cladding, important decorative elements | | | | 1 |
| Criterion average | [1.00] | | | |
| **Total** | [41.00] | | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C8 (1971): R1= 41.00**, placing the construction in class II of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. to chapter D.3.2.2 of P100/3-2019);

### 2.9.3 Building C10-C11

| **Criterion** | **The criterion is met.** | **The criterion is not met.** | |
|---|---|---|---|
| | | **Moderate unfulfillment** | **Moderate unfulfillment** |
| **1. *Conditions regarding the structure configuration*** | **50** | **30 ÷ 49** | **0 ÷ 29** |
| - the load path is continuous | | 40 | |
| - the system is redundant (the system has enough links to have lateral stability and there are enough potential plastic areas) | | | 20 |
| - there are no weak levels in terms of resistance | | | 20 |
| -there are no flexible levels | | | 20 |
| - there are no significant changes in the plan dimensions of the structural system from level to level | 50 | | |
| - there are no vertical discontinuities (all vertical elements are continuous up to the foundation) | 50 | | |
| - there are no differences between the level masses greater than 50% | | 45 | |
| - overall torsional effects are moderate | | | 20 |
| - the infrastructure (foundations) is able to transmit vertical and horizontal forces to the | | 30 | |
| **Criterion average 1** | **[33]** | | |
| **2. *Conditions regarding structure interactions 10 5 ÷ 9 0 ÷ 4*** | | | |
| - distances to neighboring buildings exceed the minimum joint size, according to P100-2013. | 10 | | |
| - the intermediate floors (struts) have their own lateral structure or are adequately anchored to the main structure | 10 | | |
| - non-structural walls are isolated (or flexibly connected) to the structure | | 8 | |
| **Criterion average 2** | **It is chosen the score [9]** | | |
| **3. *Conditions regarding the composition of structural elements 30 20 ÷ 29 0 ÷ 19*** | | | |
| - Ranking of the capable efforts of the structural elements ensures the development of a favorable mechanism for dissipating seismic energy, the dissipative areas being located at the ends of the beams in the vicinity of the beam-pole joint. | | | 10 |
| **Beams**<br>- the potentially plastic areas (at the ends of the beams) have sections of section class 1 or 2. | | | 10 |
| - both soles are supported laterally against the loss of general stability in the potentially plastic areas, the value of the force to be taken up by the respective supports being according | | | 5 |
| - the beam-pole connection is of rigid type, of total capacity, being able to transmit to the pole the entire bending moment developed at the end of the beam | | | 10 |
| **Poles**<br>- the potentially plastic areas at the base of the pole and at the upper end of the pole located on the last floor have sections of section class 1 or 2; | | 20 | |
| - the web panels of the poles in the frame node area (beam-pole connection) can take up the shear force corresponding to the capable plastic moments of the dissipative areas of the adjacent beams; | | 20 | |
| - the thickness of the pole core in the frame node area (possibly supplemented with doubling plates) has a sufficiently low flexibility (according to P 100-1) so that loss of local stability is avoided; | | 20 | |
| - at the frame node, the pole is provided with continuity stiffeners at the level of the soles (upper and lower) of the adjacent beams, which ensure the continuity of the transmission of normal stresses from one beam to another; | | 20 | |

| | | | |
|---|---|---|---|
| - in the frame node area, the pole flanges are laterally connected to the upper flange of the adjacent beams; | | 20 | |
| the slenderness of the pole, in the plane where the beams can form plastic hinges, is limited to the value 0.7 lambda e | | 20 | |
| **Criterion average 3** | **It is chosen the score [15]** | | |
| ***4. Floor conditions 10 6 ÷ 9 0 ÷ 5*** | | | |
| -The floor slab with a thickness of > 100 mm is made of monolithic reinforced concrete or prefabricated precast concrete with an appropriate overcasting | | | 1 |
| - Seismic forces in the floor plane can be transmitted to the vertical structural elements (walls, frames) through shear and compression forces in concrete, and/or through connectors and collectors made of reinforcement with sufficient cross-section | | | 1 |
| **Criterion average 4** | **It is chosen the score [1]** | | |
| **TOTAL** | **[58]** | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C10 (1966-1971): R1= 58.00**, placing the construction in class II of seismic risk (from the point of view of fulfilling the structural conditions for metal buildings (acc. to chapter D.3.2.2 of P100/3-2019);

### 2.9.4 Building C12

| **Criterion** | **The criterion is met.** | **The criterion is not met.** | |
|---|---|---|---|
| | | **Moderate unfulfillment** | **Moderate unfulfillment** |
| **1. *Conditions regarding the structure configuration*** | **50** | **30 ÷ 49** | **0 ÷ 29** |
| - the load path is continuous | | 40 | |
| - the system is redundant (the system has enough links to have lateral stability and there are enough potential plastic areas) | | | 20 |
| - there are no weak levels in terms of resistance | | | 20 |
| -there are no flexible levels | | | 20 |
| - there are no significant changes in the plan dimensions of the structural system from level to level | 50 | | |
| - there are no vertical discontinuities (all vertical elements are continuous up to the foundation) | 50 | | |
| - there are no differences between the level masses greater than 50% | | 45 | |
| - overall torsional effects are moderate | | 20 | |
| - the infrastructure (foundations) is able to transmit vertical and horizontal forces to the | | | 10 |
| **Criterion average 1** | **[30]** | | |
| ***2. Conditions regarding structure interactions 10 5 ÷ 9 0 ÷ 4*** | | | |
| - distances to neighboring buildings exceed the minimum joint size, according to P100-2013. | 10 | | |
| - the intermediate floors (struts) have their own lateral structure or are adequately anchored to the main structure | 10 | | |
| - non-structural walls are isolated (or flexibly connected) to the structure | | 8 | |
| **Criterion average 2** | **It is chosen the score [9]** | | |
| ***3. Conditions regarding the composition of structural elements 30 20 ÷ 29 0 ÷ 19*** | | | |
| - Ranking of the capable efforts of the structural elements ensures the development of a favorable mechanism for dissipating seismic energy, the dissipative areas being located at the ends of the beams in the vicinity of the beam-pole joint. | | | 10 |
| **Beams**<br>- the potentially plastic areas (at the ends of the beams) have sections of section class 1 or 2. | | | 10 |
| - both soles are supported laterally against the loss of general stability in the potentially plastic areas, the value of the force to be taken up by the respective supports being according | | | 5 |
| - the beam-pole connection is of rigid type, of total capacity, being able to transmit to the pole the entire bending moment developed at the end of the beam | | | 10 |
| **Poles**<br>- the potentially plastic areas at the base of the pole and at the upper end of the pole located on the last floor have sections of section class 1 or 2; | | | 10 |
| - the web panels of the poles in the frame node area (beam-pole connection) can take up the shear force corresponding to the capable plastic moments of the dissipative areas of the adjacent beams; | | | 10 |

| | | | |
|---|---|---|---|
| - the thickness of the pole core in the frame node area (possibly supplemented with doubling plates) has a sufficiently low flexibility (according to P 100-1) so that loss of local stability is avoided; | | | 10 |
| - at the frame node, the pole is provided with continuity stiffeners at the level of the soles (upper and lower) of the adjacent beams, which ensure the continuity of the transmission of normal stresses from one beam to another; | | | 10 |
| - in the frame node area, the pole flanges are laterally connected to the upper flange of the adjacent beams; | | | 10 |
| the slenderness of the pole, in the plane where the beams can form plastic hinges, is limited to the value 0.7 lambda e | | | 10 |
| **Criterion average 3** | **It is chosen the score [9]** | | |
| ***4. Floor conditions 10 6 ÷ 9 0 ÷ 5*** | | | |
| -The floor slab with a thickness of > 100 mm is made of monolithic reinforced concrete or prefabricated precast concrete with an appropriate overcasting | | | 1 |
| - Seismic forces in the floor plane can be transmitted to the vertical structural elements (walls, frames) through shear and compression forces in concrete, and/or through connectors and collectors made of reinforcement with sufficient cross-section | | | 1 |
| **Criterion average 4** | **It is chosen the score [1]** | | |
| **TOTAL** | **[49]** | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C12 (after 1971): R1= 49.00**, placing the construction in class II of seismic risk (from the point of view of fulfilling the structural conditions for metal buildings (acc. to chapter D.3.2.2 of P100/3-2019);

### 2.9.5 Building C13

| **Criterion** | **The criterion is met.** | **The criterion is not met.** | |
|---|---|---|---|
| | | **Moderate unfulfillment** | **Moderate unfulfillment** |
| ***1. Conditions regarding the structure configuration*** | **50** | **30 ÷ 49** | **0 ÷ 29** |
| - the load path is continuous | | 45 | |
| - the system is redundant (the system has enough links to have lateral stability and there are enough potential plastic areas) | | 30 | |
| - there are no weak levels in terms of resistance | | 40 | |
| -there are no flexible levels | | 40 | |
| - there are no significant changes in the plan dimensions of the structural system from level to level | 50 | | |
| - there are no vertical discontinuities (all vertical elements are continuous up to the foundation) | 50 | | |
| - there are no differences between the level masses greater than 50% | | 45 | |
| - overall torsional effects are moderate | | | 20 |
| - the infrastructure (foundations) is able to transmit vertical and horizontal forces to the | | 30 | |
| **Criterion average 1** | **[39]** | | |
| ***2. Conditions regarding structure interactions*** | **10** | **5 ÷ 9** | **0 ÷ 4** |
| - distances to neighboring buildings exceed the minimum joint size, according to P100-2013. | | 5 | |
| - the intermediate floors (struts) have their own lateral structure or are adequately anchored to the main structure | 10 | | |
| - non-structural walls are isolated (or flexibly connected) to the structure | | 8 | |
| - there are no short captive poles | | 8 | |
| **Criterion average 2** | **It is chosen the score [8]** | | |
| ***3. Conditions regarding the composition (reinforcement) of structural elements*** | **30** | **20 ÷ 29** | **0 ÷ 19** |
| *3.1 - reinforced concrete frame structures* | | | |
| - the ranking of the resistances of the structural elements ensures the development of a favorable mechanism for the dissipation of seismic energy: at each node the sum of the capable moments of the poles is greater than the sum of the capable moments of the beams | | | 15 |
| - the normalized axial load (axial compressive force related to the cross-sectional area and the design compressive strength of the concrete) of the poles is moderate: indicative: $\nu_d \leq 0{,}65$ | | 25 | |
| - there are no short poles in the structure: the ratio between the section height and the free height of the pole is < 0.30 | | | 15 |

| | | | |
|---|---|---|---|
| - the shear strength of the node is sufficient to mobilize the bending resistance at the ends of the beams and poles | | 20 | |
| - the reinforcement of the poles is developed over 40 diameters, with stirrups at a distance of 10 diameters on the reinforcement area | | 20 | |
| - the reinforcement in the beams is inserted outside the critical areas | | 20 | |
| - the stirrups in the posts are arranged so that each vertical bar is in the corner of a stirrup (clips) | | | 15 |
| - the distances between stirrups, in the critical areas of the poles, do not exceed 10 diameters, and in the rest of the poles, 1/4 of the side | | 20 | |
| - the distances between stirrups in the plastic zones of the beams do not exceed 12 diameters and ½ of the beam width | | 20 | |
| - the transverse reinforcement of the nodes is at least that required in the critical areas of the | | 20 | |
| - the resistance of beams to positive moments on supports is at least 30% of the resistance to negative moments in the same section | | | 15 |
| - at least 2 continuous bars are provided at the top of the beams (uninterrupted in the opening) | | 20 | |
| **Criterion average 3** | **It is chosen the score [19]** | | |
| ***4. Conditions relating to floors*** | **10** | **6 ÷ 9** | **0 ÷ 5** |
| -The floor slab with a thickness of > 100 mm is made of monolithic reinforced concrete or prefabricated precast concrete with an appropriate overcasting | | 8 | |
| - The belt reinforcements and the distributed reinforcements in the slab provide the necessary bending resistance and shear force for the seismic forces applied in the | | | 4 |
| - Seismic forces in the floor plane can be transmitted to the vertical structural elements (walls, frames) through shear and compression forces in concrete, and/or through connectors and collectors made of reinforcement with sufficient cross-section | | 8 | |
| - Floor gaps are lined with sufficient reinforcement, adequately anchored | | | 4 |
| **Criterion average 4** | **It is chosen the score [6]** | | |
| **Total** | **[72]** | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C13 (1968-1971): R1= 72.00**, placing the construction in class III of seismic risk (from the point of view of fulfilling the structural conditions for concrete buildings (acc. To chapter. D.3.2.2 of P100/3-2019);

### 2.9.6 Building C15 (methodology 1)

- **Building C15: R1= 41,00**, placing the construction in class II of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. to chap. D.3.2.2 of P100/3-2019);

### 2.9.7 Building C16

| Criterion | The criterion is met. | The criterion is not met. | |
|---|---|---|---|
| | | Moderate unfulfillment | Moderate unfulfillment |
| ***1. Conditions regarding the structure configuration*** | **50** | **30 ÷ 49** | **0 ÷ 29** |
| - the load path is continuous | | 40 | |
| - the system is redundant (the system has enough links to have lateral stability and there are enough potential plastic areas) | | 40 | |
| - there are no weak levels in terms of resistance | | 40 | |
| -there are no flexible levels | | 40 | |
| - there are no significant changes in the plan dimensions of the structural system from level to level | 50 | | |
| - there are no vertical discontinuities (all vertical elements are continuous up to the foundation) | 50 | | |
| - there are no differences between the level masses greater than 50% | | 45 | |
| - overall torsional effects are moderate | | 40 | |
| - the infrastructure (foundations) is able to transmit vertical and horizontal forces to the | | | 20 |
| **Criterion average 1** | **[41]** | | |
| ***2. Conditions regarding structure interactions*** | **10** | **5 ÷ 9** | **0 ÷ 4** |

| | | | |
|---|---|---|---|
| - distances to neighboring buildings exceed the minimum joint size, according to P100-2013. | 10 | | |
| - the intermediate floors (struts) have their own lateral structure or are adequately anchored to the main structure | 10 | | |
| - non-structural walls are isolated (or flexibly connected) to the structure | 10 | | |
| **Criterion average 2** | **It is chosen the score [10]** | | |
| ***3. Conditions regarding the composition of structural elements*** | **30** | **20 ÷ 29** | **0 ÷ 19** |
| - Ranking of the capable efforts of the structural elements ensures the development of a favorable mechanism for dissipating seismic energy, the dissipative areas being located at the ends of the beams in the vicinity of the beam-pole joint. | | | 10 |
| **Beams**<br>- the potentially plastic areas (at the ends of the beams) have sections of section class 1 or 2. | | | 10 |
| - both soles are supported laterally against the loss of general stability in the potentially plastic areas, the value of the force to be taken up by the respective supports being according | | | 5 |
| - the beam-pole connection is of rigid type, of total capacity, being able to transmit to the pole the entire bending moment developed at the end of the beam | | | 10 |
| **Poles**<br>- the potentially plastic areas at the base of the pole and at the upper end of the pole located on the last floor have sections of section class 1 or 2; | | 20 | |
| - the web panels of the poles in the frame node area (beam-pole connection) can take up the shear force corresponding to the capable plastic moments of the dissipative areas of the adjacent beams; | | 20 | |
| - the thickness of the pole core in the frame node area (possibly supplemented with doubling plates) has a sufficiently low flexibility (according to P 100-1) so that loss of local stability is avoided; | | 20 | |
| - at the frame node, the pole is provided with continuity stiffeners at the level of the soles (upper and lower) of the adjacent beams, which ensure the continuity of the transmission of normal stresses from one beam to another; | | 20 | |
| - in the frame node area, the pole flanges are laterally connected to the upper flange of the adjacent beams; | | 20 | |
| the slenderness of the pole, in the plane where the beams can form plastic hinges, is limited to the value 0.7 lambda e | | 20 | |
| **Criterion average 3** | **It is chosen the score [15]** | | |
| ***4. Conditions relating to floors*** | **10** | **6 ÷ 9** | **0 ÷ 5** |
| -The floor slab with a thickness of > 100 mm is made of monolithic reinforced concrete or prefabricated precast concrete with an appropriate overcasting | | | 1 |
| - Seismic forces in the floor plane can be transmitted to the vertical structural elements (walls, frames) through shear and compression forces in concrete, and/or through connectors and collectors made of reinforcement with sufficient cross-section | | | 1 |
| **Criterion average 4** | **It is chosen the score [1]** | | |
| **Total** | **[67]** | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C16: R1= 67.00**, placing the construction in class III of seismic risk (from the point of view of fulfilling the structural conditions for metal buildings (acc. to. chap. D.3.2.2 of P100/3-2019);

### 2.9.8 Building C17

| **Fulfilling the conditions for masonry structures in the level 2 methodology. R1 indicator** | | | | |
|---|---|---|---|---|
| **Criterion** | **Fulfilled** | **Minor unfulfillment** | **Moderate unfulfillment** | **Major unfulfillment** |
| Scoring intervals | 10 | 8…9 | 4…7 | 0…3 |
| 1. **Structural system quality: spatial collaboration efficiency** | | | | |
| The nature and quality of the connections between the walls on the orthogonal directions | | | 4 | |
| Nature and quality of connections between walls and floors | | | 4 | |
| Approximately equal masonry areas in both directions | | | 4 | |
| Criterion average | **[4.00]** | | | |
| 2. **Masonry quality** | | | | |
| Elements quality | | | 5 | |
| Structure homogeneity | | | 5 | |
| Joints regularity | | | 5 | |

| | | | | |
|---|---|---|---|---|
| Degree of mortar filling | | | 5 | |
| Existence of weakened areas | | | 5 | |
| Criterion average | [5.00] | | | |
| 3. **Type of floors** | | | | |
| Horizontal rigidity | | | | 3 |
| Efficiency of connections with walls | | | | 3 |
| Criterion average | [3.00] | | | |
| 4. **Plan configuration** | | | | |
| Compactness and geometric symmetry and structure in plan | | | | 3 |
| Criterion average | [3.00] | | | |
| 5. **Configuration in elevation** | | | | |
| Geometric and structural uniformity in elevation | | | 7 | |
| Criterion average | [7.00] | | | |
| 6. **Distance between walls** | | | | |
| Distances between structural walls, in each direction | | | | 4 |
| Criterion average | [4.00] | | | |
| 7. **Elements that give lateral thrust** | | | | |
| Existence of arches, vaults, domes, trusses with or without elements that limit the effects of thrusts | | | 5 | |
| Criterion average | [5.00] | | | |
| 8. **Type of foundation ground** | | | | |
| Nature of the ground foundation | | | 5 | |
| The capacity of the foundations to take over and transmit to the ground vertical loads, efforts from differentiated settlements and | | | 5 | |
| Criterion average | [5.00] | | | |
| 9. **Possible interactions with adjacent buildings** | | | | |
| Existence of the risk of collision with adjacent buildings, the heights of neighboring buildings, the risk of falling of some components of neighboring buildings | | | 5 | |
| Criterion average | [5.00] | | | |
| 10. **Non-structural elements** | | | | |
| Existence of major masonry elements (sills, pediments, tympani), heavy cladding, important decorative elements | | | 5 | |
| Criterion average | [5.00] | | | |
| **Total** | **[46.00]** | | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C17 (1911-1964): R1= 46.00**, placing the construction in class II of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. To chap. D.3.2.2 of P100/3-2019);

### 2.9.9 Building C20

| Fulfilling the conditions for masonry structures in the level 2 methodology. R1 indicator | | | | |
|---|---|---|---|---|
| **Criterion** | **Fulfilled** | **Minor unfulfillment** | **Moderate unfulfillment** | **Major unfulfillment** |
| Scoring intervals | 10 | 8…9 | 4…7 | 0…3 |
| 1. **Structural system quality: spatial collaboration efficiency** | | | | |
| The nature and quality of the connections between the walls on the orthogonal directions | | | 4 | |
| Nature and quality of connections between walls and floors | | | 4 | |
| Approximately equal masonry areas in both directions | | | 4 | |
| Criterion average | [4.00] | | | |
| 2. **Masonry quality** | | | | |
| Elements quality | | | 5 | |
| Structure homogeneity | | | 5 | |
| Joints regularity | | | 5 | |
| Degree of mortar filling | | | 5 | |
| Existence of weakened areas | | | 5 | |
| Criterion average | [5.00] | | | |
| 3. **Type of floors** | | | | |
| Horizontal rigidity | | | | 3 |

| | | | | |
|---|---|---|---|---|
| Efficiency of connections with walls | | | 5 | |
| Criterion average | [4.00] | | | |
| 4. **Plan configuration** | | | | |
| Compactness and geometric symmetry and structure in plan | | | 5 | |
| Criterion average | [5.00] | | | |
| 5. **Configuration in elevation** | | | | |
| Geometric and structural uniformity in elevation | | | 7 | |
| Criterion average | [7.00] | | | |
| 6. **Distance between walls** | | | | |
| Distances between structural walls, in each direction | | | 5 | |
| Criterion average | [5.00] | | | |
| 7. **Elements that give lateral thrust** | | | | |
| Existence of arches, vaults, domes, trusses with or without elements that limit the effects of thrusts | | | 5 | |
| Criterion average | [5.00] | | | |
| 8. **Type of foundation ground** | | | | |
| Nature of the ground foundation | | | 5 | |
| The capacity of the foundations to take over and transmit to the ground vertical loads, efforts from differentiated settlements and | | | 5 | |
| Criterion average | [5.00] | | | |
| 9. **Possible interactions with adjacent buildings** | | | | |
| Existence of the risk of collision with adjacent buildings, the heights of neighboring buildings, the risk of falling of some components of neighboring buildings | | | 5 | |
| Criterion average | [5.00] | | | |
| 10. **Non-structural elements** | | | | |
| Existence of major masonry elements (sills, pediments, tympani), heavy cladding, important decorative elements | | | 5 | |
| Criterion average | [5.00] | | | |
| **Total** | [50.00] | | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C20 (>1974): R1= 50.00**, placing the construction in class II of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. to chapter D.3.2.2 of P100/3-2019);

### 2.9.10 Building C24; Building C27

| **Fulfilling the conditions for masonry structures in the level 2 methodology. R1 indicator** | | | | |
|---|---|---|---|---|
| **Criterion** | **Fulfilled** | **Minor unfulfillment** | **Moderate unfulfillment** | **Major unfulfillment** |
| Scoring intervals | 10 | 8…9 | 4…7 | 0…3 |
| 1. **Structural system quality: spatial collaboration efficiency** | | | | |
| The nature and quality of the connections between the walls on the orthogonal directions | | | 7 | |
| Nature and quality of connections between walls and floors | | | 7 | |
| Approximately equal masonry areas in both directions | | | 7 | |
| Criterion average | [7.00] | | | |
| 2. **Masonry quality** | | | | |
| Elements quality | | | 6 | |
| Structure homogeneity | | | 6 | |
| Joints regularity | | | 6 | |
| Degree of mortar filling | | | 6 | |
| Existence of weakened areas | | | 6 | |
| Criterion average | [6.00] | | | |
| 3. **Type of floors** | | | | |
| Horizontal rigidity | | | 6 | |
| Efficiency of connections with walls | | | 6 | |
| Criterion average | [6.00] | | | |
| 4. **Plan configuration** | | | | |
| Compactness and geometric symmetry and structure in plan | | | 7 | |
| Criterion average | [6.00] | | | |
| 5. **Configuration in elevation** | | | | |
| Geometric and structural uniformity in elevation | | | 7 | |

| | | | | |
|---|---|---|---|---|
| Criterion average | [7.00] | | | |
| 6. **Distance between walls** | | | | |
| Distances between structural walls, in each direction | | | 4 | |
| Criterion average | [4.00] | | | |
| 7. **Elements that give lateral thrust** | | | | |
| Existence of arches, vaults, domes, trusses with or without elements that limit the effects of thrusts | 10 | | | |
| Criterion average | [10.00] | | | |
| 8. **Type of foundation ground** | | | | |
| Nature of the ground foundation | | | 5 | |
| The capacity of the foundations to take over and transmit to the ground vertical loads, efforts from differentiated settlements and | | | 5 | |
| Criterion average | [5.00] | | | |
| 9. **Possible interactions with adjacent buildings** | | | | |
| Existence of the risk of collision with adjacent buildings, the heights of neighboring buildings, the risk of falling of some components of neighboring buildings | | | 5 | |
| Criterion average | [5.00] | | | |
| 10. **Non-structural elements** | | | | |
| Existence of major masonry elements (sills, pediments, tympani), heavy cladding, important decorative elements | 10 | | | |
| Criterion average | [10.00] | | | |
| **Total** | [66.00] | | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C24 (1911-1964): R1= 66.00**, placing the construction in class III of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. To chap. D.3.2.2 of P100/3-2019);

### 2.9.11 Building C25

| Criterion | The criterion is met. | The criterion is not met. | |
|---|---|---|---|
| | | Moderate unfulfillment | Moderate unfulfillment |
| ***1. Conditions regarding the structure configuration*** | **50** | **30 ÷ 49** | **0 ÷ 29** |
| - the load path is continuous | | 40 | |
| - the system is redundant (the system has enough links to have lateral stability and there are enough potential plastic areas) | | 40 | |
| - there are no weak levels in terms of resistance | | 40 | |
| -there are no flexible levels | | 40 | |
| - there are no significant changes in the plan dimensions of the structural system from level to level | 50 | | |
| - there are no vertical discontinuities (all vertical elements are continuous up to the foundation) | 50 | | |
| - there are no differences between the level masses greater than 50% | | 45 | |
| - overall torsional effects are moderate | | 30 | |
| - the infrastructure (foundations) is able to transmit vertical and horizontal forces to the | | | 20 |
| **Criterion average 1** | [39] | | |
| ***2. Conditions regarding structure interactions*** | **10** | **5 ÷ 9** | **0 ÷ 4** |
| - distances to neighboring buildings exceed the minimum joint size, according to P100-2013. | 10 | | |
| - the intermediate floors (struts) have their own lateral structure or are adequately anchored to the main structure | 10 | | |
| - non-structural walls are isolated (or flexibly connected) to the structure | 10 | | |
| **Criterion average 2** | **It is chosen the score [10]** | | |
| ***3. Conditions regarding the composition of structural elements*** | **30** | **20 ÷ 29** | **0 ÷ 19** |
| - Ranking of the capable efforts of the structural elements ensures the development of a favorable mechanism for dissipating seismic energy, the dissipative areas being located at the ends of the beams in the vicinity of the beam-pole joint. | | | 10 |
| **Beams**<br>- the potentially plastic areas (at the ends of the beams) have sections of section class 1 or 2. | | | 10 |
| - both soles are supported laterally against the loss of general stability in the potentially plastic areas, the value of the force to be taken up by the respective supports being according | | | 5 |

| | | | |
|---|---|---|---|
| - the beam-pole connection is of rigid type, of total capacity, being able to transmit to the pole the entire bending moment developed at the end of the beam | | | 10 |
| **Poles**<br>- the potentially plastic areas at the base of the pole and at the upper end of the pole located on the last floor have sections of section class 1 or 2; | | 15 | |
| - the web panels of the poles in the frame node area (beam-pole connection) can take up the shear force corresponding to the capable plastic moments of the dissipative areas of the adjacent beams; | | 15 | |
| - the thickness of the pole core in the frame node area (possibly supplemented with doubling plates) has a sufficiently low flexibility (according to P 100-1) so that loss of local stability is avoided; | | 15 | |
| - at the frame node, the pole is provided with continuity stiffeners at the level of the soles (upper and lower) of the adjacent beams, which ensure the continuity of the transmission of normal stresses from one beam to another; | | 15 | |
| - in the frame node area, the pole flanges are laterally connected to the upper flange of the adjacent beams; | | 15 | |
| the slenderness of the pole, in the plane where the beams can form plastic hinges, is limited to the value 0.7 lambda e | | 15 | |
| **Criterion average 3** | **It is chosen the score [13]** | | |
| ***4. Conditions relating to floors*** | **10** | **6 ÷ 9** | **0 ÷ 5** |
| -The floor slab with a thickness of > 100 mm is made of monolithic reinforced concrete or prefabricated precast concrete with an appropriate overcasting | | | 1 |
| - Seismic forces in the floor plane can be transmitted to the vertical structural elements (walls, frames) through shear and compression forces in concrete, and/or through connectors and collectors made of reinforcement with sufficient cross-section | | | 1 |
| **Criterion average 4** | **It is chosen the score [1]** | | |
| **Total** | **[63]** | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C25 (before 1895): R1= 63.00**, placing the construction in class III of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. to chap. D.3.2.2 of P100/3-2019);

### 2.9.12 Building C26

| **Fulfilling the conditions for masonry structures in the level 2 methodology. R1 indicator** | | | | |
|---|---|---|---|---|
| **Criterion** | **Fulfilled** | **Minor unfulfillment** | **Moderate unfulfillment** | **Major unfulfillment** |
| Scoring intervals | 10 | 8…9 | 4…7 | 0…3 |
| 1. **Structural system quality: spatial collaboration efficiency** | | | | |
| The nature and quality of the connections between the walls on the orthogonal directions | | | 7 | |
| Nature and quality of connections between walls and floors | | | 7 | |
| Approximately equal masonry areas in both directions | | | 7 | |
| Criterion average | **[7.00]** | | | |
| 2. **Masonry quality** | | | | |
| Elements quality | | | 6 | |
| Structure homogeneity | | | 6 | |
| Joints regularity | | | 6 | |
| Degree of mortar filling | | | 6 | |
| Existence of weakened areas | | | 6 | |
| Criterion average | **[6.00]** | | | |
| 3. **Type of floors** | | | | |
| Horizontal rigidity | | | 6 | |
| Efficiency of connections with walls | | | 6 | |
| Criterion average | **[6.00]** | | | |
| 4. **Plan configuration** | | | | |
| Compactness and geometric symmetry and structure in plan | | | 7 | |
| Criterion average | **[6.00]** | | | |
| 5. **Configuration in elevation** | | | | |
| Geometric and structural uniformity in elevation | | | 5 | |
| Criterion average | **[6.00]** | | | |
| 6. **Distance between walls** | | | | |

| Criterion | Fulfilled | Minor unfulfillment | Moderate unfulfillment | Major unfulfillment |
|---|---|---|---|---|
| Distances between structural walls, in each direction | | | 4 | |
| Criterion average | | | [4.00] | |
| 7. **Elements that give lateral thrust** | | | | |
| Existence of arches, vaults, domes, trusses with or without elements that limit the effects of thrusts | 10 | | | |
| Criterion average | | | [10.00] | |
| 8. **Type of foundation ground** | | | | |
| Nature of the ground foundation | | | 5 | |
| The capacity of the foundations to take over and transmit to the ground vertical loads, efforts from differentiated settlements and | | | 5 | |
| Criterion average | | | [5.00] | |
| 9. **Possible interactions with adjacent buildings** | | | | |
| Existence of the risk of collision with adjacent buildings, the heights of neighboring buildings, the risk of falling of some components of neighboring buildings | | | 5 | |
| Criterion average | | | [5.00] | |
| 10. **Non-structural elements** | | | | |
| Existence of major masonry elements (sills, pediments, tympani), heavy cladding, important decorative elements | 10 | | | |
| Criterion average | | | [10.00] | |
| **Total** | | | [65.00] | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

- **Building C26 (1911-1964): R1= 65.00**, placing the construction in class III of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. To chap. D.3.2.2 of P100/3-2019);

### 2.9.13 Building C28-C29

| Fulfilling the conditions for masonry structures in the level 2 methodology. R1 indicator | | | | |
|---|---|---|---|---|
| **Criterion** | **Fulfilled** | **Minor unfulfillment** | **Moderate unfulfillment** | **Major unfulfillment** |
| Scoring intervals | 10 | 8…9 | 4…7 | 0…3 |
| 1. **Structural system quality: spatial collaboration efficiency** | | | | |
| The nature and quality of the connections between the walls on the orthogonal directions | | | 8 | |
| Nature and quality of connections between walls and floors | | | 8 | |
| Approximately equal masonry areas in both directions | | | 8 | |
| Criterion average | | | [8.00] | |
| 2. **Masonry quality** | | | | |
| Elements quality | | | 6 | |
| Structure homogeneity | | | 6 | |
| Joints regularity | | | 6 | |
| Degree of mortar filling | | | 6 | |
| Existence of weakened areas | | | 6 | |
| Criterion average | | | [6.00] | |
| 3. **Type of floors** | | | | |
| Horizontal rigidity | | | 6 | |
| Efficiency of connections with walls | | | 6 | |
| Criterion average | | | [6.00] | |
| 4. **Plan configuration** | | | | |
| Compactness and geometric symmetry and structure in plan | | | 7 | |
| Criterion average | | | [6.00] | |
| 5. **Configuration in elevation** | | | | |
| Geometric and structural uniformity in elevation | | | 7 | |
| Criterion average | | | [7.00] | |
| 6. **Distance between walls** | | | | |
| Distances between structural walls, in each direction | | | 4 | |
| Criterion average | | | [4.00] | |
| 7. **Elements that give lateral thrust** | | | | |
| Existence of arches, vaults, domes, trusses with or without elements that limit the effects of thrusts | | | 7 | |

| | | | | |
|---|---|---|---|---|
| Criterion average | [7.00] | | | |
| 8. **Type of foundation ground** | | | | |
| Nature of the ground foundation | | | 5 | |
| The capacity of the foundations to take over and transmit to the ground vertical loads, efforts from differentiated settlements and | | | 5 | |
| Criterion average | [5.00] | | | |
| 9. **Possible interactions with adjacent buildings** | | | | |
| Existence of the risk of collision with adjacent buildings, the heights of neighboring buildings, the risk of falling of some components of neighboring buildings | | | 5 | |
| Criterion average | [5.00] | | | |
| 10. **Non-structural elements** | | | | |
| Existence of major masonry elements (sills, pediments, tympani), heavy cladding, important decorative elements | | | 7 | |
| Criterion average | [7.00] | | | |
| **Total** | [60.00] | | | |

The indicator representing the qualitative assessment of the seismic compliance of the structure resulted equal to the following values:

**- Building C28-C29 (1911-1964; 1964-1971): R1= 62.00**, placing the construction in class III of seismic risk (from the point of view of fulfilling the structural conditions for masonry buildings (acc. to chap. D.3.2.2 of P100/3-2019);

## 2.10 Indicator R2 - Degree of structural damage

The state of degradation of the structure is characterized by the degree of structural damage R2 which is calculated based on the score assigned to the different categories of degradation, presented in the type-specific list in the Annex corresponding to the structural material used acc. to P100-3/2019.

### 2.10.1 Building C7

| **State of degradation of vertical structural elements** | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| **Scoring intervals** | **70** | **65** | **50** | **25** |
| **(i)** Vertical cracks in parapets, lintels and arches | | | | 25 |
| **(ii)** Inclined cracks in parapets, lintels and arches | | | | 25 |
| **(iii)** Inclined cracks in slits | | | | 25 |
| **(iv)** Crushing of masonry caused by local concentration of compressive stresses, possibly with expulsion of material | | 65 | | |
| **(v)** Horizontal cracks at the ends of slits | | 65 | | |
| **(vi)** Damage at the intersections of walls with a tendency to detach | | 65 | | |
| **(vii)** Vertical cracks/cracks at the connections between perpendicular walls | | | 50 | |
| **(viii)** Local expulsion of masonry from the horizontal elements on which the floors rest | | 65 | | |
| **Total** | [48.12] | | | |

| **State of degradation of horizontal structural elements** | | | | |
|---|---|---|---|---|
| Criterion | Insignificant | Moderate | Serious | Very serious |
| Scoring intervals | 30 | 25 | 20 | 15 |
| Sheets | | | 20 | |
| **Total** | [20.00] | | | |

| **State of degradation of the structure. R2 indicator** | | | |
|---|---|---|---|
| Damage category | Vertical elements ($A_v$) | Horizontal elements ($A_h$) | Total |

| | Affected surface | | | Affected surface | | | |
|---|---|---|---|---|---|---|---|
| | ≤1/3 | 1/3…2/3 | >2/3 | ≤1/3 | 1/3…2/3 | >2/3 | $R_2$ |
| Insignificant | 70 | 70 | 70 | 30 | 30 | 30 | |
| Moderate | 65 | 60 | 50 **(48,12)** | 25 | **20** | 15 **(20.0)** | **68.12** |
| Serious | 50 | 45 | 35 | 20 | 15 | 10 | |
| Very serious | 30 | 25 | 15 | 15 | 10 | 5 | |

The R2 indicator, which represents the level of structural degradation of the building, resulted equal to the following values:

**- BUILDING C7 (1852): R2=68.12 points, which places the structure in class II of seismic risk, from the point of view of the state of degradation of the structural elements in the masonry (acc. to chap.** D.3.2.2. of P100/3-2019);

| **R2 values associated with seismic risk classes**<br>(acc. to P100-3/2019, chap. 8.1.2) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_2$ | | | |
| <50 | **50-70** | 70-90 | 90-100 |

### 2.10.2 Building C8

| **State of degradation of vertical structural elements** | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| **Scoring intervals** | **70** | **65** | **50** | **25** |
| **(i)** Vertical cracks in parapets, lintels and arches | | | | 25 |
| **(ii)** Inclined cracks in parapets, lintels and arches | | | | 25 |
| **(iii)** Inclined cracks in slits | | | | 25 |
| **(iv)** Crushing of masonry caused by local concentration of compressive stresses, possibly with expulsion of material | | | 50 | |
| **(v)** Horizontal cracks at the ends of slits | | | 50 | |
| **(vi)** Damage at the intersections of walls with a tendency to detach | | | 50 | |
| **(vii)** Vertical cracks/cracks at the connections between perpendicular walls | | | 50 | |
| **(viii)** Local expulsion of masonry from the horizontal elements on which the floors rest | | 65 | | |
| **Total** | **[42.50]** | | | |

| **State of degradation of horizontal structural elements** | | | | |
|---|---|---|---|---|
| Criterion | **Insignific ant** | **Moderate** | **Serious** | **Very serio** |
| Scoring intervals | 30 | 25 | 20 | 15 |
| Sheets | | | | 15 |
| **Total** | **[15.00]** | | | |

| **State of degradation of the structure. R2 indicator** | | | | | | | |
|---|---|---|---|---|---|---|---|
| Damage category | Vertical elements ($A_v$) | | | Horizontal elements ($A_h$) | | | Total |
| | Affected surface | | | Affected surface | | | |
| | ≤1/3 | 1/3…2/3 | >2/3 | ≤1/3 | 1/3…2/3 | >2/3 | $R_2$ |
| Insignifiant | 70 | 70 | 70 | 30 | 30 | 30 | |
| Moderate | 65 | 60 | 50 **(42,50)** | 25 | 20 | 15 **(15.0)** | **57.50** |
| Serious | 50 | 45 | 35 | 20 | 15 | 10 | |
| Very serious | 30 | 25 | 15 | 15 | 10 | 5 | |

The R2 indicator, which represents the level of structural degradation of the building, resulted equal to the following values:

**- BUILDING C8 (1971): R2=57.50 points, which places the structure in class II of seismic risk, from the point of view of the state of degradation of the structural elements in the masonry (acc. to chap.** D.3.2.2. of P100/3-2019);

| **R2 values associated with seismic risk classes** (acc. P100-3/2019, chap. | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_2$ | | | |
| <50 | **50-70** | 70-90 | 90-100 |

### 2.10.3 Building C10-C11

| Criterion | Fulfilled | Moderate unfulfillment | Major unfulfillment |
|---|---|---|---|
| ***1 Damage caused by the earthquake*** | **40** | **21 ÷ 39** | **0 ÷ 20** |
| Beams: plastic deformations, wall warping of the section, cracks and partial ruptures | | 25 | |
| Dissipative bars (links): severe plastic deformations, cracks and partial ruptures | | | 15 |
| Poles: moderate deformations, warping of the soles, incursions into the plastic field (in some poles) | | 25 | |
| Beam / dissipative bar connection – pole: pronounced deformations, breakages of the clamping elements with a decrease in the resistance (without affecting the clamping means that transmit the cutting force) | | | 15 |
| Frame node: pronounced deformations, warping, cracks and partial ruptures of welds | | 35 | |
| Continuity connections of poles and beams: forays into the plastic domain without breaks in continuity elements or means of connection | | | 15 |
| Vertical bracing: buckling, plastic deformation, failure of fasteners | | 35 | |
| Pole base: plastic deformations of the base plate, crossbars, plastic deformations/breaking of the bottom fastening bolts | | 35 | |
| **Criterion average 1** | **[25]** | | |
| *2. Degradation caused by vertical loads* | 20 | 11 ÷ 19 | 0 ÷ 10 |
| Cracks and degradation in floor slabs | | 15 | |
| Loss of overall stability of poles and beams | | 15 | |
| Loss of local stability of pole and beam components | | 15 | |
| **Criterion average 2** | **[15]** | | |
| *3. Degradations caused by deformation loading* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| Degradation caused by deformation loading (support settlement, contractions, temperature action) | | 5 | |
| **Criterion average 3** | **[5]** | | |
| *4. Deterioration caused by faulty execution* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| Deterioration caused by faulty execution of poles, bracing, defects in welded joints, defects in bolted joints) or by interventions on structural elements (reduction of sections of elements by cutting, sawing or drilling) | | | 5 |
| **Criterion average 4** | **[5]** | | |
| *5. Degradation caused by environmental factors* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| - steel (corrosion, peeling) | | 8 | |

| | | | |
|---|---|---|---|
| - joint elements | | | |
| **Criterion average 5** | **[8]** | | |
| **Total** | **[58]** | | |

### 2.10.4 Building C12

| Criterion | Fulfilled | Moderate unfulfillment | Major unfulfillment |
|---|---|---|---|
| ***1 Damage caused by the earthquake*** | **40** | **21 ÷ 39** | **0 ÷ 20** |
| Beams: plastic deformations, wall warping of the section, cracks and partial ruptures | | | 10 |
| Dissipative bars (links): severe plastic deformations, cracks and partial ruptures | | | 0 |
| Poles: moderate deformations, warping of the soles, incursions into the plastic field (in some poles) | | | 15 |
| Beam / dissipative bar connection – pole: pronounced deformations, breakages of the clamping elements with a decrease in the resistance (without affecting the clamping means that transmit the cutting force) | | | 10 |
| Frame node: pronounced deformations, warping, cracks and partial ruptures of welds | | 21 | |
| Continuity connections of poles and beams: forays into the plastic domain without breaks in continuity elements or means of connection | | | 15 |
| Vertical bracing: buckling, plastic deformation, failure of fasteners | | | 0 |
| Pole base: plastic deformations of the base plate, crossbars, plastic deformations/breaking of the bottom fastening bolts | | | 20 |
| **Criterion average 1** | **[11]** | | |
| *2. Degradation caused by vertical loads* | 20 | 11 ÷ 19 | 0 ÷ 10 |
| Cracks and degradation in floor slabs | | 15 | |
| Loss of overall stability of poles and beams | | 15 | |
| Loss of local stability of pole and beam components | | 15 | |
| **Criterion average 2** | **[15]** | | |
| *3. Degradations caused by deformation loading* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| Degradation caused by deformation loading (support settlement, contractions, temperature action) | | 5 | |
| **Criterion average 3** | **[5]** | | |
| *4. Deterioration caused by faulty execution* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| Deterioration caused by faulty execution of poles, bracing, defects in welded joints, defects in bolted joints) or by interventions on structural elements (reduction of sections of elements by cutting, sawing or drilling) | | | 5 |
| **Criterion average 4** | **[5]** | | |
| *5. Degradation caused by environmental factors* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| - steel (corrosion, peeling) | | | 5 |
| - joint elements | | | |
| **Criterion average 5** | **[5]** | | |
| **Total** | **[41]** | | |

### 2.10.5 Building C13

| Criterion | Fulfilled | Moderate unfulfillment | Major unfulfillment |
|---|---|---|---|
| ***1 Damage caused by the earthquake*** | **50** | **26 ÷ 49** | **0 ÷ 25** |

| | | | |
|---|---|---|---|
| Cracks and permanent deformations in critical areas (plastic areas) of poles, walls and beams | | | 25 |
| Fractures and inclined residual cracks produced by shear force in beams | | | 10 |
| Fractures and open longitudinal cracks in poles and/or walls caused by compressive stresses | | 40 | |
| Fractures or inclined cracks produced by shear force in poles and/or walls | | 45 | |
| Shear cracks caused by sliding of reinforcement at joints | | 30 | |
| Anchorage failure and reinforcement bar insertion | | 30 | |
| Pronounced cracking of the floors | | 45 | |
| Degradation of foundations or foundation soil | | 30 | |
| **Criterion average 1** | | **[32]** | |
| ***2. Degradation caused by vertical loads*** | **20** | **11 ÷ 19** | **0 ÷ 10** |
| Cracks and deterioration in beams and floor slabs | | 15 | |
| Cracks and deterioration in poles and walls | | 15 | |
| **Criterion average 2** | | **[15]** | |
| ***3. Degradations caused by deformation loading*** | **10** | **6 ÷ 9** | **1 ÷ 5** |
| Settlement of supports, shrinkage, temperature action, slow flow of concrete | | 6 | |
| **Criterion average 3** | | **[6]** | |
| ***4. Deterioration caused by faulty execution*** | **10** | **6 ÷ 9** | **1 ÷ 5** |
| Segregated concrete, incorrect work joints | | 6 | |
| **Criterion average 4** | | **[6]** | |
| ***5. Degradation caused by environmental factors*** | **10** | **6 ÷ 9** | **1 ÷ 5** |
| Freezing, thawing, chemical or biological corrosive agents, etc., on concrete | | 8 | |
| Freezing, thawing, chemical or biological corrosive agents, etc., on steel reinforcement (including its adhesion properties) | | 8 | |
| **Criterion average 5** | | **[8]** | |
| **Total** | | **[81]** | |

**Building C13 (1968-1971): R2= 81.00**, placing the construction in class III of seismic risk (from the point of view of fulfilling the structural conditions for concrete buildings);

### 2.10.6 Building C17

| State of degradation of vertical structural elements | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| **Scoring intervals** | **70** | **65** | **50** | **25** |
| **(i)** Vertical cracks in parapets, lintels and arches | | | 50 | |
| **(ii)** Inclined cracks in parapets, lintels and arches | | | 50 | |
| **(iii)** Inclined cracks in slits | | | 50 | |
| **(iv)** Crushing of masonry caused by local concentration of compressive stresses, possibly with expulsion of material | | 65 | | |
| **(v)** Horizontal cracks at the ends of slits | | 65 | | |
| **(vi)** Damage at the intersections of walls with a tendency to detach | | 65 | | |
| **(vii)** Vertical cracks/cracks at the connections between perpendicular walls | | 65 | | |
| **(viii)** Local expulsion of masonry from the horizontal elements on which the floors rest | | 65 | | |
| **Total** | | **[59.37]** | | |

| State of degradation of horizontal structural elements |
|---|

| Criterion | Insignificant | Moderate | Serious | Very serious |
|---|---|---|---|---|
| Scoring intervals | 30 | 25 | 20 | 15 |
| Sheets | | | 20 | |
| **Total** | **[20.00]** | | | |

| State of degradation of the structure. R2 indicator | | | | | | | |
|---|---|---|---|---|---|---|---|
| Damage category | Vertical elements ($A_v$) | | | Horizontal elements ($A_h$) | | | Total |
| | Affected surface | | | Affected surface | | | |
| | ≤1/3 | 1/3…2/3 | >2/3 | ≤1/3 | 1/3…2/3 | >2/3 | $R_2$ |
| Insignifiant | 70 | 70 | 70 | 30 | 30 | 30 | |
| Moderate | 65 | 60 **(59)** | 50 | 25 | 20 **(20)** | 15 | **79.37** |
| Serious | 50 | 45 | 35 | 20 | 15 | 10 | |
| Very serious | 30 | 25 | 15 | 15 | 10 | 5 | |

The R2 indicator, which represents the level of structural degradation of the building, resulted equal to the following values:

- **BUILDING C17 (1911-1971): $R_2$=79** points, which places the structure in **class III of seismic risk**, from the point of view of the state of degradation of the structural elements in the masonry (acc. To chap. D.3.2.2. of P100/3-2019);

| R2 values associated with seismic risk classes (acc. to P100-3/2019, chap. 8.1.2) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | II | **III** | IV |
| Values $R_2$ | | | |
| <50 | 50-70 | **70-90** | 90-100 |

### 2.10.7 Building C20

| State of degradation of vertical structural elements | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| **Scoring intervals** | **70** | **65** | **50** | **25** |
| **(i)** Vertical cracks in parapets, lintels and arches | | | 50 | |
| **(ii)** Inclined cracks in parapets, lintels and arches | | | 50 | |
| **(iii)** Inclined cracks in slits | | | 50 | |
| **(iv)** Crushing of masonry caused by local concentration of compressive stresses, possibly with expulsion of material | | 65 | | |
| **(v)** Horizontal cracks at the ends of slits | | 65 | | |
| **(vi)** Damage at the intersections of walls with a tendency to detach | | 65 | | |
| **(vii)** Vertical cracks/cracks at the connections between perpendicular walls | | 65 | | |
| **(viii)** Local expulsion of masonry from the horizontal elements on which the floors rest | | 65 | | |
| **Total** | **[59.37]** | | | |

| State of degradation of horizontal structural elements | | | | |
|---|---|---|---|---|
| Criterion | Insignificant | Moderate | Serious | Very serious |
| Scoring intervals | 30 | 25 | 20 | 15 |
| Sheets | | | 20 | |
| **Total** | **[20.00]** | | | |

| State of degradation of the structure. R2 indicator | | | |
|---|---|---|---|
| Damage category | Vertical elements ($A_v$) | Horizontal elements ($A_h$) | Total |

| | Affected surface | | | Affected surface | | | |
|---|---|---|---|---|---|---|---|
| | ≤1/3 | 1/3…2/3 | >2/3 | ≤1/3 | 1/3…2/3 | >2/3 | $R_2$ |
| Insignifiant | 70 | 70 | 70 | 30 | 30 | 30 | |
| Moderate | 65 | 60 **(59)** | 50 | 25 | 20 **(20)** | 15 | **79.37** |
| Serious | 50 | 45 | 35 | 20 | 15 | 10 | |
| Very serious | 30 | 25 | 15 | 15 | 10 | 5 | |

The R2 indicator, which represents the level of structural degradation of the building, resulted equal to the following values:

**- BUILDING C20 (>1974): R2=79 points, which places the structure in class III of seismic risk, from the point of view of the state of degradation of the structural elements in the masonry (acc. to chap.** D.3.2.2. of P100/3-2019);

| **R2 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.2) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | II | **III** | IV |
| Values $R_2$ | | | |
| <50 | 50-70 | **70-90** | 90-100 |

### 2.10.8 Building C22

Building C22 is partially decommissioned, it is proposed to demolish the building.

### 2.10.9 Building C24; Building C27

| **State of degradation of vertical structural elements** | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| **Scoring intervals** | **70** | **65** | **50** | **25** |
| **(i)** Vertical cracks in parapets, lintels and arches | | | | 25 |
| **(ii)** Inclined cracks in parapets, lintels and arches | | | | 25 |
| **(iii)** Inclined cracks in slits | | | | 25 |
| **(iv)** Crushing of masonry caused by local concentration of compressive stresses, possibly with expulsion of material | | 65 | | |
| **(v)** Horizontal cracks at the ends of slits | | 65 | | |
| **(vi)** Damage at the intersections of walls with a tendency to detach | | 65 | | |
| **(vii)** Vertical cracks/cracks at the connections between perpendicular walls | | | 50 | |
| **(viii)** Local expulsion of masonry from the horizontal elements on which the floors rest | | 65 | | |
| **Total** | **[48.12]** | | | |

| **State of degradation of horizontal structural elements** | | | | |
|---|---|---|---|---|
| Criterion | Insignific ant | Moderate | Serious | Very seriou |
| Scoring intervals | 30 | 25 | 20 | 15 |
| Sheets | | | 20 | |
| **Total** | **[20.00]** | | | |

| **State of degradation of the structure. R2 indicator** | | | | | | | |
|---|---|---|---|---|---|---|---|
| Damage category | Vertical elements ($A_v$) | | | Horizontal elements ($A_h$) | | | Total |
| | Affected surface | | | Affected surface | | | |
| | ≤1/3 | 1/3…2/3 | >2/3 | ≤1/3 | 1/3…2/3 | >2/3 | $R_2$ |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Insignifiant | 70 | 70 | 70 | 30 | 30 | 30 | |
| Moderate | 65 | 60 | 50 (**48,12**) | 25 | **20** | 15 (**20.0**) | **68.12** |
| Serious | 50 | 45 | 35 | 20 | 15 | 10 | |
| Very serious | 30 | 25 | 15 | 15 | 10 | 5 | |

The R2 indicator, which represents the level of structural degradation of the building, resulted equal to the following values:

- **Building C24, C27 (1911-1964): $R_2$=68** points, which places the structure in **class II of seismic risk**, from the point of view of the state of degradation of the structural elements in the masonry (acc. to chap. D.3.2.2. of P100/3-2019);

-

| **R2 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.2) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_2$ | | | |
| <50 | **50-70** | 70-90 | 90-100 |

## 2.10.10 Building C25

| Criterion | Fulfilled | Moderate unfulfillment | Major unfulfillment |
|---|---|---|---|
| ***1 Damage caused by the earthquake*** | **40** | **21 ÷ 39** | **0 ÷ 20** |
| Beams: plastic deformations, wall warping of the section, cracks and partial ruptures | | 25 | |
| Dissipative bars (links): severe plastic deformations, cracks and partial ruptures | | | 15 |
| Poles: moderate deformations, warping of the soles, incursions into the plastic field (in some poles) | | | 15 |
| Beam / dissipative bar connection – pole: pronounced deformations, breakages of the clamping elements with a decrease in the resistance (without affecting the clamping means that transmit the cutting force) | | | 15 |
| Frame node: pronounced deformations, warping, cracks and partial ruptures of welds | | 35 | |
| Continuity connections of poles and beams: forays into the plastic domain without breaks in continuity elements or means of connection | | | 15 |
| Vertical bracing: buckling, plastic deformation, failure of fasteners | | 35 | |
| Pole base: plastic deformations of the base plate, crossbars, plastic deformations/breaking of the bottom fastening bolts | | | 15 |
| **Criterion average 1** | | **[21]** | |
| *2. Degradation caused by vertical loads* | 20 | 11 ÷ 19 | 0 ÷ 10 |
| Cracks and degradation in floor slabs | | 15 | |
| Loss of overall stability of poles and beams | | 15 | |
| Loss of local stability of pole and beam components | | 15 | |
| **Criterion average 2** | | **[15]** | |
| *3. Degradations caused by deformation loading* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| Degradation caused by deformation loading (support settlement, contractions, temperature action) | | 5 | |
| **Criterion average 3** | | **[5]** | |
| *4. Deterioration caused by faulty execution* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| Deterioration caused by faulty execution of poles, bracing, defects in welded joints, defects in bolted joints) or by interventions on | | | 5 |

| | | | |
|---|---|---|---|
| structural elements (reducing the sections of elements by cutting, sawing or drilling) | | | |
| **Criterion average 4** | [5] | | |
| *5. Degradation caused by environmental factors* | 10 | 6 ÷ 9 | 1 ÷ 5 |
| - steel (corrosion, peeling) | | 8 | |
| - joint elements | | | |
| **Criterion average 5** | [8] | | |
| **Total** | [54] | | |

### 2.10.11 Building C26

| **State of degradation of vertical structural elements** | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| **Scoring intervals** | **70** | **65** | **50** | **25** |
| **(i)** Vertical cracks in parapets, lintels and arches | | | | 25 |
| **(ii)** Inclined cracks in parapets, lintels and arches | | | | 25 |
| **(iii)** Inclined cracks in slits | | | | 25 |
| **(iv)** Crushing of masonry caused by local concentration of compressive stresses, possibly with expulsion of material | | | 35 | |
| **(v)** Horizontal cracks at the ends of slits | | | | 25 |
| **(vi)** Damage at the intersections of walls with a tendency to detach | | | | 25 |
| **(vii)** Vertical cracks/cracks at the connections between perpendicular walls | | | 50 | |
| **(viii)** Local expulsion of masonry from the horizontal elements on which the floors rest | | 65 | | |
| **Total** | [34.37] | | | |

| **State of degradation of horizontal structural elements** | | | | |
|---|---|---|---|---|
| Criterion | Insignificant | Moderate | Serious | Very serious |
| Scoring intervals | 30 | 25 | 20 | 15 |
| Sheets | | | 20 | |
| **Total** | [20.00] | | | |

| **State of degradation of the structure. R2 indicator** | | | | | | | |
|---|---|---|---|---|---|---|---|
| Damage category | Vertical elements ($A_v$) | | | Horizontal elements ($A_h$) | | | Total |
| | Affected surface | | | Affected surface | | | |
| | ≤1/3 | 1/3…2/3 | >2/3 | ≤1/3 | 1/3…2/3 | >2/3 | $R_2$ |
| Insignifiant | 70 | 70 | 70 | 30 | 30 | 30 | |
| Moderate | 65 | 60 | 50 | 25 | **20** | 15 **(20.0)** | |
| Serious | 50 | 45 | 35 **(34.37)** | 20 | 15 | 10 | **54.37** |
| Very serious | 30 | 25 | 15 | 15 | 10 | 5 | |

The R2 indicator, which represents the level of structural degradation of the building, resulted equal to the following values:

- **BUILDING C26 (1911-1964): $R_2$ = 54** points, which places the structure in **class II of seismic risk**, from the point of view of the state of degradation of the structural elements in the masonry (acc. To chap. D.3.2.2. of P100/3-2019);

-

| **R2 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.2) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_2$ | | | |
| <50 | **50-70** | 70-90 | 90-100 |

### 2.10.12 Building C28-C29

| State of degradation of vertical structural elements | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| Scoring intervals | 70 | 65 | 50 | |
| **(i)** Vertical cracks in parapets, lintels and arches | | 65 | | |
| **(ii)** Inclined cracks in parapets, lintels and arches | | 65 | | |
| **(iii)** Inclined cracks in slits | | 65 | | |
| **(iv)** Crushing of masonry caused by local concentration of compressive stresses, possibly with expulsion of material | | 65 | | |
| **(v)** Horizontal cracks at the ends of slits | | 65 | | |
| **(vi)** Damage at the intersections of walls with a tendency to detach | | 65 | | |
| **(vii)** Vertical cracks/cracks at the connections between perpendicular walls | | | 50 | |
| **(viii)** Local expulsion of masonry from the horizontal elements on which the floors rest | | 65 | | |
| **Total** | **[63.00]** | | | |

| State of degradation of horizontal structural elements | | | | |
|---|---|---|---|---|
| **Criterion** | **Insignificant** | **Moderate** | **Serious** | **Very serious** |
| Scoring intervals | 30 | 25 | 20 | 15 |
| Sheets | | | 20 | |
| **Total** | **[20.00]** | | | |

| State of degradation of the structure. R2 indicator | | | | | | | |
|---|---|---|---|---|---|---|---|
| Damage category | Vertical elements ($A_v$) | | | Horizontal elements ($A_h$) | | | Total |
| | Affected surface | | | Affected surface | | | |
| | ≤1/3 | 1/3…2/3 | >2/3 | ≤1/3 | 1/3…2/3 | >2/3 | $R_2$ |
| Insignifiant | 70 | 70 | 70 | 30 | 30 | 30 | |
| Moderate | 65 | 60 **(63)** | 50 | 25 | 20 **(20)** | 15 | **83** |
| Serious | 50 | 45 | 35 | 20 | 15 | 10 | |
| Very serious | 30 | 25 | 15 | 15 | 10 | 5 | |

The R2 indicator, which represents the level of structural degradation of the building, resulted equal to the following values:

- **BUILDING C28-C29 (1911-1964; 1964-1971): $R_2$=83** points, which places the structure in **class III of seismic risk**, from the point of view of the state of degradation of the structural elements in the masonry (acc. To chap. D.3.2.2. of P100/3-2019);

| R2 values associated with seismic risk classes (acc. to P100-3/2019, chap. 8.1.2) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | II | **III** | IV |
| Values $R_2$ | | | |
| <50 | 50-70 | **70-90** | 90-100 |

## 2.11 Indicator R3 - Degree of seismic structural insurance

The seismic insurance degree, R3, characterizes the resistance and ductility capacity of the structure, as a whole, in relation to seismic requirements. The seismic insurance degree for the structure, R3, is the minimum of the values determined separately for each orthogonal main horizontal direction considered in the building assessment.

The R3 indicator highlights the resistance and deformability capacity of the structure, as a whole, in relation to seismic requirements and is determined at the base level of the structure.

To determine the seismic load, in the case of the existing structure unaffected by the change of destination (compared to the designed one) or consolidation, the value of the behavior factor was used **q = 1,5;**

### 2.11.1 Building C7

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 328***), the $R_3$ indicator resulted in the following values:

| **Analytical evaluation of the degree of seismic insurance. R3 indicator** | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **53** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

**- Building C7:** with a value of **R3 = 53.00** points**, placing the structure in seismic risk class II** from the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| **R3 values associated with seismic risk classes**<br>(acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_3$ | | | |
| <35 | **35-65** | 65-90 | 90-100 |

### 2.11.2 Building C8

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 336***), the $R_3$ indicator resulted in the following values:

| **Analytical evaluation of the degree of seismic insurance. R3 indicator** | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **54** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

**- Building C8:** with a value of **R3 = 54.00** points**, placing the structure in seismic risk class II** from the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| **R3 values associated with seismic risk classes**<br>(acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_3$ | | | |
| <35 | **35-65** | 65-90 | 90-100 |

### 2.11.3 Building C10-C11

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 345***), the $R_3$ indicator resulted in the following values:

| Analytical evaluation of the degree of seismic insurance. R3 indicator | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **53** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

- **Building C10-C11:** with the value of **R3 = 53.00 points, placing the structure in seismic risk class II** from the point of view of the analytical evaluation of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| **R3 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_3$ | | | |
| <35 | **35-65** | 65-90 | 90-100 |

### 2.11.4 Building C13

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 352***), the $R_3$ indicator resulted in the following values:

| Analytical evaluation of the degree of seismic insurance. R3 indicator | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **86** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

- **Building C13:** with a value of **R3 = 86.00** points**, placing the structure in seismic risk class III from** the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| **R3 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | II | **III** | IV |
| Values $R_3$ | | | |
| <35 | 35-65 | **65-90** | 90-100 |

### 2.11.5 Building C17

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 361***), the $R_3$ indicator resulted in the following values:

| Analytical evaluation of the degree of seismic insurance. R3 indicator | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **45** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

- **Building C17:** with a value of **R3 = 45.00** points**, placing the structure in seismic risk class II** from the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| R3 values associated with seismic risk classes (acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_3$ | | | |
| <35 | **35-65** | 65-90 | 90-100 |

### 2.11.6 Building C20

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 370***), the $R_3$ indicator resulted in the following values:

| Analytical evaluation of the degree of seismic insurance. R3 indicator | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **53** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

**- Building C20:** with a value of **R3 = 53.00** points**, placing the structure in seismic risk class II** from the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| R3 values associated with seismic risk classes (acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_3$ | | | |
| <35 | **35-65** | 65-90 | 90-100 |

### 2.11.7 Building C24; C26; C27

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 378***), the $R_3$ indicator resulted in the following values:

| Analytical evaluation of the degree of seismic insurance. R3 indicator | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **61** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

**- Building C24; C26; C27:** with the value of **R3 = 61.00** points**, placing the structure in seismic risk class II** from the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| R3 values associated with seismic risk classes (acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_3$ | | | |
| <35 | **35-65** | 65-90 | 90-100 |

### 2.11.8 Building C25

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 387***), the $R_3$ indicator resulted in the following values:

| **Analytical evaluation of the degree of seismic insurance. R3 indicator** | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **60** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

**- Building C25:** with a value of **R3 = 60.00** points**, placing the structure in seismic risk class II** from the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| **R3 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | **II** | III | IV |
| Values $R_3$ | | | |
| <35 | **35-65** | 65-90 | 90-100 |

### 2.11.9 Building C28-C29

The structural calculation was carried out according to the provisions of the code P100/3-2019. According to the calculation notes in **Annex C** (***page 394***), the $R_3$ indicator resulted in the following values:

| **Analytical evaluation of the degree of seismic insurance. R3 indicator** | |
|---|---|
| Department | Value |
| Min ($R_{3T}$, $R_{3L}$) | **66** |

The R3 indicator, which highlights the resistance and ductility capacity of the structure, resulted equal to the following values:

**- Building C25:** with a value of **R3 = 66.00** points**, placing the structure in seismic risk class II** from the point of view of the analytical assessment of the degree of seismic insurance (acc. to Chap. 8.1.3 of P100/3-2019);

| **R3 values associated with seismic risk classes** (acc. to P100-3/2019, chap. 8.1.3) | | | |
|---|---|---|---|
| Seismic risk class | | | |
| I | II | **III** | IV |
| Values $R_3$ | | | |
| <35 | 35-65 | **65-90** | 90-100 |

## 2.12 Checking lateral movements

Acc. to Annex E.1 of P100-1/2013, the verification of lateral movements at the serviceability limit state aims to maintain the main function of the building following earthquakes that may occur several times during the life of the building, by limiting the degradation of non-structural elements and components of the building's installations.

Acc. to Annex E.1 of P100-1/2013, the verification of lateral movements at the ultimate limit state aims to avoid loss of human life during a major earthquake by preventing the collapse of non-structural elements.

## 2.13 Evaluation summary

The synthetic expression of the susceptibility of an existing building to seismic damage to the action of the design earthquake, corresponding to the Ultimate Limit State, is done by classifying it in a seismic risk class.

According to P100-3/2019 – Seismic Design Code, Part III, provisions for the seismic assessment of existing buildings, the seismic risk classes are defined as follows:

- **RsI class**, which includes buildings susceptible to total or partial collapse under the design earthquake action corresponding to the Ultimate Limit State;
- **RsII class**, which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely;
- **RsIII class**, which includes buildings susceptible to moderate damage under the design earthquake action corresponding to the Ultimate Limit State, which may endanger the safety of users;
- **RsIV class**, which includes buildings for which the expected seismic response under the effect of the design earthquake, corresponding to the Ultimate Limit State, is similar to that expected for buildings designed based on the technical regulations in force.

The technical expert decides to classify the building in a certain seismic risk class based on the values of the three indicators, the associated seismic risk classes and a complex and comprehensive analysis of the set of conditions of different natures. The relevance of each indicator for the seismic assessment of the building was assessed by the technical expert. The seismic risk class of the building is the minimum class associated with the three indicators R1, R2, R3. By way of exception, when the technical expert assesses that one of the indicators R2, R3 has low relevance in the case of the assessed building, the seismic risk class of the building is the minimum class associated with the other two indicators.

From the qualitative and quantitative assessment, the following classifications in seismic risk classes resulted:

### 2.13.1 Building C7

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **$30 \leq R_1 = 44.00 < 60$** | **RS II** |
| Degree of structural damage – R2 coefficient | **$50 \leq R_2 = 68 < 70$** | **RS II** |
| Insurance level – R3 coefficient | **$35.00 \leq R_3 = 53 < 65$** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| **R3 indicator** | <0.40 | **0,40 – 0,60** | 0.61 - 0.80 | > 0.80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | **High** | Moderate | Low |

- Building C7 is characterized by the value of the indicator **R3 = 0.53 and presents high vulnerability** to seismic actions.

### 2.13.2 Building C8

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **$30 \leq R_1 = 41.00 < 60$** | **RS II** |
| Degree of structural damage – R2 coefficient | **$50 \leq R_2 = 57.5 < 70$** | **RS II** |
| Insurance level – R3 coefficient | **$35.00 \leq R_3 = 54 < 65$** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| R3 indicator | <0.40 | 0,40 – 0,60 | 0,61 – 0,80 | > 0,80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | **High** | Moderate | Low |

- Building C8 is characterized by the value of the **R3 indicator = 0.54 and presents high vulnerability** to seismic actions;

### 2.13.3 Building C10-C11

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **$30 \leq R_1 = 58.00 < 60$** | **RS II** |
| Degree of structural damage – R2 coefficient | **$50 \leq R_2 = 58 < 70$** | **RS II** |
| Insurance level – R3 coefficient | **$35.00 \leq R_3 = 55 < 65$** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| R3 indicator | <0.40 | 0.40 - 0.60 | 0.61 - 0.80 | > 0.80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | **High** | Moderate | Low |

- Building C10 is characterized by the value of the indicator **R3 = 0.55 and presents high vulnerability** to seismic actions;

### 2.13.4 Building C12

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **$30 \leq R_1 = 49.00 < 60$** | **RS II** |
| Degree of structural damage – R2 coefficient | **$R_2 = 41,00 < 50$** | **RS II** |
| Insurance level – R3 coefficient | **$R_3$ = -** | **-** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

### 2.13.5 Building C13

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **$60 \leq R_1 = 72{,}00 < 90$** | **RS III** |
| Degree of structural damage – R2 coefficient | **$70 \leq R_2 = 81{,}00 < 90$** | **RS III** |
| Insurance level – R3 coefficient | **$65 < R_3 = 86.00 < 90$** | **RS III** |
| **Final classification into a seismic risk class** | | **RS III** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS III-**
which includes buildings where structural degradation is expected that does not significantly affect structural safety, but where degradation of non-structural elements may be important

| R3 indicator | <0.40 | 0,40 – 0,60 | 0,61 – 0,80 | > 0,80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | High | **Moderate** | Low |

- Building C13 is characterized by the value of the **R3 indicator = 0.86 and presents moderate vulnerability** to seismic actions.;

### 2.13.6 Building C17

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **$30 \leq R_1 = 46.00 < 60$** | **RS II** |
| Degree of structural damage – R2 coefficient | **$70 \leq R_2 = 79 < 90$** | **RS III** |
| Insurance level – R3 coefficient | **$35.00 \leq R_3 = 45 < 65$** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| R3 indicator | <0.40 | 0,40 – 0,60 | 0,61 – 0,80 | > 0,80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | **High** | Moderate | Low |

- Building C17 is characterized by the value of the indicator **R3 = 0.45 and presents high vulnerability** to seismic actions;

### 2.13.7 Building C20

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **30 ≤ R1 = 41.00 < 90** | **RS II** |
| Degree of structural damage – R2 coefficient | **R2 = 41 < 50** | **RS I** |
| Insurance level – R3 coefficient | **R3 = -** | **-** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

### 2.13.8 Building C24

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **60 ≤ R1 = 66.00 < 90** | **RS II** |
| Degree of structural damage – R2 coefficient | **50 ≤ R2 = 68.24 < 70** | **RS II** |
| Insurance level – R3 coefficient | **35.00 ≤ R3 = 61 < 65** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| **R3 indicator** | **<0.40** | **0,40 – 0,60** | **0,61 – 0,80** | **> 0,80** |
|---|---|---|---|---|
| **Vulnerability** | Very high | High | **Moderate** | Low |

- Building C24 is characterized by the value of the **R3 indicator = 0.61 and presents moderate vulnerability** to seismic actions.;

### 2.13.9 Building C25

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **60≤R1 =63,00<90** | **RS II** |
| Degree of structural damage – R2 coefficient | **50 ≤ R2 = 54 < 70** | **RS II** |
| Insurance level – R3 coefficient | **35.00 ≤ R3 = 60 < 65** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| R3 indicator | <0.40 | 0,40 – 0,60 | 0,61 – 0,80 | > 0,80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | **High** | Moderate | Low |

- Building C25 is characterized by the value of the indicator **R3 = 0.60 and presents high vulnerability** to seismic actions;

### 2.13.10 Building C26

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **60 ≤ R1 = 65.00 < 90** | **RS II** |
| Degree of structural damage – R2 coefficient | **50 ≤ R2 = 54.67 < 70** | **RS II** |
| Insurance level – R3 coefficient | **35.00 ≤ R3 = 61 < 65** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| R3 indicator | <0.40 | 0,40 – 0,60 | 0,61 – 0,80 | > 0,80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | High | **Moderate** | Low |

- Building C26 is characterized by the value of the **R3 indicator = 0.61 and presents moderate vulnerability** to seismic actions.;

### 2.13.11 Building C27

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **60 ≤ R1 = 66.00 < 90** | **RS II** |
| Degree of structural damage – R2 coefficient | **50 ≤ R2 = 68.12 < 70** | **RS II** |
| Insurance level – R3 coefficient | **35.00 ≤ R3 = 61 < 65** | **RS II** |
| **Final classification into a seismic risk class** | | **RS II** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| R3 indicator | <0.40 | 0,40 – 0,60 | 0,61 – 0,80 | > 0,80 |
|---|---|---|---|---|
| **Vulnerability** | Very high | High | **Moderate** | Low |

- Building C27 is characterized by the value of the **R3 indicator = 0.61 and presents moderate vulnerability** to seismic actions.;

### 2.13.12 Building C28-C29

| Final classification in seismic risk class | | |
|---|---|---|
| **Analyzed factor** | **Score** | **Seismic risk class** |
| Degree of fulfillment of seismic design conditions – R1 coefficient | **$60 \leq R_1 = 62.00 < 90$** | **RS II** |
| Degree of structural damage – R2 coefficient | **$70 \leq R_2 = 83 < 90$** | **RS III** |
| Insurance level – R3 coefficient | **$65 < R_3 = 66.00 < 90$** | **RS III** |
| **Final classification into a seismic risk class** | | **RS III** |

Taking into account the three categories of conditions that were the subject of the investigations and analyses carried out within the framework of this expert report, we consider it rational to classify the property being appraised as:

**- SEISMIC RISK CLASS RS II-**
which includes buildings susceptible to major damage under the design earthquake action corresponding to the Ultimate Limit State, which endangers the safety of users, but for which total or partial collapse is unlikely

| **R3 indicator** | **<0.40** | **0,40 – 0,60** | **0,61 – 0,80** | **> 0,80** |
|---|---|---|---|---|
| **Vulnerability** | Very high | High | **Moderate** | Low |

- Building C26 is characterized by the value of the **R3 indicator = 0.61 and presents moderate vulnerability** to seismic actions.;

## 2.14 Proposal for intervention to ensure safety

According to point 8.4 of P100/3-2019, if the R3 indicator shows values lower than 0.65, structural intervention works are necessary to achieve a rational safety level during the expected operating period after consolidation.

**Structural interventions are necessary for the building under analysis.**

Considering the seismic risk class in which the structure is classified and its state of degradation, the following intervention works are proposed, grouped into a **minimal solution (according to point 2.14.1)** and a **maximal one (according to point 2.14.2)**.

### 2.14.1 Minimal intervention option

#### 2.14.1.1 Building C7 – Guard building

A. **Masonry repair works**

**A.1. Injection of cracks in masonry with cement-based mixtures and epoxy resins**

The cracks will be injected with cement-based mixtures and epoxy resins. The injection procedure will be established in agreement with the product data sheet and the technical procedure of the material manufacturer. As a rule, epoxy resin is injected into cracks with an opening smaller than 2mm, and for larger openings,

**A.2. Masonry resurfacing works with dislocations, cracked elements, friability**

Masonry areas with dislocations or friable elements and mortar will be cut out and sealed with materials with mechanical characteristics similar to the existing ones.

**A.3. Injection works in masonry mass with binders based on natural hydraulic lime (foundations)**

The masonry walls will be strengthened by introducing into their volume special binders based on hydraulic lime (natural-NHL with pozzolanic reaction and low salt content), in accordance with the provisions established for such works (Technical Instructions C149-87). Compressive strength of the injected binder, measured on samples at 28 days after manufacture, will be at least 15/N/mm2 (if this strength is not obtained, a cement additive with a reduced salt content of up to 30% can be used in the binder).

B. **Interventions on foundations**

**B.1. Laying out the flanged belts at the wall foundations**

- excavations will be carried out up to 20 cm above the foundation level of the existing walls;
- on a concrete leveling layer of approximately 5 cm, foundation shear belts with a section of 30x40 cm will be arranged;
- in front of the proposed reinforced concrete poles, the belts at the foundation level will be associated by means of braces;
- the reinforced concrete belts will be executed in stages;

**B.2. Waterproofing works**

- the lateral faces of the foundations will be waterproofed with 2 layers of bituminous cardboard; if necessary, the lateral surfaces of the masonry foundations will be leveled with a lime-based mortar;
- immediately below elevation 0, a horizontal hydrophobic barrier will be created in the walls, by lateral injections;

C. **Walls interventions**

**C.1. Arrangement of cores and belts embedded in masonry**

- restoring the continuity of the intersections of the masonry diaphragms in front of the chimney. The soot paths will be cleaned and the masonry will be concreted or demolished and re-laid using similar brick blocks;
- at the corners of the walls, inside the walls, reinforced concrete pillars (embedded in the masonry) with a section of 15x50x50, C20/25 concrete will be arranged;
- reinforced concrete belts will be constructed at the upper level of the walls;

**C.2. Arrangement of reinforced concrete lintels**

- in front of window and door openings, where they do not exist or are degraded, reinforced concrete lintels will be installed;
- the lintels will be provided with a minimum support of 40cm above the masonry walls on the inner faces of the opening;

**D. Floor interventions**

**D.1. Interventions on the floor above the ground floor**

- the existing floor will be dismantled and rebuilt from reinforced concrete;
- the floors will rest on the walls through reinforced concrete belts;

**E. Framing interventions**

- the degraded wooden elements of the framework will be repaired;
- the wooden elements that cannot be saved will be dismantled and replaced with healthy ones;
- additions to the framework structure will be ordered as appropriate to stiffen and improve the load-bearing capacity; the additions will be marked differently from the existing elements;

#### 2.14.1.2 Building C8 - Warehouse

A. **Masonry repair works**

**A.1. Injection of cracks in masonry with cement-based mixtures and epoxy resins**

The cracks will be injected with cement-based mixtures and epoxy resins. The injection procedure will be established in agreement with the product data sheet and the technical procedure of the material manufacturer. As a rule, epoxy resin is injected into cracks with an opening smaller than 2mm, and for larger openings,

**A.2. Masonry resurfacing works with dislocations, cracked elements, friability**

Masonry areas with dislocations or friable elements and mortar will be cut out and sealed with materials with mechanical characteristics similar to the existing ones.

**A.3. Injection works in masonry mass with binders based on natural hydraulic lime (foundations)**

The masonry walls will be strengthened by introducing into their volume special binders based on hydraulic lime (natural-NHL with pozzolanic reaction and low salt content), in accordance with the provisions established for such works (Technical Instructions C149-87). The compressive strength of the injected binder, measured on samples 28 days after manufacture, will be at least 15/N/mm2 (if this strength is not obtained, a cement additive with a reduced salt content of up to 30% can be used in the binder).

B. **Interventions on foundations**

**B.1. Laying out the flanged belts at the wall foundations**

- excavations will be carried out up to 20 cm above the foundation level of the existing walls;
- on a concrete leveling layer of approximately 5 cm, foundation shear belts with a section of 30x40 cm will be arranged;
- in front of the proposed reinforced concrete poles, the belts at the foundation level will be associated by means of braces;
- the reinforced concrete belts will be executed in stages;

**B.2. Waterproofing works**

- the lateral faces of the foundations will be waterproofed with 2 layers of bituminous cardboard; if necessary, the lateral surfaces of the masonry foundations will be leveled with a lime-based mortar;
- immediately below elevation 0, a horizontal hydrophobic barrier will be created in the walls, by lateral injections;

C. **Walls interventions**

it is recommended to structurally separate the building and the adjacent building with a 5 cm joint; these are buildings built in different stages and have different structural and dynamic characteristics as well as different intervention needs and possibilities; the joints can be apparently closed (they will be repaired after each seismic action) or visually treated as joints;

**C.1. Arrangement of reinforced concrete cores and belts embedded in masonry**

- at the corners and at the intersection of the walls, reinforced concrete pillars will be placed (embedded in the masonry);
- new reinforced concrete belts will be constructed;

**C.2. Between the masonry pillars on axis C, masonry with similar characteristics will be placed, resulting in a continuous wall;**

**C.3. Arrangement of reinforced concrete lintels**

- in front of window and door openings, where they do not exist or are degraded, reinforced concrete lintels will be installed;
- the lintels will be supported at least 40 cm above the masonry walls on the inner faces of the opening;

**D. Floor interventions**

D.1. Interventions on the floor above the ground floor

- the existing floor will be dismantled and rebuilt from reinforced concrete;
- the floors will rest on the walls through the embedded belts over the walls;

**E. Framing interventions**

- the degraded wooden elements of the framework will be repaired;
- the wooden elements that cannot be saved will be dismantled and replaced with healthy ones;
- additions to the framework structure will be ordered as appropriate to stiffen and improve the load-bearing capacity; the additions will be marked differently from the existing elements;

### 2.14.1.3 Building C10-C11 – Covered terrace A.

**Interventions on metal poles**

- restoration and replacement of heavily corroded elements -
bracing of the metal structure in plan
- dismantling of brick pillars, inspection of all metal profiles and their restoration reinforced concrete pillars
- cable tensioning B.

**Foundation interventions**

**B.1. Restoration of isolated foundations, especially in the area of corroded poles (Building C11)**

**B.2. Waterproofing works**

- the lateral faces of the foundations will be waterproofed with 2 layers of bituminous cardboard;

C. **Interventions on the metal structure**

- replacement of failed or corroded elements (more than 10% of their cross-section) with new elements, made of similar steel and with similar cross-sections; the joints will be made with rivets, similar to the existing ones;

- the entire metal structure will be cleaned by sandblasting and will be protected against corrosion by painting with special paints, with guaranteed adhesion.

- execution of a program for current monitoring of the operation of the building, in accordance with Normative P130 of 1999, GD 766 of 1997 and Law 10 of 1995 (with subsequent amendments).

### 2.14.1.4 Building C12 - Repository

Considering that the existing structure unloads through the concrete platform and the metal structure does not have foundations but also due to the undersizing of all the elements that make up the structure, we propose the demolition of building C12.

### 2.14.1.5 Building C13 - Kitchen

**A. Dismantling works**

In a first stage, all demolition works will be carried out throughout the building. No other works will be started before the completion of the demolition works provided for in the project.

- demolition of interior partition walls made of brick masonry with a thickness of less than 25cm (half-brick walls); if necessary, these will be replaced with plasterboard walls with a light metal structure properly anchored to the reinforced concrete structural elements;

- the interior and exterior plasters will be rigorously inspected and those in danger of detachment will be dismantled and restored;

- all plasters in the intervention areas will be dismantled;

- all existing layers above the floor above the last level will be dismantled;

- all exterior sidewalks will be dismantled;

- the existing screeds will be removed;

**B. Repair work on existing reinforced concrete elements**

If, following the removal of the finishing layers, deterioration of the reinforced concrete elements is found (cracks, damage to the reinforcement coating, deep damage, corroded reinforcement), these will be recorded by the builder in a survey of the structure, specifying the type, size, position and a photograph. The survey of degradation of existing reinforced concrete elements will be transmitted to the technical expert and structural designer.

Defects in concrete/reinforced concrete elements shall not be covered with finishing layers without applying the remedial measures detailed below..

Repair of defects or degradation of reinforced concrete elements will be carried out in accordance with the C149-87 standard, "Technical instructions on defect repair procedures for concrete and reinforced concrete elements" and the measures indicated below.

The procedures for remediation of the degradations found in existing reinforced concrete elements will be chosen on a case-by-case basis from the following:

Table 2 – pag. 20/normative C149-87: Technical instructions on defect remediation procedures for concrete and reinforced concrete elements

| Item. no. | Type of damage | | Characterization of damage | Remediation procedures | |
|---|---|---|---|---|---|
| | description | notation | | a) Cement-based | b) Epoxy-based |
| 1 | Cracks <0.5mm | fo | opening <0.5mm | sealing with cement paste according to point 2.7 | sealing with epoxy putty according to point 3.12 |
| 2 | Cracks 0.5-2mm | f1 | opening 0.5-2mm | - | injection with epoxy resin according to point 3.13 |
| 3 | Cracks <2mm | f2 | opening <2mm | injection with cement paste according to point 2.11 | injection with epoxy putty according to point 3.14 |
| 4 | Damage in the reinforcement coating | DSA | max. depth 4 cm | cement-based mortar according to point 2.8 | epoxy mortar according to point 3.16 |
| 5 | Reduced depth and surface damage | DASR | max. depth ¼ h and max. surface 0.3m2 | cement-based concrete according to point 2.9 | epoxy concrete according to point 3.16 |
| 6 | Damage of depth and/or large surface | DASM | depth <¼ h and surface >0.3m2<br>depth >¼ h and surface <0.3m2 | - excess concreting acc. to point2.12<br>- shot blasting according to point 2.13 | epoxy concrete according to point 3.16 |
| 7 | Corroded reinforcement | AC | corroded reinforcement | exposing/loosening-replacing reinforcement/passivation/repairing concrete | |

It is generally recommended to carry out remedial procedures based on epoxy mixtures, as these are more efficient and perform better. If there is no staff with experience in using epoxy resins or the necessary equipment and materials, cement-based remedial procedures can be used.

The procedures and materials indicated above for remedial work on reinforced concrete elements are extracted from C149/87. Since since the date of issuance of the regulation, repair materials and installation procedures have advanced technically, they can be replaced with similar products superior in terms of physical and chemical characteristics. Only products technically approved in Romania or the European Union will be used, strictly following the manufacturer's technical sheets.

**C. Floor interventions**

**C.1. Removing existing screeds and replacing them with lightweight perlite screeds**

- the existing screeds will be dismantled and replaced with light perlite screeds with a volume weight of max. 600kg/mc;

- repairs to slabs and beams will be carried out according to chapter B;

**D. *Wall interventions***

**D.1. Masonry resurfacing works with dislocations, cracked elements, friability**

Masonry areas with dislocations or friable elements and mortar will be cut out and sealed with materials with mechanical characteristics similar to the existing ones.

**D.2. Injection works in masonry mass with binders based on natural hydraulic lime (foundations)**

The masonry walls will be strengthened by introducing into their volume special binders based on hydraulic lime (natural-NHL with pozzolanic reaction and low salt content), in accordance with the provisions established for such works (Technical Instructions C149-87). The compressive strength of the injected binder, measured on samples 28 days after manufacture, will be at least 15/N/mm2 (if this strength is not obtained, a cement additive with a reduced salt content of up to 30% can be used in the binder).

**D.3. Arrangement of reinforced concrete lintels**

- in front of window and door openings, where they do not exist or are degraded, reinforced concrete lintels will be installed;
- the lintels will be provided with a minimum support of 40cm above the masonry walls on the inner faces of the opening;

**D.4. Removing the chimney from the masonry E.**
**Attic interventions**

- if there is a need to raise the attic, reinforced concrete pillars 20x20cm/2.5m will be built (with a flared base in the form of a 40x40x15cm bearing for proper anchoring to the reinforced concrete floor of the terrace; the anchoring of the vertical reinforcement of the pillars will be done with chemical anchors in the existing, perimeter, reinforced concrete belts), BCA masonry and a 20x20cm reinforced concrete belt;

**F. Interventions at the floor level above the last level**

- all thermal and waterproofing layers added over time will be stripped;
- repairs will be made to the concrete slab for any reported degradation;
- new layers of hydro-thermal insulation will be made, with the following mentions:
- - any necessary slopes will be made either from light perlite screeds (max. 600 kg/m3), or from the variable thickness of the thermal insulation;
- the maximum load from all layers above the terrace slab will not exceed 250 kg/m2;

### 2.14.1.6 Building C17 - Kitchen

A. **Masonry repair works**

**A.1. Injection of cracks in masonry with cement-based mixtures and epoxy resins**

The cracks will be injected with cement-based mixtures and epoxy resins. The injection procedure will be established in agreement with the product data sheet and the technical procedure of the material manufacturer. As a rule, epoxy resin is injected into cracks with an opening smaller than 2mm, and for larger openings,

**A.2. Masonry resurfacing works with dislocations, cracked elements, friability**

Masonry areas with dislocations or friable elements and mortar will be cut out and sealed with materials with mechanical characteristics similar to the existing ones.

**A.3. Injection works in masonry mass with binders based on natural hydraulic lime (foundations)**

The masonry walls will be strengthened by introducing into their volume special binders based on hydraulic lime (natural-NHL with pozzolanic reaction and low salt content), in accordance with the provisions established for such works (Technical Instructions C149-87). The compressive strength of the injected binder, measured on samples 28 days after manufacture, will be at least 15/N/mm2 (if this strength is not obtained, a cement additive with a reduced salt content of up to 30% can be used in the binder).

B. **Interventions on foundations**

**B.1. Laying out the flanged belts at the wall foundations**

- excavations will be carried out up to 20 cm above the foundation level of the existing walls;
- on a concrete leveling layer of approximately 5 cm, foundation shear belts with a section of 30x40 cm will be arranged;
- in front of the proposed reinforced concrete poles, the belts at the foundation level will be associated by means of braces;
- the reinforced concrete belts will be executed in stages; **B.2.**

**Waterproofing works**

- the lateral faces of the foundations will be waterproofed with 2 layers of bituminous cardboard; if necessary, the lateral surfaces of the masonry foundations will be leveled with a lime-based mortar;
- immediately below elevation 0, a horizontal hydrophobic barrier will be created in the walls, by lateral injections;

C. **Walls interventions**

**C.1. Arrangement of embedded cores in the masonry**

- at the corners of the walls (at the walls built later and vaulted, where no pillars were identified), inside the walls, reinforced concrete pillars (embedded in the masonry) with a section of 15x50x50, C20/25 concrete will be installed;

**C.2. Arrangement of reinforced concrete lintels**

- in front of window and door openings, where they do not exist or are degraded, reinforced concrete lintels will be installed;
- the lintels will be provided with a minimum support of 40cm above the masonry walls on the inner faces of the opening;

D. **Floor interventions**

**D.1. Interventions on the floor above the ground floor**

- the existing floor will be dismantled and rebuilt from reinforced concrete; - the floors will rest on the walls through reinforced concrete belts;

E. **Framing interventions**

- the degraded wooden elements of the framework will be repaired;
- additions to the framework structure will be ordered as appropriate to stiffen and improve the load-bearing capacity; the additions will be marked differently from the existing elements;

#### 2.14.1.7 Building C20 - Kitchen

A. **Masonry repair works**

**A.1. Injection of cracks in masonry with cement-based mixtures and epoxy resins**

The cracks will be injected with cement-based mixtures and epoxy resins. The injection procedure will be established in agreement with the product data sheet and the technical procedure of the material manufacturer. As a rule, epoxy resin is injected into cracks with an opening smaller than 2mm, and for larger openings,

**A.2. Masonry resurfacing works with dislocations, cracked elements, friability**

Masonry areas with dislocations or friable elements and mortar will be cut out and sealed with materials with mechanical characteristics similar to the existing ones.

**A.3. Injection works in masonry mass with binders based on natural hydraulic lime (foundations)**

The masonry walls will be strengthened by introducing into their volume special binders based on hydraulic lime (natural-NHL with pozzolanic reaction and low salt content), in accordance with the provisions established for such works (Technical Instructions C149-87). The compressive strength of the injected binder, measured on samples 28 days after manufacture, will be at least 15/N/mm2 (if this strength is not obtained, a cement additive with a reduced salt content of up to 30% can be used in the binder).

**B. Interventions on foundations**

**B.1. Laying of lap belts at the wall foundations, on both sides of the foundations**

- excavations will be carried out up to 20 cm above the foundation level of the existing walls;
- on a concrete leveling layer of approximately 5 cm, foundation shear belts with a section of 30x40 cm will be arranged;

**B.2. Waterproofing works**

- the lateral faces of the foundations will be waterproofed with 2 layers of bituminous cardboard; if necessary, the lateral surfaces of the masonry foundations will be leveled with a lime-based mortar;
- immediately below elevation 0, a horizontal hydrophobic barrier will be created in the walls, by lateral injections;

C. **Walls interventions**

**C.1. Belt arrangement at the top**

- arranging reinforced concrete belts for walls where they are missing;**C.2. Arrangement of reinforced concrete lintels**

- in front of window and door openings, where they do not exist or are degraded, reinforced concrete lintels will be installed;
- the lintels will be provided with a minimum support of 40cm above the masonry walls on the inner faces of the opening;

D. **Floor interventions**

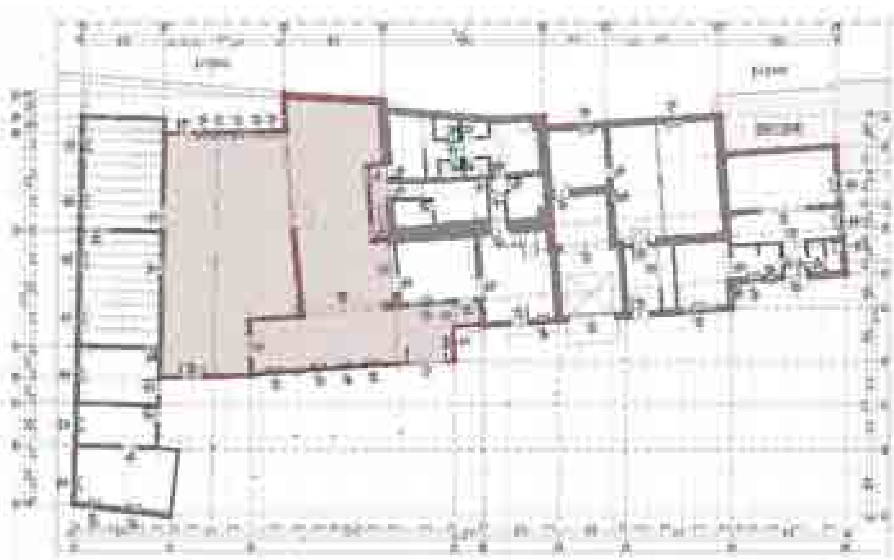
**D.1. Interventions on the floor above the ground floor**

- the existing floor will be dismantled and rebuilt from reinforced concrete; - the floors will rest on the walls through reinforced concrete belts;

E. **Framing interventions**

- the degraded wooden elements of the framework will be repaired;
- the wooden elements that cannot be saved will be dismantled and replaced with healthy ones;
- additions to the framework structure will be ordered as appropriate to stiffen and improve the load-bearing capacity; the additions will be marked differently from the existing elements;

F. ***the metal elements and wooden beams connecting the C17 and C20 buildings (most likely built at the time the C18 building was built), which were built later, will be dismantled***



### 2.14.1.8 Building C22 – Wood repository

Considering that the existing structure unloads through the concrete platform, the metal structure does not have foundations. The existing building being partially decommissioned, we propose the demolition of building C22

### 2.14.1.9 Building C24 – Greenhouse; Building C27 – Greenhouse

A. **Masonry repair works**

**A.1. Injection of cracks in masonry with cement-based mixtures and epoxy resins**

The cracks will be injected with cement-based mixtures and epoxy resins. The injection procedure will be established in agreement with the product data sheet and the technical procedure of the material manufacturer. As a rule, epoxy resin is injected into cracks with an opening smaller than 2mm, and for larger openings,

**A.2. Masonry resurfacing works with dislocations, cracked elements, friability**

Masonry areas with dislocations or friable elements and mortar will be cut out and sealed with materials with mechanical characteristics similar to the existing ones.

**A.3. Injection works in masonry mass with binders based on natural hydraulic lime (foundations)**

The masonry walls will be strengthened by introducing into their volume special binders based on hydraulic lime (natural-NHL with pozzolanic reaction and low salt content), in accordance with the provisions established for such works (Technical Instructions C149-87). The compressive strength of the injected binder, measured on samples 28 days after manufacture, will be at least 15/N/mm2 (if this strength is not obtained, a cement additive with a reduced salt content of up to 30% can be used in the binder).

B. **Interventions on foundations**

**B.1. Layout of internal wall foundation flashing belts**

- excavations will be carried out on the exterior of the building, up to 20 cm above the foundation level of the walls;
- on a concrete leveling layer of approximately 5 cm, foundation shear belts with a section of 30x40 cm will be arranged;
- in front of the proposed reinforced concrete pillars, the belts will be associated with braces;

**B.2. Waterproofing works**

- the lateral faces of the foundations will be waterproofed with 2 layers of bituminous cardboard; if necessary, the lateral surfaces of the masonry foundations will be leveled with a lime-based mortar;
- immediately below elevation 0, a horizontal hydrophobic barrier will be created in the walls, by lateral injections;

C. **Walls interventions**

**C.1. Wall cladding**

- the walls will be sheathed on the inside with a reinforced plaster, M100T, 6 cm thick, reinforced with bars ∅6 / 10 52. The mixture is made with a low consistency - settlement of about 20 ÷ 25 cm on the standard cone of 30 cm height - according to CR6 - 2013

- preparation of the masonry surface: cleaning, drying, etc. Cleaning will be done by rubbing with a wire brush and the masonry joints will be deepened by about 1.5 – 2.0 cm with well-sharpened pliers, it will be blown and washed with a water or sand jet;
- restoring the continuity of the masonry. After completely removing the interior and exterior plasters, the masonry diaphragms will be investigated and where cracks or fissures with an opening of <10 mm are noted, they will be manually injected with cement slurry and the addition of 40% glue. For fractures larger than 10 mm, rebuilding the area with brick blocks of

the same type or possibly concreting with fluid microconcrete (injection of existing cracks with SIKA type materials (special cement mortars) on both sides) will be used;

**C.2. Creation of a reinforced concrete belt, embedded in the masonry.**

- reinforced concrete belts embedded in the masonry will be executed on their upper part. **C.3. Arrangement of reinforced concrete lintels**

- in front of window and door openings, where they do not exist or are degraded, reinforced concrete lintels will be installed;
- the lintels will be supported at least 40 cm above the masonry walls on the inner faces of the opening;
- decoupling the structure from the main structure by doubling the 2 common walls, with one wall each inside the building C24 and C27;

D. **Interventions to the metal structure**

- ensuring the proper anchoring of the vertical elements of the metal structure to the reinforced concrete belts; an articulated joint solution will be designed;

- the entire metal structure will be cleaned by sandblasting and will be protected against corrosion by painting with special paints, with guaranteed adhesion.

#### 2.14.1.10 Building C25 – Greenhouse

- considering the joint of the pillars, which were probably replaced as a result of the strong corrosion of the metal profiles, they were made improperly, it is mandatory to restore them

- restoring the metal bracings on the perimeter of the building analyzed;

- replacing the failed or corroded elements (more than 10% of their cross-section) with new elements, made of a similar steel and with similar cross-sections; the joints will be made with rivets, similar to the existing ones;

- the entire metal structure will be cleaned by sandblasting and will be protected against corrosion by painting with special paints, with guaranteed adhesion.

- dismantling the glass panels and replacing them with ones calculated for the corresponding loads from snow and wind; special attention will be paid to their anchoring to the metal structure uprights;

- mandatory provision of a de-icing system, with continuous operation during winter to avoid the deposition and accumulation of snow on the roof; it is mentioned that the metal structure is not capable of supporting the load corresponding to the action of snow on the studied site; the operation of the de-icing system will be constantly monitored and controlled by a person specially trained in this regard;

- execution of a program for current monitoring during the exploitation of the building, in accordance with Normative P130 of 1999, GD 766 of 1997 and Law 10 of 1995 (with subsequent amendments).

#### 2.14.1.11 Building C28-C29 – Repository

A. **Masonry repair works**

**A.1. Injection of cracks in masonry with cement-based mixtures and epoxy resins**

The cracks will be injected with cement-based mixtures and epoxy resins. The injection procedure will be established in agreement with the product data sheet and the technical procedure of the

the material manufacturer. As a rule, epoxy resin is injected into cracks with an opening smaller than 2mm, and for larger openings,

**A.2. Injection works in masonry mass with binders based on natural hydraulic lime (foundations)**

The masonry walls will be strengthened by introducing into their volume special binders based on hydraulic lime (natural-NHL with pozzolanic reaction and low salt content), in accordance with the provisions established for such works (Technical Instructions C149-87). The compressive strength of the injected binder, measured on samples 28 days after manufacture, will be at least 15/N/mm2 (if this strength is not obtained, a cement additive with a reduced salt content of up to 30% can be used in the binder).

B. **Interventions on foundations**

**B.1. Laying out the flanged belts at the wall foundations**

- excavations will be carried out up to 20 cm above the foundation level of the existing walls;
- on a concrete leveling layer of approximately 5 cm, foundation shear belts with a section of 30x40 cm will be arranged;
- in front of the proposed reinforced concrete poles, the belts at the foundation level will be associated by means of braces;
- the reinforced concrete belts will be executed in stages;

**B.2. Waterproofing works**

- the lateral faces of the foundations will be waterproofed with 2 layers of bituminous cardboard; if necessary, the lateral surfaces of the masonry foundations will be leveled with a lime-based mortar;
- immediately below elevation 0, a horizontal hydrophobic barrier will be created in the walls, by lateral injections;

C. **Walls interventions**

**C.1. Construction of a reinforced concrete belt**

- reinforced concrete belts embedded in the masonry of the walls built later (axis wall 5) will be executed

**C.2. Between the masonry and concrete poles in axis D, masonry with similar characteristics will be placed, resulting in a continuous wall;**

**C.3. Arrangement of reinforced concrete lintels**

- in front of window and door openings, where they do not exist or are degraded, reinforced concrete lintels will be installed;
- the lintels will be supported at least 40 cm above the masonry walls on the inner faces of the opening;

D. **Floor interventions**

- the existing wooden floor is braced in plan;

E. **Framing interventions**

- the degraded wooden elements of the frame will be repaired; **For**

**buildings where substructures are necessary:**

**During the interventions at the foundation level, it is mandatory to build supports for the banks and walls.**

– *before starting the work, supports will be made to the walls,*
– *substructures will be made of plain concrete C8/10 and foundation linings, ensuring the foundation depth provided for in the geotechnical study and the foundation width resulting from the calculations, with a minimum embedment of 20cm in the good foundation ground,*

– the foundation linings will be made of concrete C16/20, 10cm thick, reinforced with bars / meshes Φ10/15 from PC52,

– at the upper part of the foundations, two shear belts will be executed, connected to each other with reinforced concrete posts at approx. 100-150cm,

– *the sub-buildings will be executed alternatively in sections of approx. 1.00m, according to specific technologies, in the order 1, 4, 2, 5, 3, 1, 4, 2, 5, 3, etc. so that between two sub-buildings there is a time of 28 days for the concrete in the sub-buildings to harden,*

– *all work will be carried out with all protection and occupational safety measures in place.*

According to table 1 for ag = 0.30g, constructions with walls made of ZNA are not allowed, confined masonry is required, according to P100-2013, chap. 8.5.4.3.1. Provisions regarding pillars, therefore at both ends of all spandrels.

If the minimal intervention solution is executed, for buildings classified in seismic risk class I or II, from the analyzed buildings, it **will be classified in the RSIV seismic risk class**.

### 2.14.2 The maximum intervention option

Compared to the intervention works proposed in the minimal version, the following are additionally recommended:

- lining the solid brick masonry walls on both sides with a M100 reinforced plaster.

| | |
|---|---|
| **Technical expert:** | eng. IOAN ROTARESCU |
| **Signature:** | |

| | |
|---|---|
| **Technical expert:** | eng. DANIEL DIACONU |
| **Signature:** | |

## 3. CONCLUSIONS AND RECOMMENDATIONS

*According to P100-3/2019, chapter 3.3, art.5, in the case of buildings belonging entirely to the public or private domain of the state or administrative-territorial units, where the intervention works are accompanied by capital repair works, the type and scope of the intervention works are established so that, after their completion, the building can be classified in the RSIV seismic risk class.*

*According to P100-3/2019, chapter 2.1, art.9, in the case of carrying out the recommended intervention works, the technical expertise may be completed, detailed or final upon completion of the works to uncover the structural elements, a situation that may influence the volume, costs and duration of the seismic rehabilitation works.*

*According to P100-3/2019, chapter 1.1, art.23, the conclusions and recommendations of the technical expertise become obsolete in the event of a change in the normative documents compared to those in force at the date of the expertise, in the event of a significant change in the state of degradation of the building compared to the situation at the time of the expertise or when there have been changes to the building regarding: function, structural system or non-structural components. The evolution of the state of degradation of the building, compared to the situation at the time of the expertise, is recorded by a technical expert..*

*By signing the minutes of receipt of this technical expertise, the beneficiary/person designated by him/her confirms that he/she has been informed of and agreed with the content of the expertise, the classification in the seismic risk class, the intervention measures and the seismic risk class in which the building will be classified after the execution of the recommended intervention works.*

*According to art. 12, par. II of Decision no. 742/2018 on the amendment of Government Decision no. 925/1995, the conclusions and, where applicable, the solutions and intervention measures proposed and substantiated by the technical expert in the technical expertise report are adopted by the owner/administrator of the construction and form the basis of the intervention decision to make the construction safe in order to achieve the applicable fundamental requirements or its dismantling, as the case may be.*

| | |
|---|---|
| **Technical expert:** | eng. IOAN ROTARESCU |
| **Signature:** | |

| | |
|---|---|
| **Technical expert:** | eng. DANIEL DIACONU |
| **Signature:** | |

# Annex A – GEOMETRIC SURVEY

**Building C7**
R.7.1. C7– PLAN
R.7.2. C7– SECTIONS ST-01, SL-01
R.7.3.C7- FACADES F-01, F-02, F-03, F-04

**Building C8**
R.8.1. C8– PLAN
R.8.2. C8– SECTIONS ST-01, SL-01
R.8.3.C8 - FACADES F-01, F-02

**Building C9-10**
R.9.1. C9– PLAN
R.9.2. C9- FACADES F-01, F-02, F-03, F-04
R.9.3. C9– SECTIONS ST-01, SL-01

**Building C12**
R.12.1. C12-PLAN
R.12.2. C12-SECTIONS ST-01, SL-01
R.12.3. C12- FACADES F-01, F-02, F-03, F-04

**Building C13**
R.13.1. C13-PLAN
R.13.2. C13-SECTIONS ST-01, ST-02
R.13.3. C13-FACADES F-01, F-03
R.13.4. C13-FACADES F-02, F-04

**Building C15**
R.15.1. C15-PLAN
R.15.2. C15-FACADES F-01, F-02, F-03, F-04
R.15.3. C15-SECTIONS S-01, S-02

**Building C16**
R.16.1. C16-PLAN
R.16.2. C16-SECTIONS S-01, S-02
R.16.3. C16-FACADES F-01, F-02, F-03, F-04
**Building C17**

**Building C17; Building C20**
R.17.1. C17-20-PLAN
R.17.2. C17-20-SECTIONS ST-01, ST-02, ST-03, SL-01
R.17.4. C17-20-FACADES F-01, F-02
R.17.5. C17-20-FACADES F-03, F-04

**Building C22** R.22.1.
C22-PLAN
R.22.2. C22-FACADES F-01, F-02, F-03, F-04
R.22.3. C22-SECTIONS S-01, S-02

**Building C24; Building C26; Building C27**
R.24.1. C24-C26-C27- PLAN
R.24.2. C24-C26-C27- FACADES F-01, F-02, F-03, F-04
R.24.3. C24-C26-C27- SECTIONS S-01, S-02

**Building C25**
R.25.1. C25-PLAN
R.25.2. C25-FACADES F-01, F-02, F-03, F-04
R.25.3. C25-SECTIONS S-01, S-02

**Building C28-C29**
R.28.1. C28-C29-PLAN
R.28.2. C28-C29-SECTIONS ST-01, SL-01
R.28.3. C28-C29- FACADES F-01, F-02, F-03, F-04

# Annex B – PHOTOGRAPHIC SURVEY

## Annex B.1 – Building C7

*Photo no. 1 -*

Ground floor plan

*Photo no. 2 -*

Ground floor plan

| Photo no. 3 - | Ground floor plan |
|---|---|

| Photo no. 4 - | Ground floor plan |
|---|---|

*Photo no. 5 -* Ground floor plan

*Photo no. 6 -* Ground floor plan

| *Photo no. 7 -* | Ground floor plan |
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| *Photo no. 8 -* | Ground floor plan |
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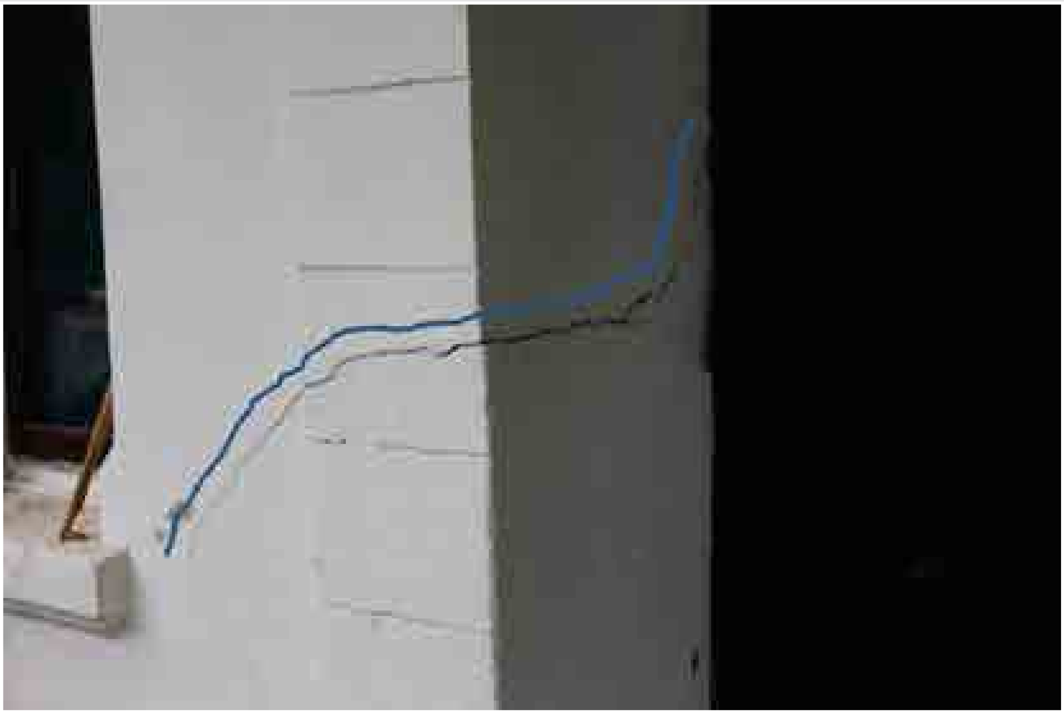
| *Photo no. 9 -* | Ground floor plan |
|---|---|



| *Photo no. 10 -* | Ground floor plan |
|---|---|

| *Photo no. 11 -* | Ground floor plan |
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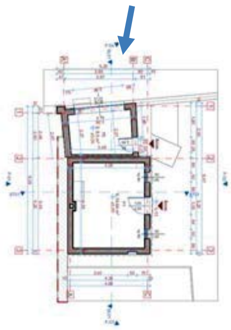
| *Photo no. 12 -* | Ground floor plan |
|---|---|

*Photo no. 13* -

Ground floor plan

*Photo no. 14* -

Ground floor plan



| *Photo no. 15 -* | Ground floor plan |
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| *Photo no. 16 -* | Ground floor plan |
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| *Photo no. 17 -* | Ground floor plan |
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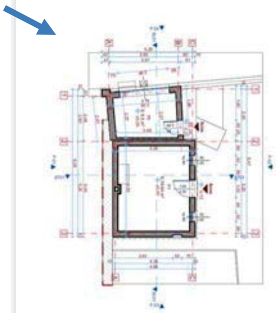


| *Photo no. 18 -* | Ground floor plan |
|---|---|

## Annex B.2 – Building C8

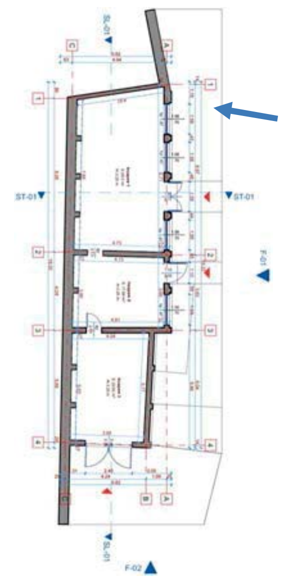
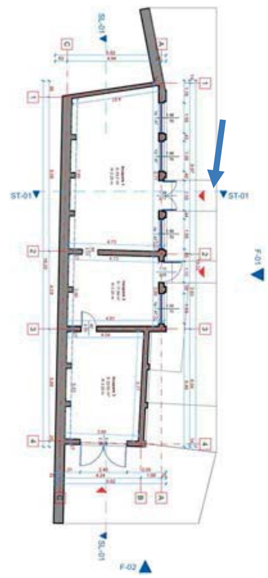


*Photo no. 19 -* | Ground floor plan

*Photo no. 20 -* | Ground floor plan



| *Photo no. 21 -* | Ground floor plan |
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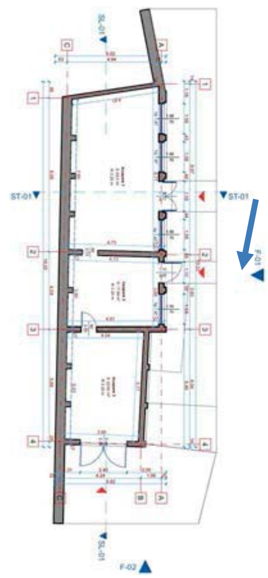


| *Photo no. 22 -* | Ground floor plan |
|---|---|

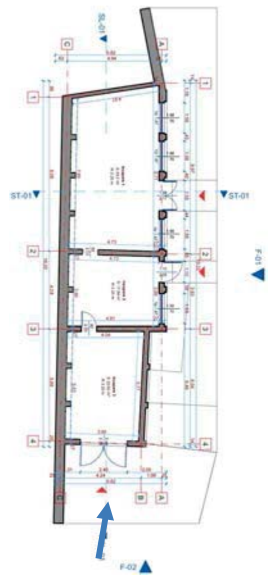



*Photo no. 23 -* | Ground floor plan


*Photo no. 24 -* | Ground floor plan

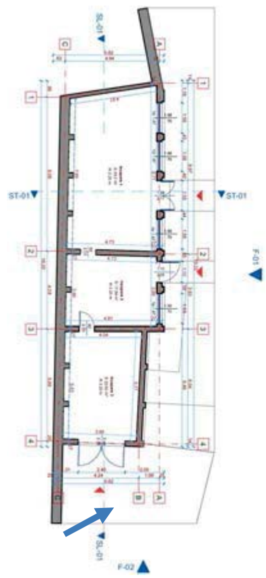


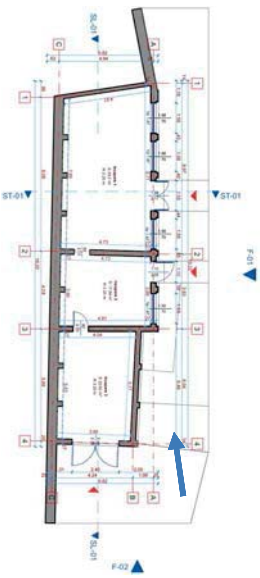
*Photo no. 25 -*

Ground floor plan

*Photo no. 26 -*

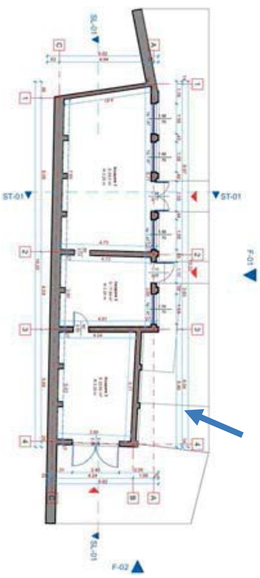
Ground floor plan

*Photo no. 27 -*



Ground floor plan

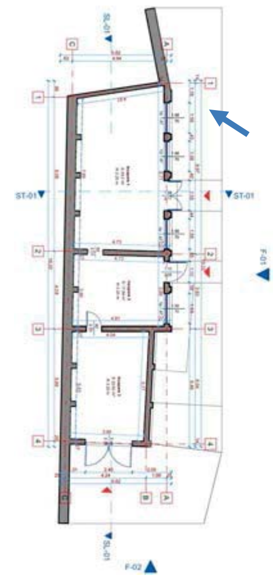
*Photo no. 28 -*



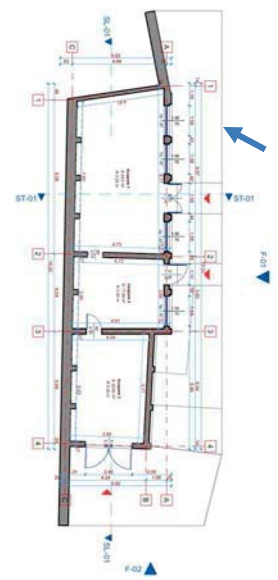
Ground floor plan


*Photo no. 29* - | Ground floor plan


*Photo no. 30* - | Ground floor plan



| Photo no. 31 - | Ground floor plan |
|---|---|



| Photo no. 32 - | Ground floor plan |
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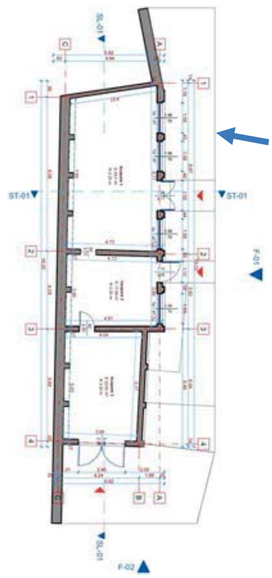

| *Photo no. 33 -* | Ground floor plan |
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| *Photo no. 34 -* | Ground floor plan |
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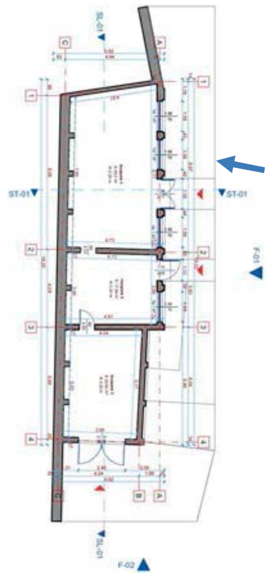
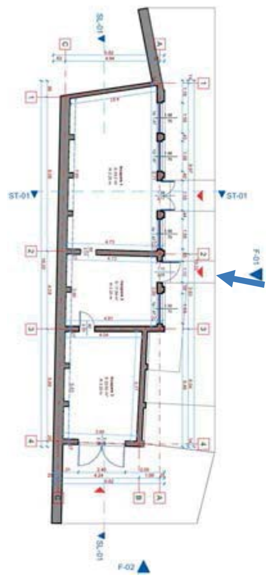


*Photo no. 35 -*

Ground floor plan

*Photo no. 36 -*

Ground floor plan



| *Photo no. 37 -* | Ground floor plan |
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| *Photo no. 38 -* | Ground floor plan |
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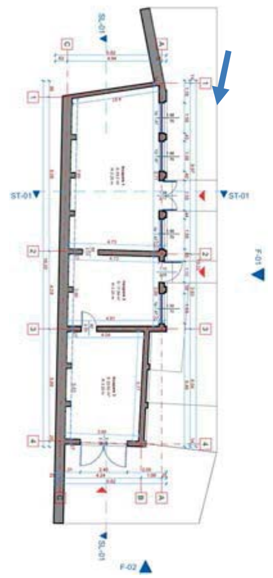

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|---|---|
| *Photo no. 39* - | Ground floor plan |

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|---|---|
| *Photo no. 40* - | Ground floor plan |

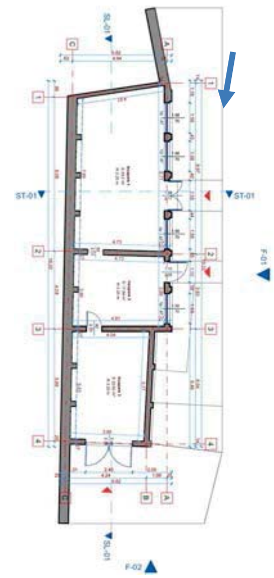

*Photo no. 41 -*

Ground floor plan

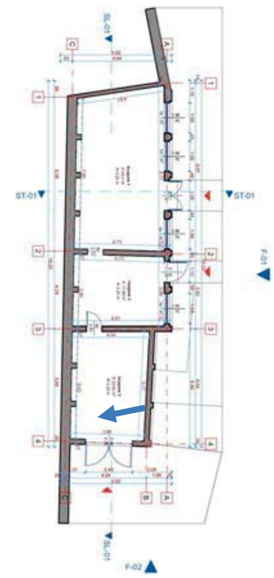

*Photo no. 42 -*

Ground floor plan


| *Photo no. 43 -* | Ground floor plan |
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| *Photo no. 44 -* | Ground floor plan |
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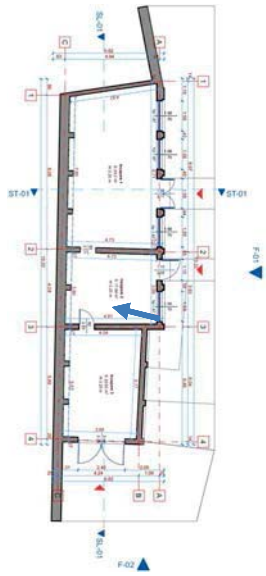

*Photo no. 45 -*

Ground floor plan

*Photo no. 46 -*

Ground floor plan

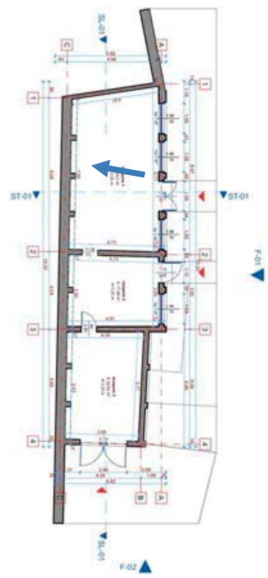

*Photo no. 47* - Ground floor plan

*Photo no. 48* - Ground floor plan

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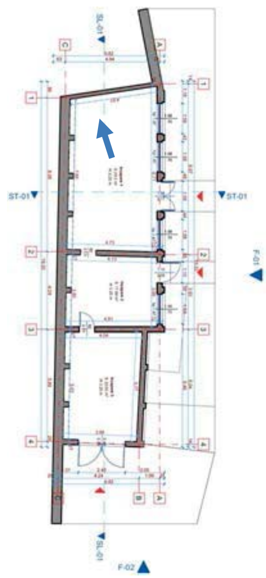
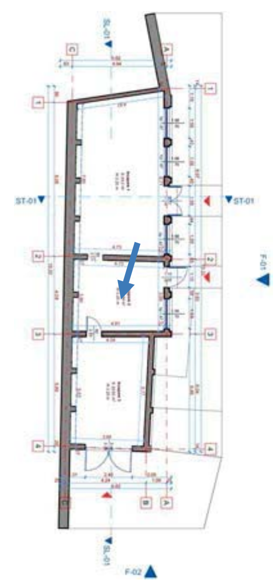
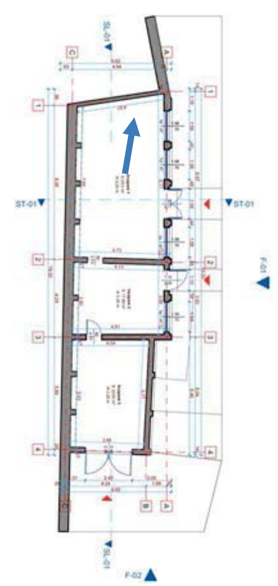
|---|---|
| *Photo no. 49 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 50 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 51 -* | Ground floor plan |



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|---|---|
| *Photo no. 52 -* | Ground floor plan |

## Annex B.3 – Building C10

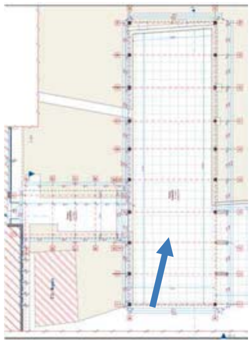



*Photo no. 53 -* Ground floor plan

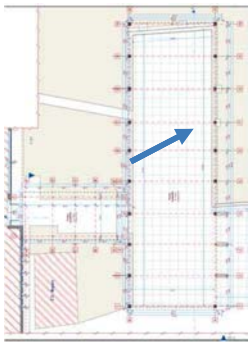



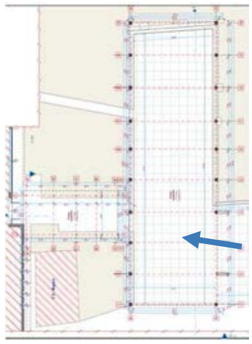
*Photo no. 54 -* Ground floor plan

*Photo no. 55* -

Ground floor plan

*Photo no. 56* -

Ground floor plan



| *Photo no. 57* - | Ground floor plan |
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| *Photo no. 58* - | Ground floor plan |
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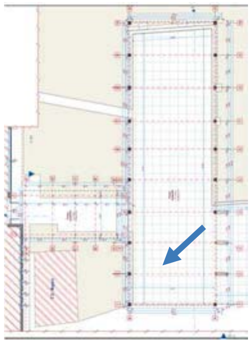
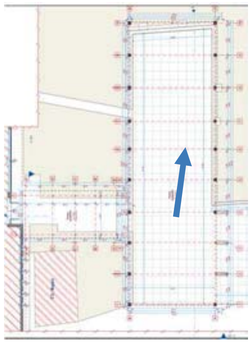


| *Photo no. 59* - | Ground floor plan |
|---|---|



| *Photo no. 60* - | Ground floor plan |
|---|---|

*Photo no. 61 -* Ground floor plan

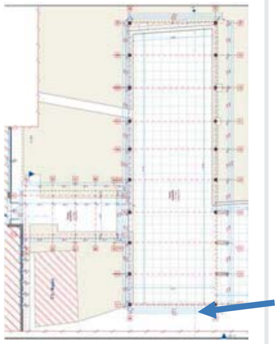


*Photo no. 62 -* Ground floor plan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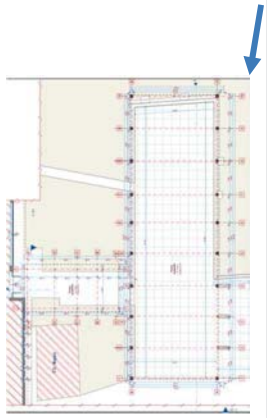
|---|---|
| *Photo no. 63 -* | Ground floor plan |

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|---|---|
| *Photo no. 64 -* | Ground floor plan |

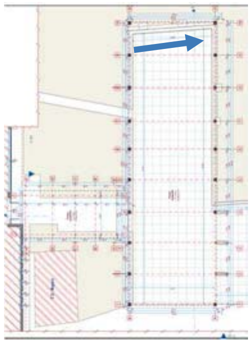

*Photo no. 65 -* Ground floor plan

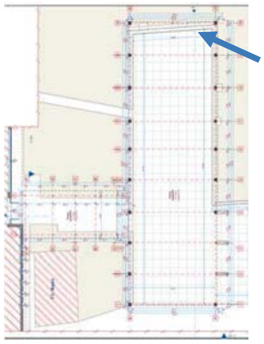

*Photo no. 66 -* Ground floor plan

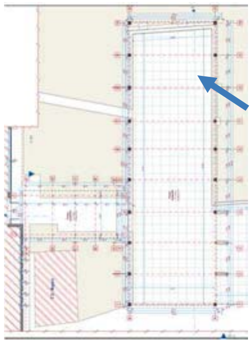

*Photo no. 67 -*

Ground floor plan

*Photo no. 68 -*

Ground floor plan

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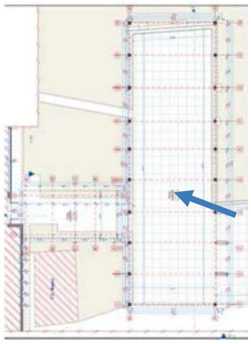
|---|---|
|  |  |
| *Photo no. 69* - | Ground floor plan |

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| *Photo no. 70* - | Ground floor plan |

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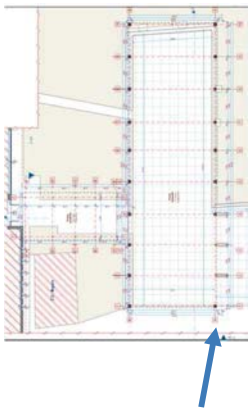
|---|---|
| *Photo no. 71 -* | Ground floor plan |



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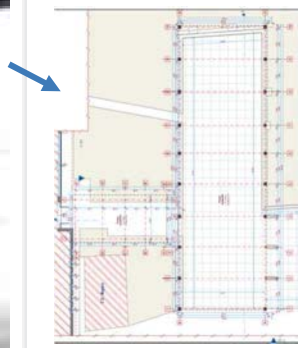
|---|---|
| *Photo no. 72 -* | Ground floor plan |



| *Photo no. 73 -* | Ground floor plan |
|---|---|

| *Photo no. 74 -* | Ground floor plan |
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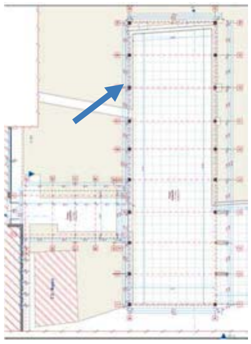


*Photo no. 75* - | Ground floor plan

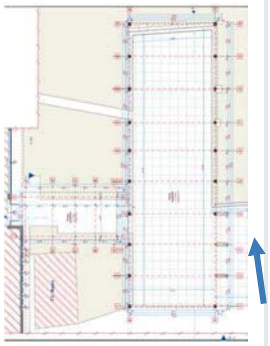


*Photo no. 76* - | Ground floor plan

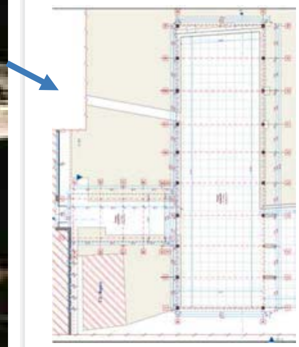


*Photo no. 77 -* Ground floor plan

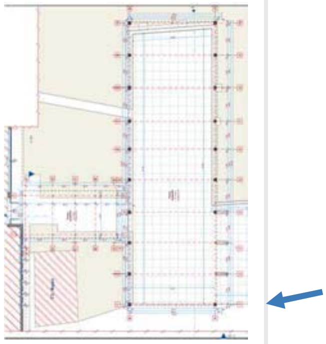


*Photo no. 78 -* Ground floor plan

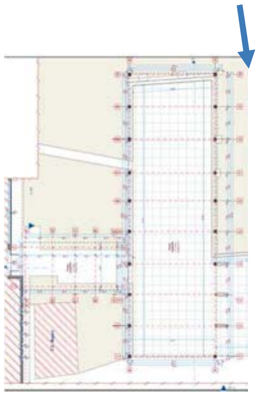


*Photo no. 79* - | Ground floor plan

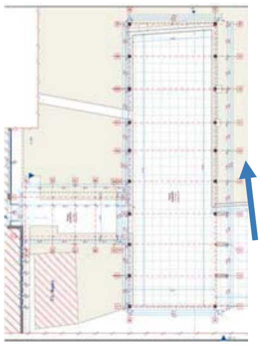


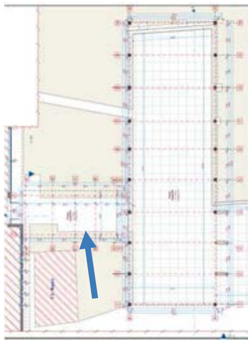
*Photo no. 80* - | Ground floor plan

*Photo no. 81 -*

Ground floor plan

*Photo no. 82 -*

Ground floor plan



| *Photo no. 83 -* | Ground floor plan |
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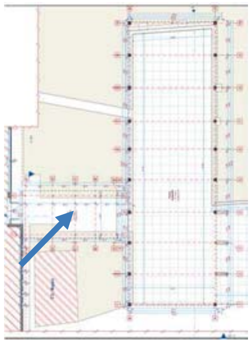

| *Photo no. 84 -* | Ground floor plan |
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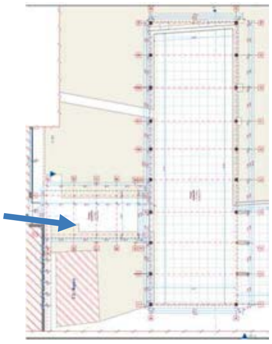
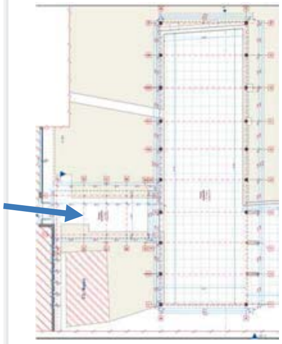


| *Photo no. 85 -* | Ground floor plan |
|---|---|



| *Photo no. 86 -* | Ground floor plan |
|---|---|

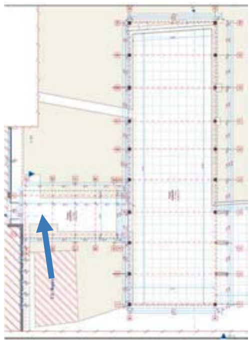


*Photo no. 87 -* Ground floor plan

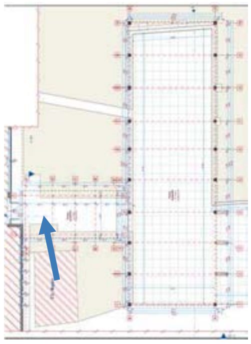


*Photo no. 88 -* Ground floor plan

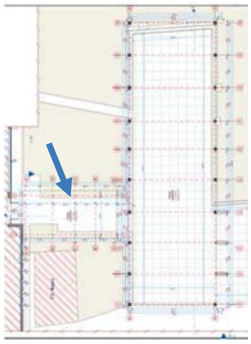


*Photo no. 89* - Ground floor plan

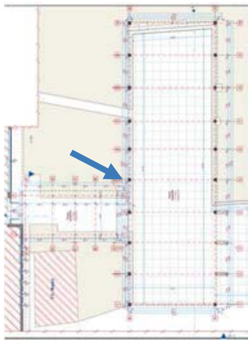
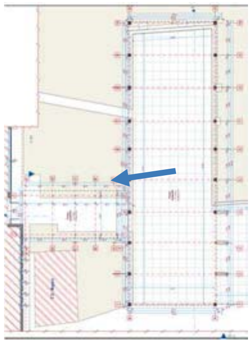
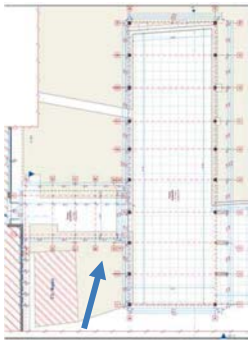


*Photo no. 90* - Ground floor plan



| *Photo no. 91 -* | Ground floor plan |
|---|---|



| *Photo no. 92 -* | Ground floor plan |
|---|---|

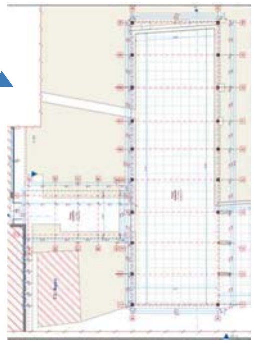


*Photo no. 93 -* | Ground floor plan

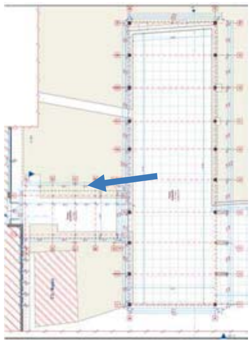


*Photo no. 94 -* | Ground floor plan

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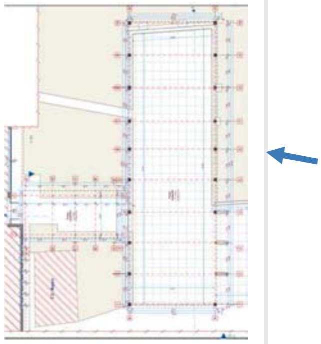
|---|---|
|  |  |
| *Photo no. 95 -* | Ground floor plan |

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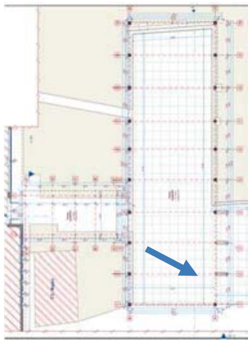
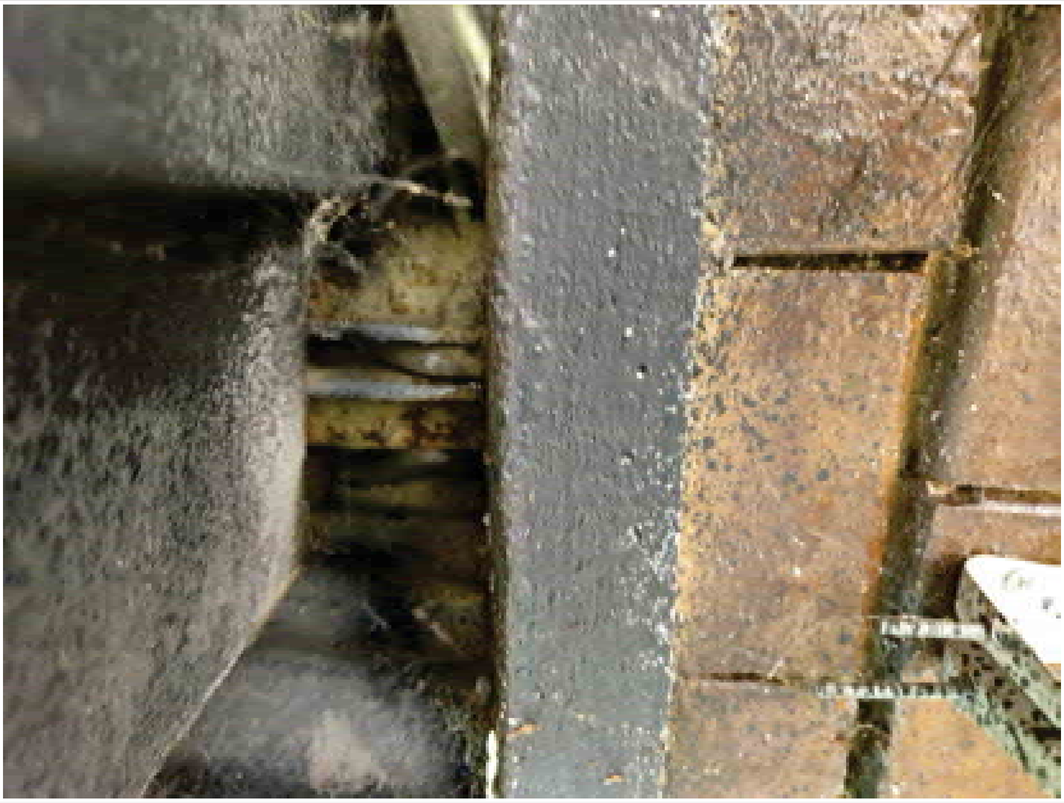
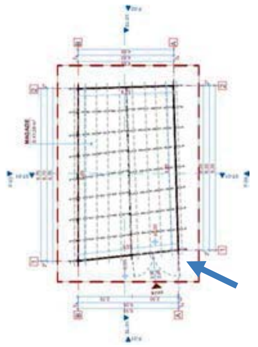
|---|---|
|  |  |
| *Photo no. 96 -* | Ground floor plan |



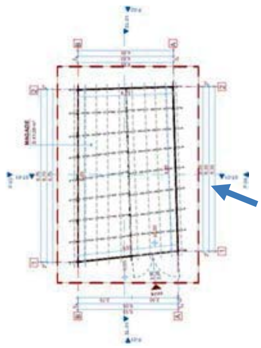
| *Photo no. 97 -* | Ground floor plan |
|---|---|

| *Photo no. 98 -* | Ground floor plan |
|---|---|

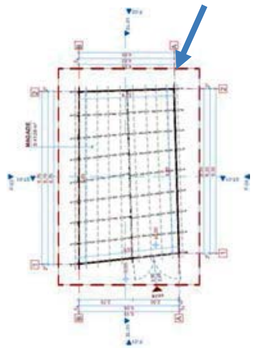
## Annex B.4 – Building C12



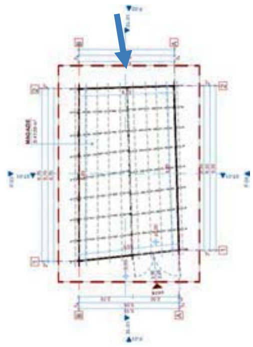
| *Photo no. 99 -* | Ground floor plan |
|---|---|



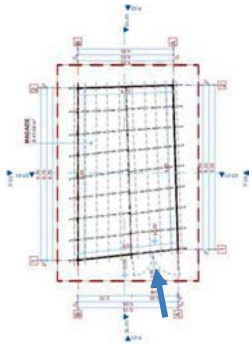
| *Photo no. 100 -* | Ground floor plan |
|---|---|

| *Photo no. 101 -* | Ground floor plan |
|---|---|

| *Photo no. 102 -* | Ground floor plan |
|---|---|



| *Photo no. 103 -* | Ground floor plan |
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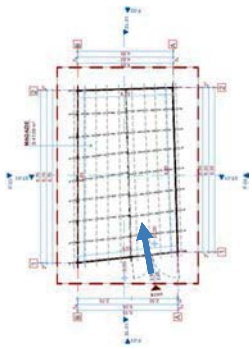

| *Photo no. 104 -* | Ground floor plan |
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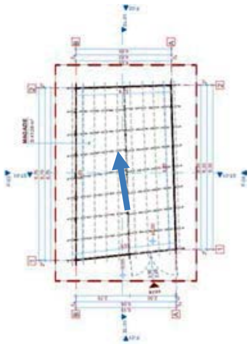

*Photo no. 105 -* Ground floor plan

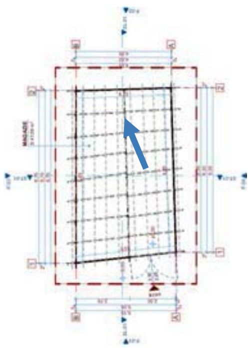

*Photo no. 106 -* Ground floor plan

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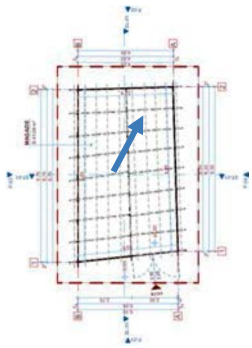
|  |  |
| *Photo no. 107 -* | Ground floor plan |

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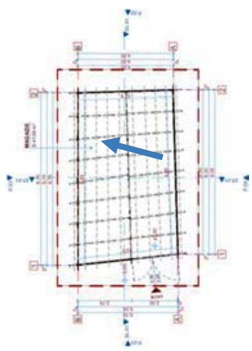
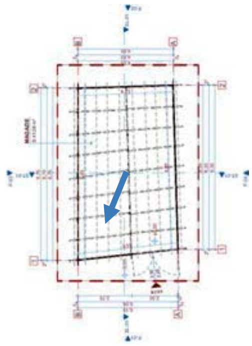
|  |  |
| *Photo no. 108 -* | Ground floor plan |



| Photo no. 109 - | Ground floor plan |
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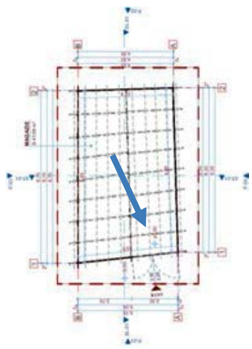

| Photo no. 110 - | Ground floor plan |
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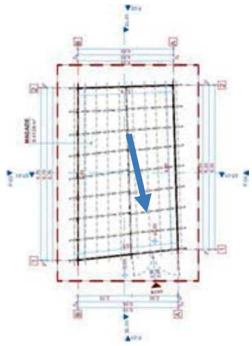
|  |  |
| *Photo no. 111 -* | Ground floor plan |

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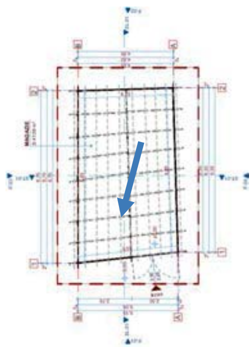
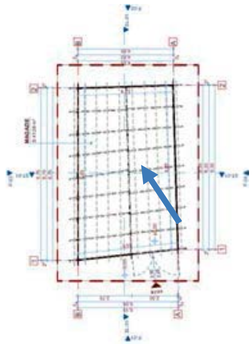
|  |  |
| *Photo no. 112 -* | Ground floor plan |



| *Photo no. 113 -* | Ground floor plan |
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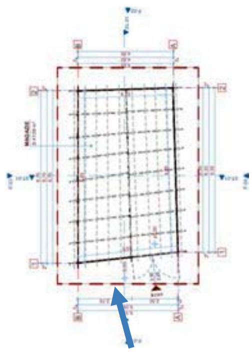

| *Photo no. 114 -* | Ground floor plan |
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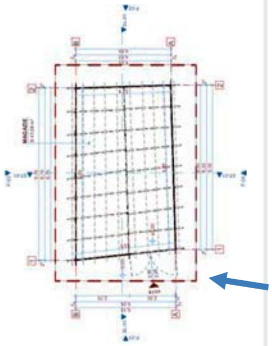


*Photo no. 115 -* Ground floor plan

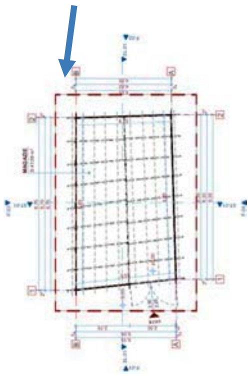
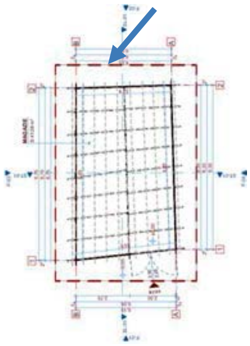


*Photo no. 116 -* Ground floor plan

|  |  |
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| *Photo no. 117* - | Ground floor plan |

## Annex B.5 – Building C13

| *Photo no. 118 -* | Ground floor plan |
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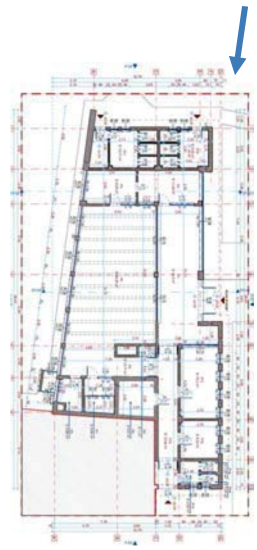

| *Photo no. 119 -* | Ground floor plan |
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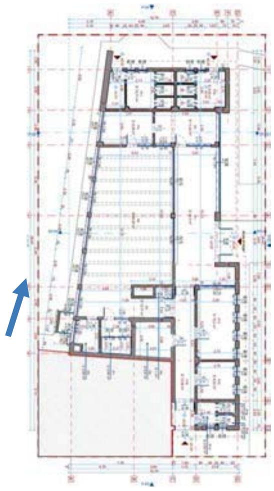


*Photo no. 120 -* Ground floor plan

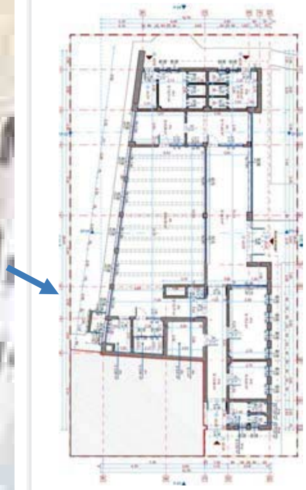
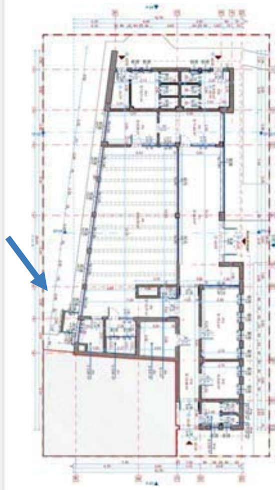



*Photo no. 121 -* Ground floor plan

| *Photo no. 122 -* | Ground floor plan |
|---|---|



| *Photo no. 123 -* | Ground floor plan |
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| *Photo no. 124 -* | Ground floor plan |
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| *Photo no. 125 -* | Ground floor plan |
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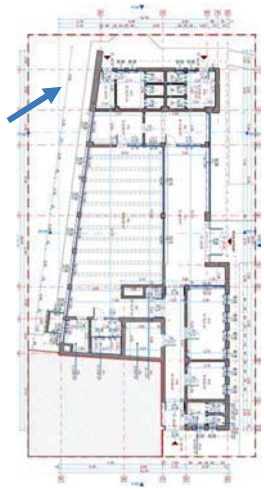

| *Photo no. 126 -* | Ground floor plan |
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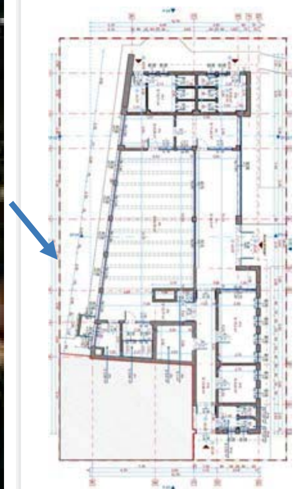

| *Photo no. 127 -* | Ground floor plan |
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*Photo no. 128 -* | Ground floor plan

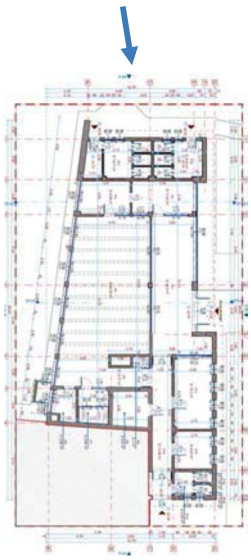
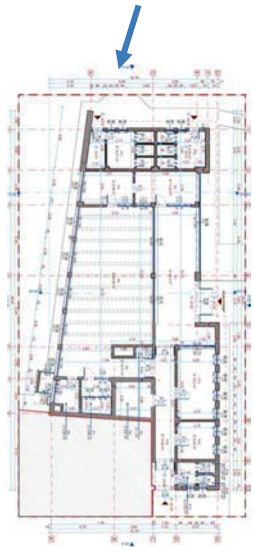



*Photo no. 129 -* | Ground floor plan



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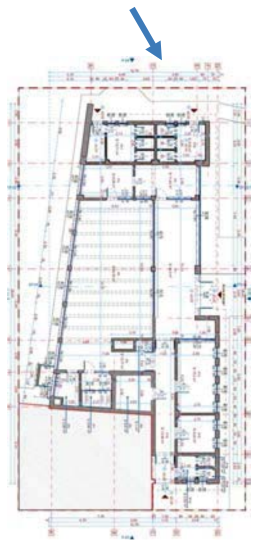
| *Photo no. 130 -* | Ground floor plan |



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| *Photo no. 131 -* | Ground floor plan |

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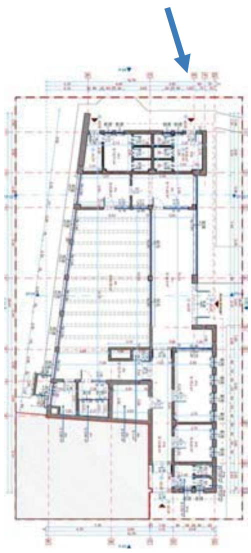
| *Photo no. 132 -* | Ground floor plan |



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| *Photo no. 133 -* | Ground floor plan |

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| *Photo no. 134 -* | Ground floor plan |

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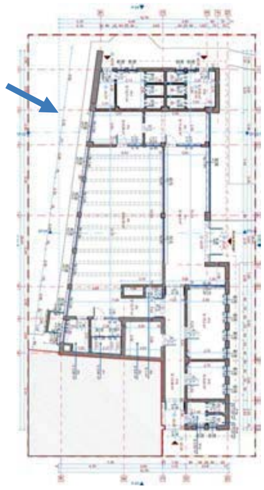
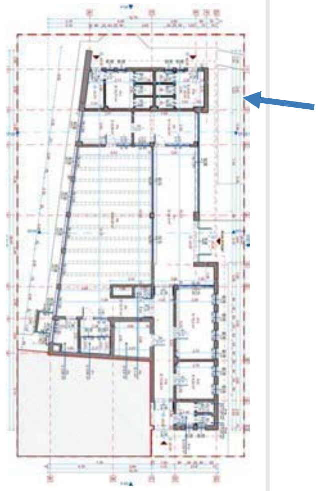
|  |  |
| *Photo no. 135 -* | Ground floor plan |

*Photo no. 136 -* Ground floor plan

*Photo no. 137 -* Ground floor plan



| *Photo no. 138 -* | Ground floor plan |
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| *Photo no. 139 -* | Ground floor plan |
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*Photo no. 140 -* Ground floor plan

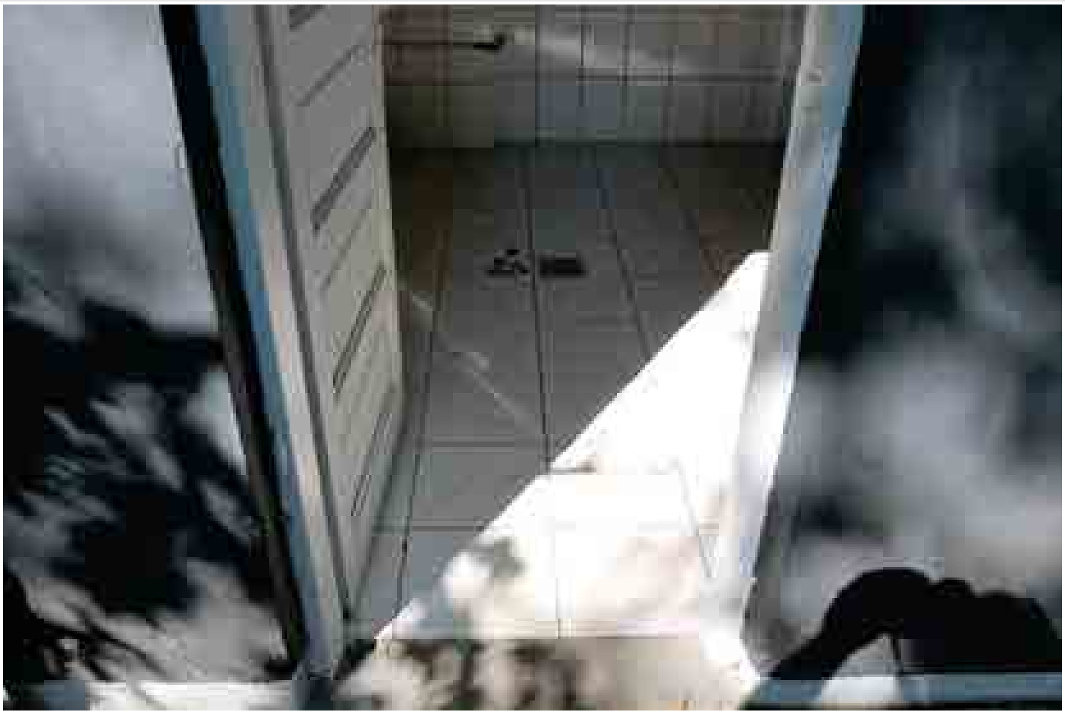

*Photo no. 141 -* Ground floor plan


| *Photo no. 142 -* | Ground floor plan |
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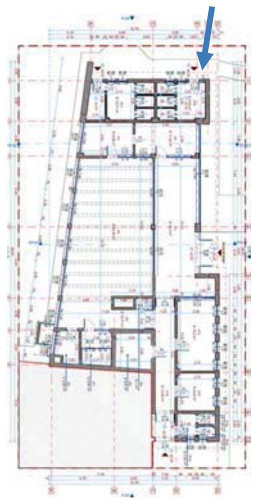

| *Photo no. 143 -* | Ground floor plan |
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| Photo no. 144 - | Ground floor plan |
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| Photo no. 145 - | Ground floor plan |
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| *Photo no. 146 -* | Ground floor plan |
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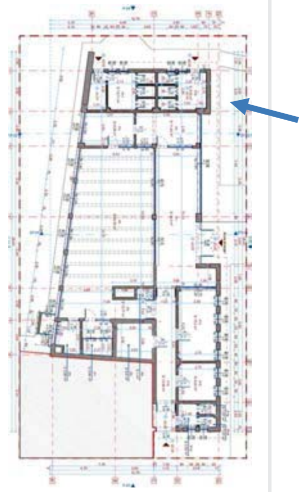

| *Photo no. 147 -* | Ground floor plan |
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| *Photo no. 148 -* | Ground floor plan |
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| *Photo no. 149 -* | Ground floor plan |
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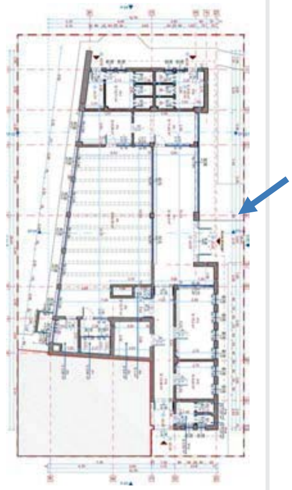
| *Photo no. 150 -* | Ground floor plan |

|  |  |
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| *Photo no. 151 -* | Ground floor plan |

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| *Photo no. 152 -* | Ground floor plan |

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| *Photo no. 153 -* | Ground floor plan |

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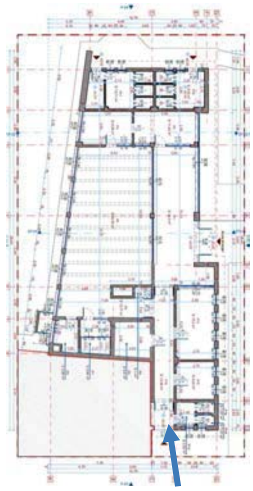
| *Photo no. 154 -* | Ground floor plan |



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| *Photo no. 155 -* | Ground floor plan |

| *Photo no. 156 -* | Ground floor plan |
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| *Photo no. 157 -* | Ground floor plan |
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| *Photo no. 158* - | Ground floor plan |

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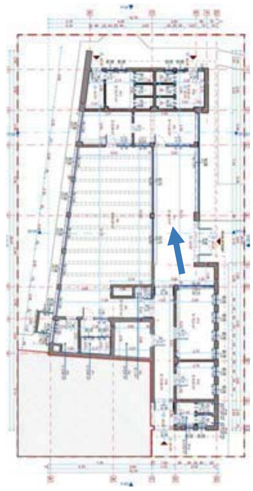
| *Photo no. 159* - | Ground floor plan |



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| *Photo no. 160 -* | Ground floor plan |

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| *Photo no. 161 -* | Ground floor plan |

| Photo no. 162 - | Ground floor plan |
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| Photo no. 163 - | Ground floor plan |
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| *Photo no. 164 -* | Ground floor plan |
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| *Photo no. 165 -* | Ground floor plan |
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| *Photo no. 166 -* | Ground floor plan |

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| *Photo no. 167 -* | Ground floor plan |

| *Photo no. 168 -* | Ground floor plan |
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| *Photo no. 169 -* | Ground floor plan |
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| *Photo no. 170 -* | Ground floor plan |

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| *Photo no. 171 -* | Ground floor plan |

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| *Photo no. 172 -* | Ground floor plan |

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| *Photo no. 173 -* | Ground floor plan |

| *Photo no. 174 -* | Ground floor plan |
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| *Photo no. 175 -* | Ground floor plan |
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| *Photo no. 176 -* | Ground floor plan |

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| *Photo no. 177 -* | Ground floor plan |

| *Photo no. 178 -* | Ground floor plan |
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| *Photo no. 179 -* | Ground floor plan |
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| *Photo no. 180 -* | Ground floor plan |

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| *Photo no. 181 -* | Ground floor plan |

| *Photo no. 182 -* | Ground floor plan |
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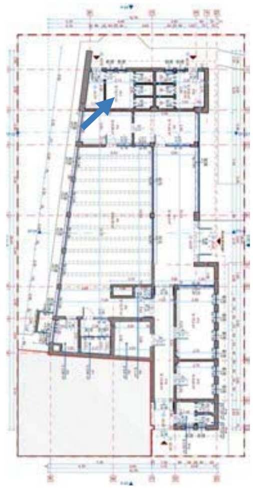

| *Photo no. 183 -* | Ground floor plan |
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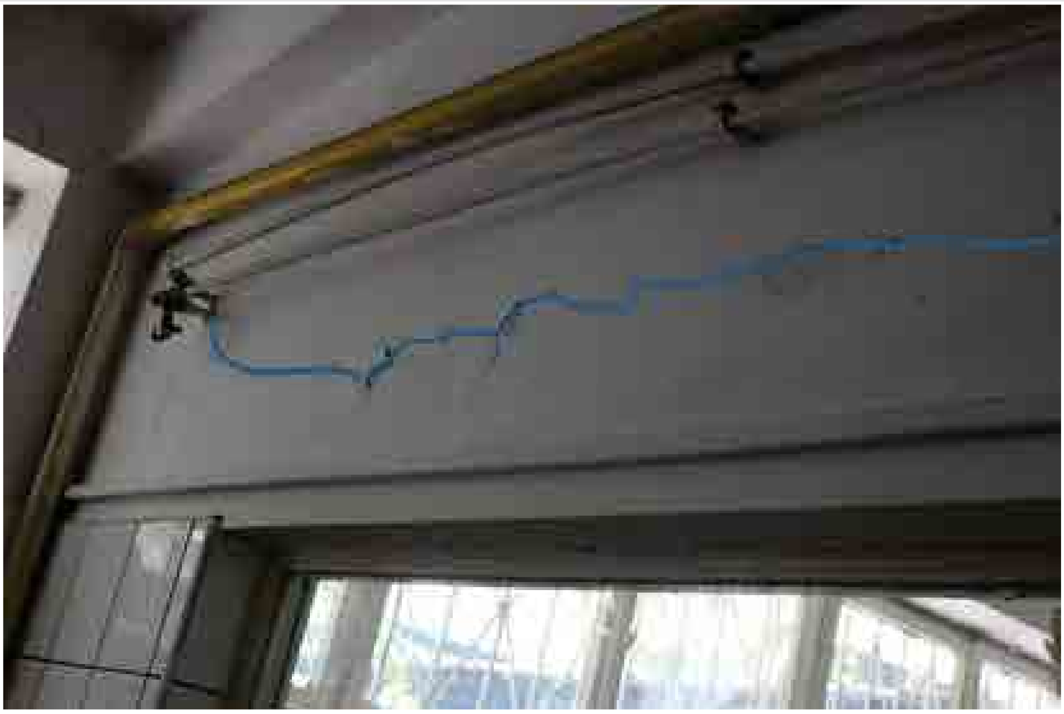
*Photo no. 184 -* | Ground floor plan

*Photo no. 185 -* | Ground floor plan

*Photo no. 186 -* | Ground floor plan

*Photo no. 187 -* | Ground floor plan



| *Photo no. 188 -* | Ground floor plan |
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| *Photo no. 189 -* | Ground floor plan |
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| *Photo no. 190 -* | Ground floor plan |
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| *Photo no. 191 -* | Ground floor plan |
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| Photo no. 192 - | Ground floor plan |
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| Photo no. 193 - | Ground floor plan |
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| *Photo no. 194 -* | Ground floor plan |
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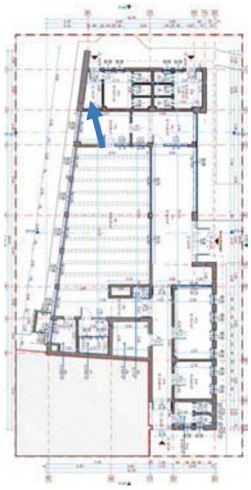

| *Photo no. 195 -* | Ground floor plan |
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*Photo no. 196 -* Ground floor plan

*Photo no. 197 -* Ground floor plan

*Photo no. 198* - | Ground floor plan

*Photo no. 199* - | Ground floor plan

*Photo no. 200 -* Ground floor plan

*Photo no. 201 -* Ground floor plan

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| *Photo no. 202 -* | Ground floor plan |

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| *Photo no. 203 -* | Ground floor plan |

| *Photo no. 204 -* | Ground floor plan |
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| *Photo no. 205 -* | Ground floor plan |
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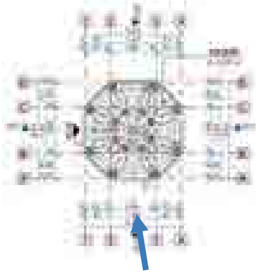
## Annex B.6 – Building C15

*Photo no. 206 -*

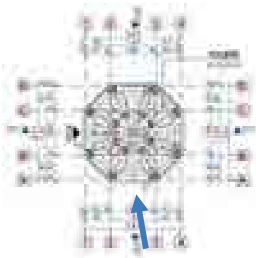
## Annex B.7 – Building C16

*Photo no. 207 -*



Ground floor plan

*Photo no. 208 -*



Ground floor plan

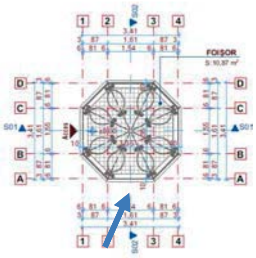


*Photo no. 209 -* Ground floor plan

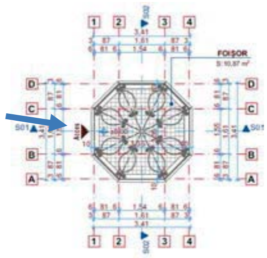
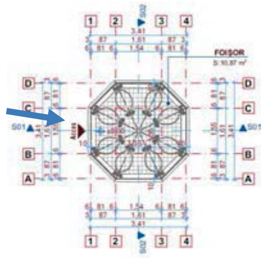


*Photo no. 210 -* Ground floor plan



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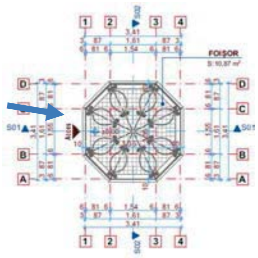
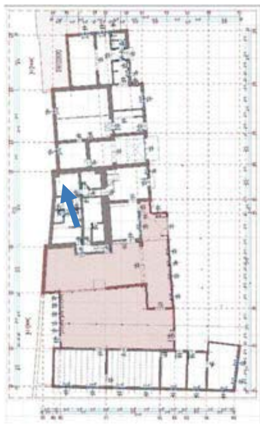
|---|---|
| *Photo no. 211 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 212 -* | Ground floor plan |

## Annex B.8 – Building C17

*Photo no. 213 -*



Ground floor plan

*Photo no. 214 -*

Ground floor plan

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| *Photo no. 215* - | Ground floor plan |

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|  |  |
| *Photo no. 216* - | Ground floor plan |

*Photo no. 217 -* Ground floor plan

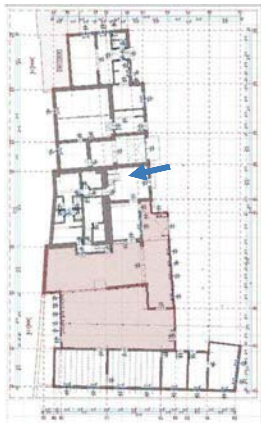

*Photo no. 218 -* Ground floor plan

| Photo no. 219 - | Ground floor plan |
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| Photo no. 220 - | Ground floor plan |
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| *Photo no. 221 -* | Ground floor plan |

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| *Photo no. 222 -* | Ground floor plan |

*Photo no. 223 -* Ground floor plan

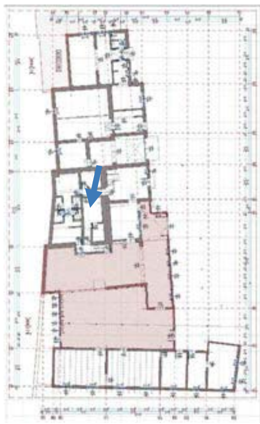

*Photo no. 224 -* Ground floor plan


*Photo no. 225 -* Ground floor plan


*Photo no. 226 -* Ground floor plan

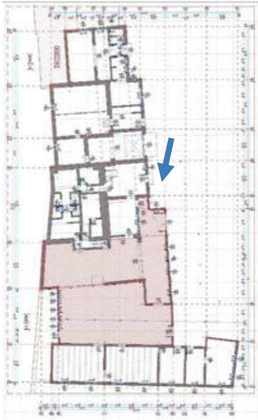

*Photo no. 227 -* Ground floor plan

*Photo no. 228 -* Ground floor plan

*Photo no. 229 -*

Ground floor plan

*Photo no. 230 -*

Ground floor plan

*Photo no. 231 -* | Ground floor plan

*Photo no. 232 -* | Ground floor plan

*Photo no. 233 -* Ground floor plan

*Photo no. 234 -* Ground floor plan

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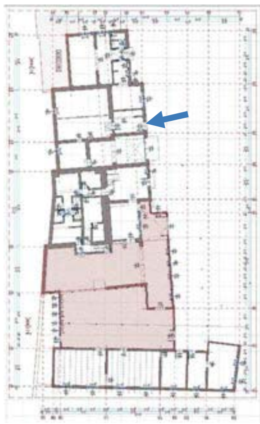
|---|---|
| *Photo no. 235 -* | Ground floor plan |



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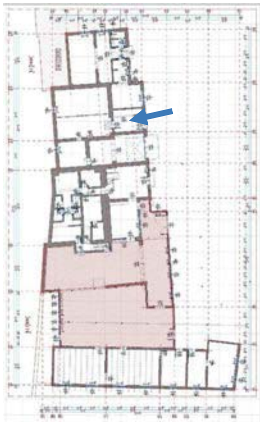
|---|---|
| *Photo no. 236 -* | Ground floor plan |



| *Photo no. 237 -* | Ground floor plan |
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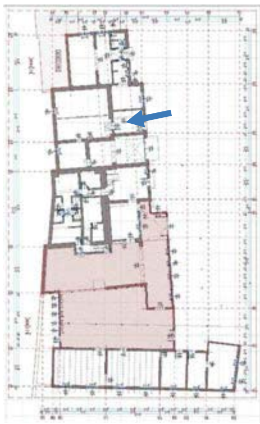

| *Photo no. 238 -* | Ground floor plan |
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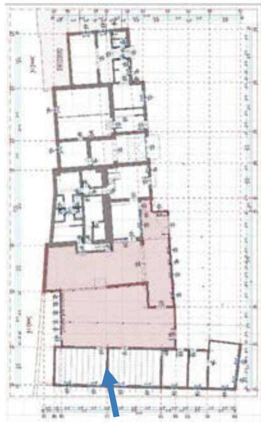
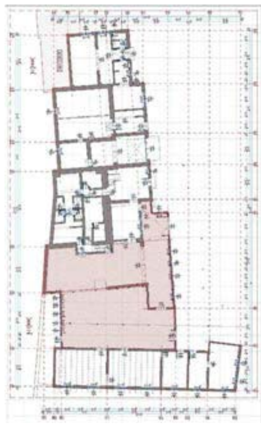


| *Photo no. 239 -* | Ground floor plan |
|---|---|



| *Photo no. 240 -* | Ground floor plan |
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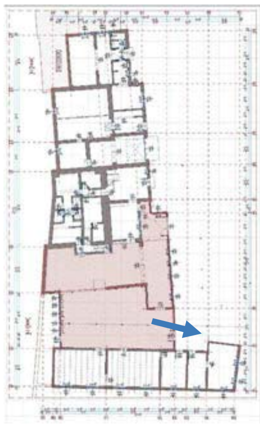
*Photo no. 241 -* Ground floor plan

*Photo no. 242 -* Ground floor plan

## Annex B.9 – Building C18



| *Photo no. 243 -* | Ground floor plan |
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| *Photo no. 244 -* | Ground floor plan |
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| *Photo no. 245 -* | Ground floor plan |
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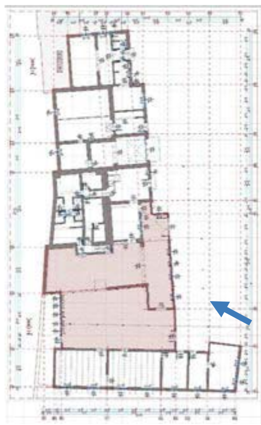


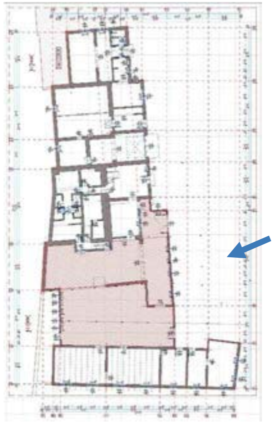
| *Photo no. 246 -* | Ground floor plan |
|---|---|

*Photo no. 247 -*

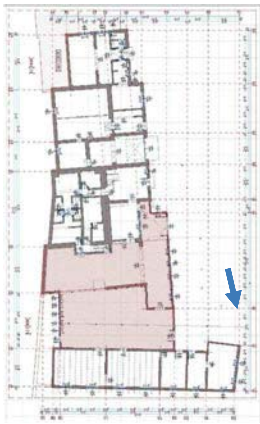
Ground floor plan

*Photo no. 248 -*

Ground floor plan



| *Photo no. 249 -* | Ground floor plan |
|---|---|



| *Photo no. 250 -* | Ground floor plan |
|---|---|


*Photo no. 251 -* Ground floor plan


*Photo no. 252 -* Ground floor plan

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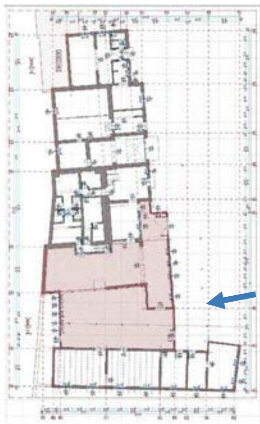
|---|---|
| *Photo no. 253 -* | Ground floor plan |

|  |  |
|---|---|
| *Photo no. 254 -* | Ground floor plan |

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|  |  |
| *Photo no. 255 -* | Ground floor plan |

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|  |  |
| *Photo no. 256 -* | Ground floor plan |

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|  |  |
| *Photo no. 257 -* | Ground floor plan |

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|  |  |
| *Photo no. 258 -* | Ground floor plan |


| *Photo no. 259 -* | Ground floor plan |
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| *Photo no. 260 -* | Ground floor plan |
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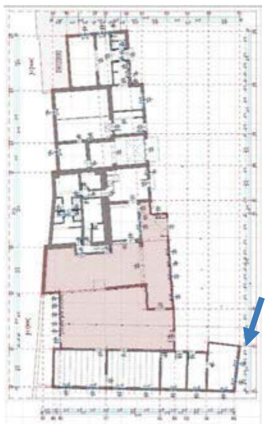
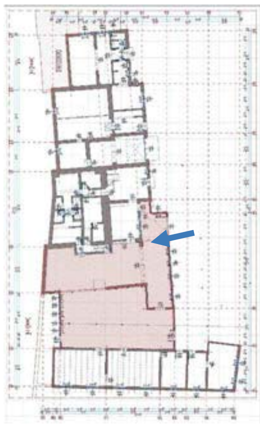
| | |
|---|---|
| *Photo no. 261 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 262 -* | Ground floor plan |

## Annex B.10 – Building C20

*Photo no. 263 -*



Ground floor plan

*Photo no. 264 -*

Ground floor plan

| *Photo no. 265 -* | Ground floor plan |
|---|---|

| *Photo no. 266 -* | Ground floor plan |
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*Photo no. 267 -* Ground floor plan

*Photo no. 268 -* Ground floor plan


*Photo no. 269 -* Ground floor plan


*Photo no. 270 -* Ground floor plan

| *Photo no. 271 -* | Ground floor plan |
|---|---|

| *Photo no. 272 -* | Ground floor plan |
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| *Photo no. 273 -* | Ground floor plan |
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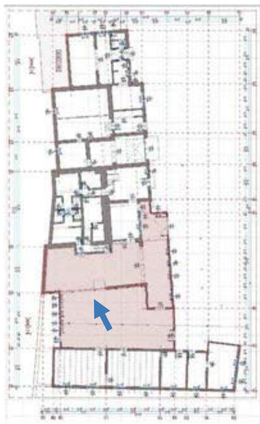


| *Photo no. 274 -* | Ground floor plan |
|---|---|

*Photo no. 275 -* Ground floor plan

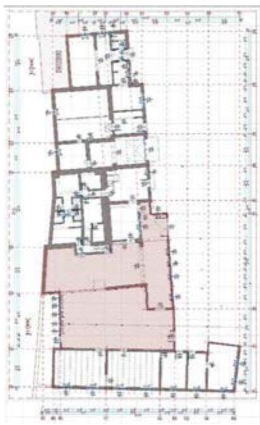


*Photo no. 276 -* Ground floor plan

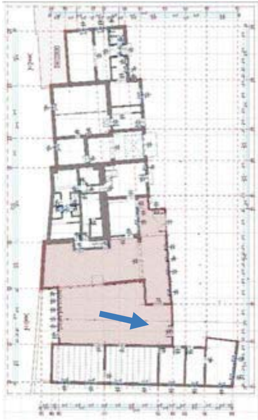


*Photo no. 277 -* Ground floor plan

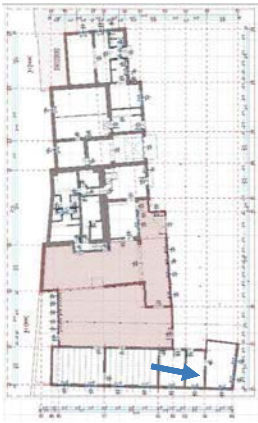


*Photo no. 278 -* Ground floor plan

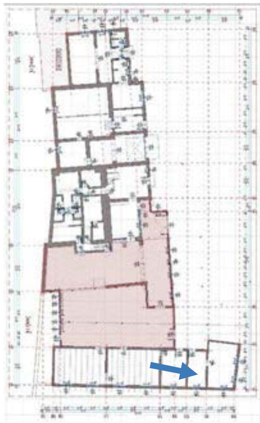
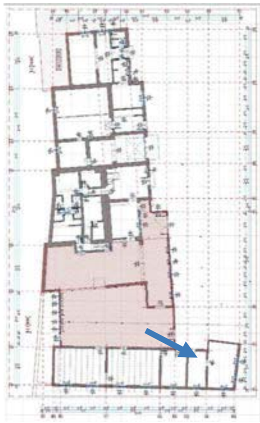


*Photo no. 279 -* Ground floor plan

*Photo no. 280 -* Ground floor plan

| *Photo no. 281 -* | Ground floor plan |
|---|---|



| *Photo no. 282 -* | Ground floor plan |
|---|---|

| *Photo no. 283 -* | Ground floor plan |
|---|---|

| *Photo no. 284 -* | Ground floor plan |
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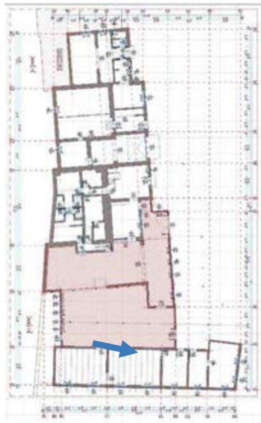


*Photo no. 285 -*

Ground floor plan

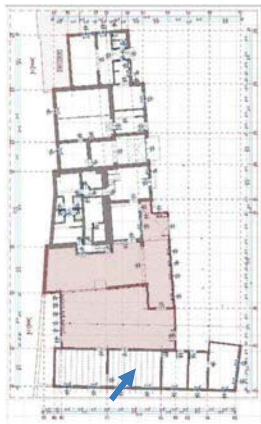
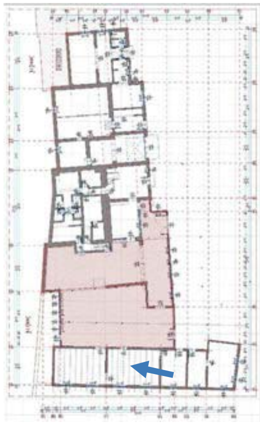
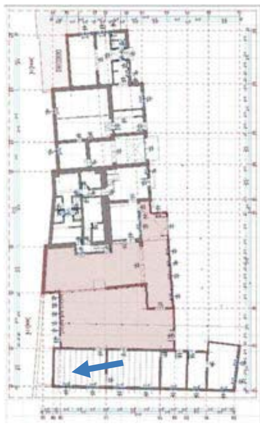


*Photo no. 286 -*

Ground floor plan

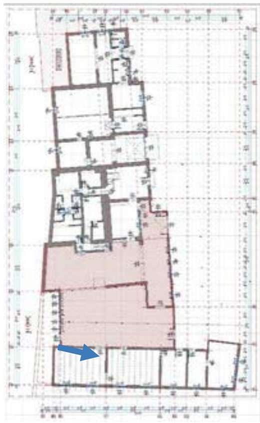


| | |
|---|---|
| *Photo no. 287 -* | Ground floor plan |

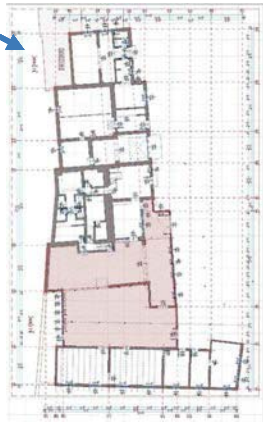


| | |
|---|---|
| *Photo no. 288 -* | Ground floor plan |

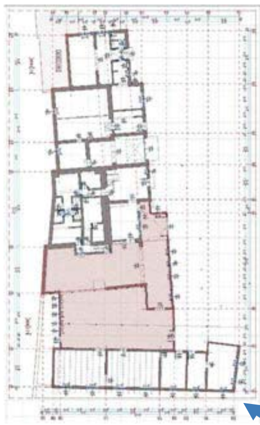

| *Photo no. 289 -* | Ground floor plan |
|---|---|

| *Photo no. 290 -* | Ground floor plan |
|---|---|



| *Photo no. 291 -* | Ground floor plan |
|---|---|



| *Photo no. 292 -* | Ground floor plan |
|---|---|

*Photo no. 293 -*

Ground floor plan

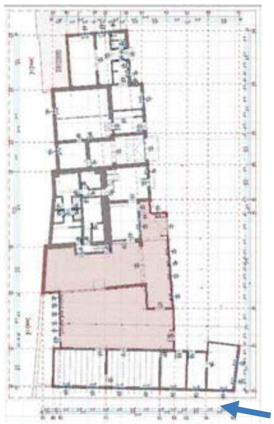
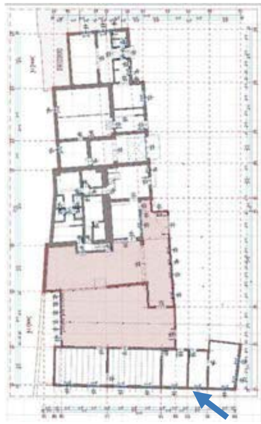
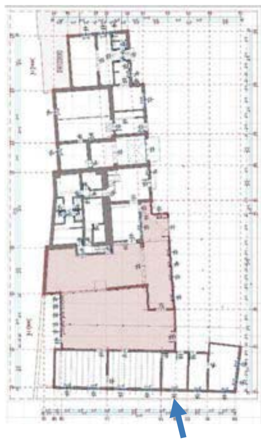


*Photo no. 294 -*

Ground floor plan



| Photo no. 295 - | Ground floor plan |
|---|---|



| Photo no. 296 - | Ground floor plan |
|---|---|

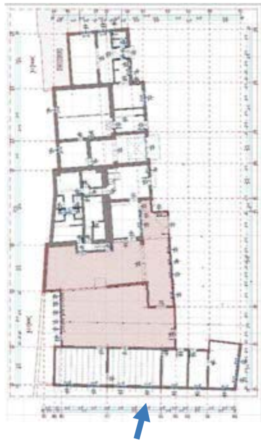
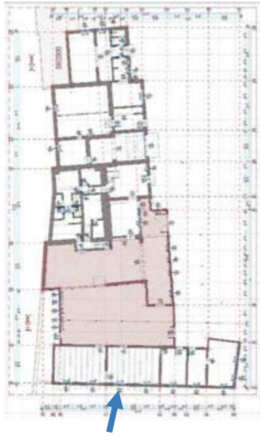
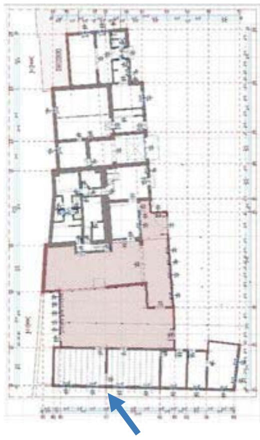


*Photo no. 297 -* Ground floor plan

*Photo no. 298 -* Ground floor plan



| *Photo no. 299 -* | Ground floor plan |
|---|---|



| *Photo no. 300 -* | Ground floor plan |
|---|---|

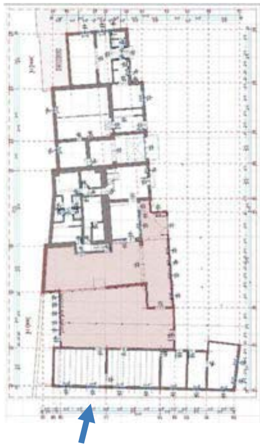


*Photo no. 301 -* Ground floor plan

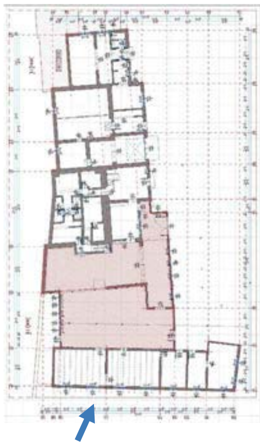


*Photo no. 302 -* Ground floor plan

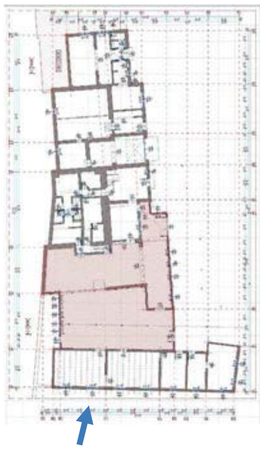


*Photo no. 303 -* | Ground floor plan

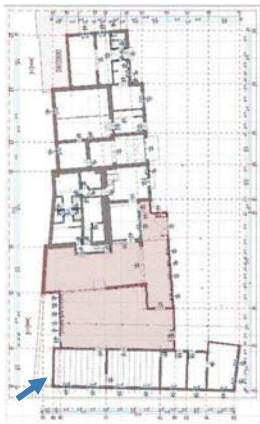
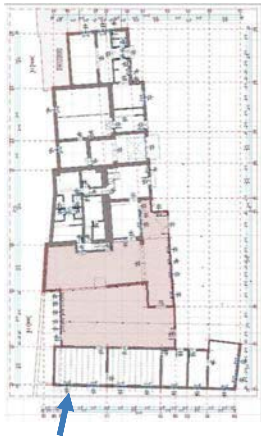
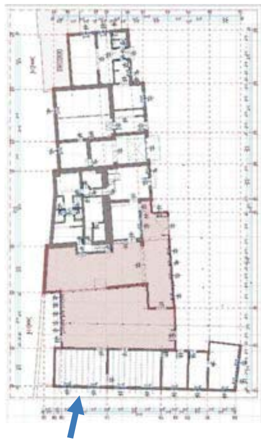


*Photo no. 304 -* | Ground floor plan



| *Photo no. 305 -* | Ground floor plan |
|---|---|



| *Photo no. 306 -* | Ground floor plan |
|---|---|

*Photo no. 307* -

Ground floor plan

*Photo no. 308* -

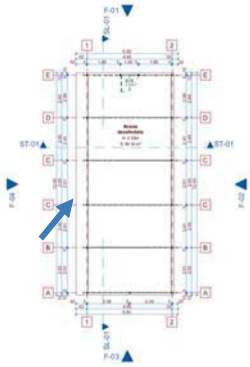
Ground floor plan

| *Photo no. 309 -* | Ground floor plan |
|---|---|

| *Photo no. 310 -* | Ground floor plan |
|---|---|

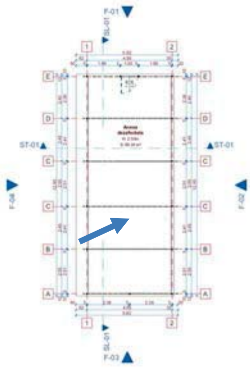
## Annex B.11 – Building C22

*Photo no. 311 -*

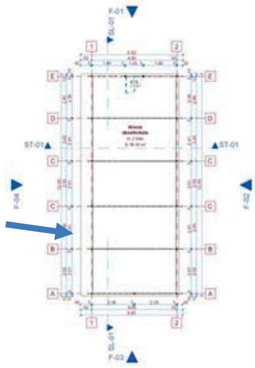


Ground floor plan

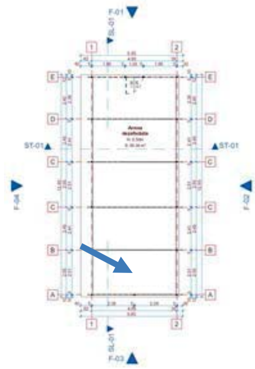
*Photo no. 312 -*



Ground floor plan

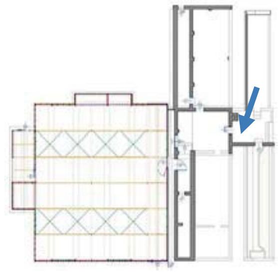


| *Photo no. 313 -* | Ground floor plan |
|---|---|

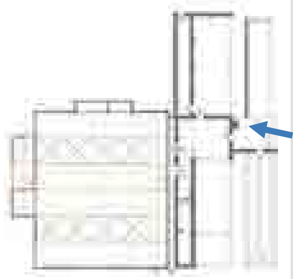


| *Photo no. 314 -* | Ground floor plan |
|---|---|

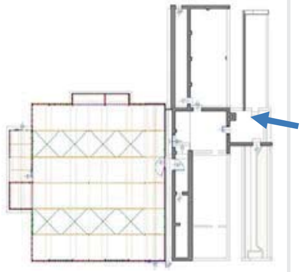
## Annex B.12 – Building C24



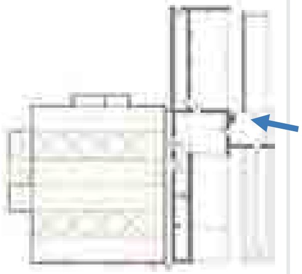
| *Photo no. 315 -* | Ground floor plan |
|---|---|



| *Photo no. 316 -* | Ground floor plan |
|---|---|



| | |
|---|---|
| *Photo no. 317 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 318 -* | Ground floor plan |

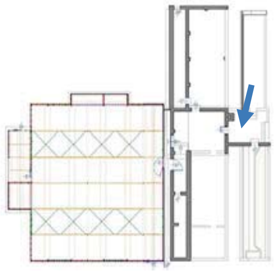


*Photo no. 319 -* | Ground floor plan

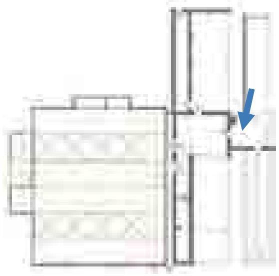


*Photo no. 320 -* | Ground floor plan

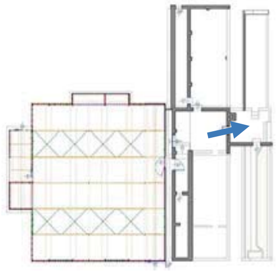


*Photo no. 321 -* | Ground floor plan

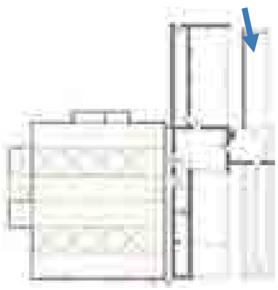


*Photo no. 322 -* | Ground floor plan

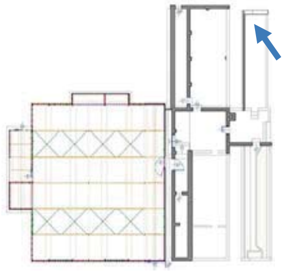
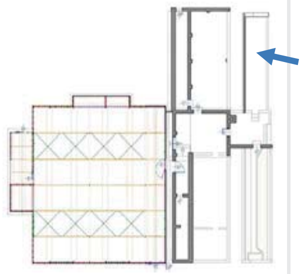
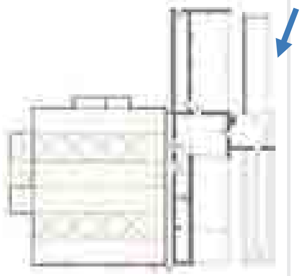


*Photo no. 323 -* | Ground floor plan

*Photo no. 324 -* | Ground floor plan



| *Photo no. 325 -* | Ground floor plan |
|---|---|



| *Photo no. 326 -* | Ground floor plan |
|---|---|

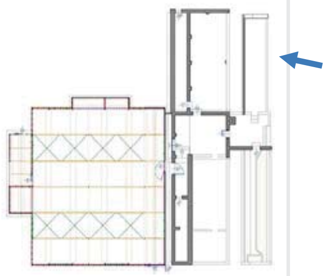



*Photo no. 327 -* Ground floor plan

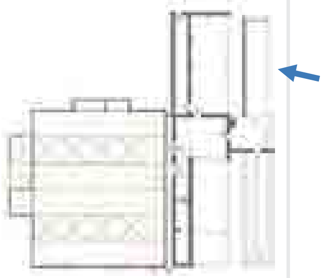
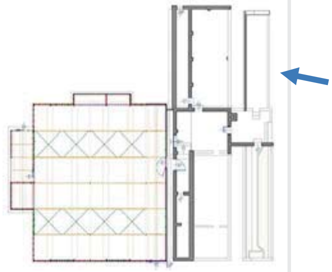
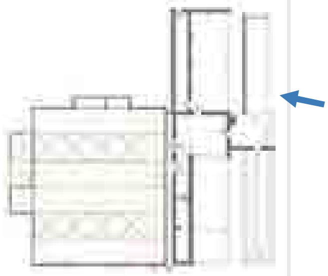



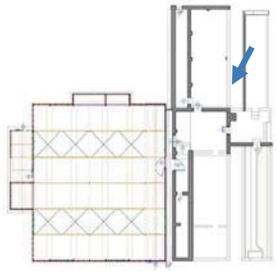
*Photo no. 328 -* Ground floor plan



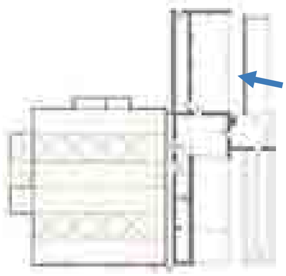
| | |
|---|---|
| *Photo no. 329 -* | Ground floor plan |



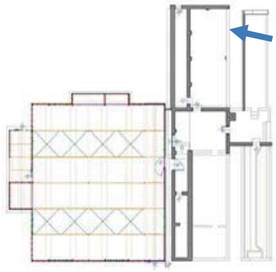
| | |
|---|---|
| *Photo no. 330 -* | Ground floor plan |



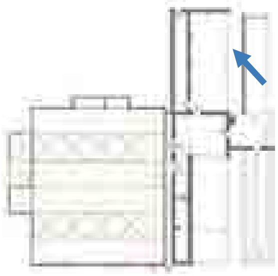
| *Photo no. 331 -* | Ground floor plan |
|---|---|



| *Photo no. 332 -* | Ground floor plan |
|---|---|



| *Photo no. 333 -* | Ground floor plan |
|---|---|



| *Photo no. 334 -* | Ground floor plan |
|---|---|

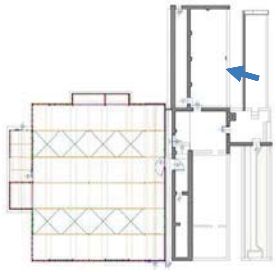
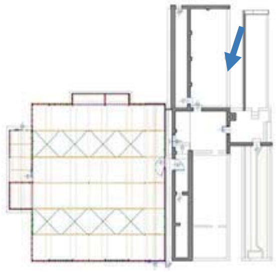
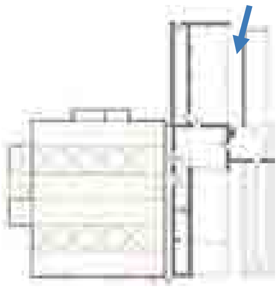


*Photo no. 335 -* Ground floor plan

*Photo no. 336 -* Ground floor plan



| *Photo no. 337 -* | Ground floor plan |
|---|---|



| *Photo no. 338 -* | Ground floor plan |
|---|---|

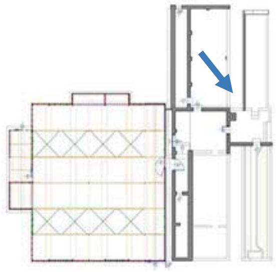


*Photo no. 339 -*

Ground floor plan

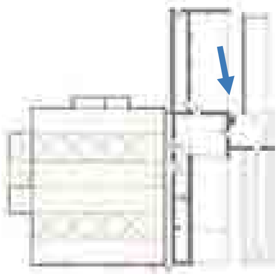
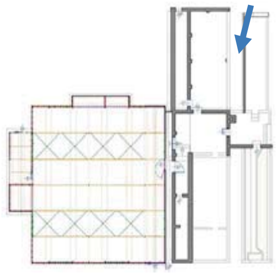


*Photo no. 340 -*

Ground floor plan



| *Photo no. 341 -* | Ground floor plan |
|---|---|

| *Photo no. 342 -* | Ground floor plan |
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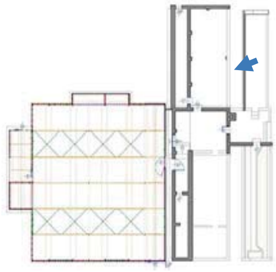


*Photo no. 343 -* | Ground floor plan

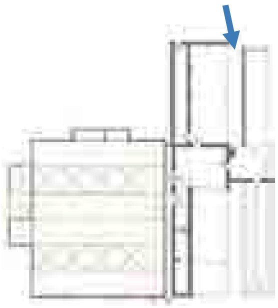
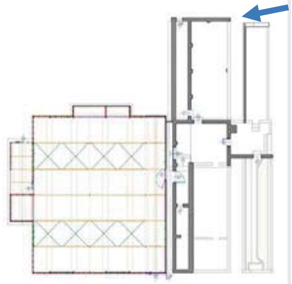


*Photo no. 344 -* | Ground floor plan

| | |
|---|---|
|  |  |
| *Photo no. 345 -* | Ground floor plan |

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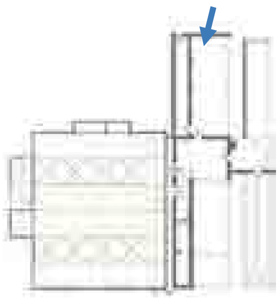
|---|---|
|  |  |
| *Photo no. 346 -* | Ground floor plan |

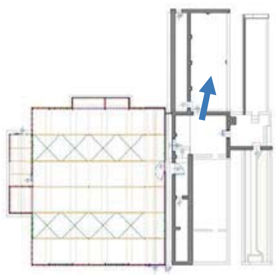


*Photo no. 347 -* Ground floor plan

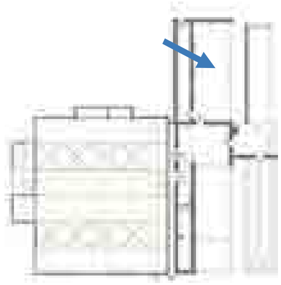


*Photo no. 348 -* Ground floor plan

## Annex B.13 – Building C25

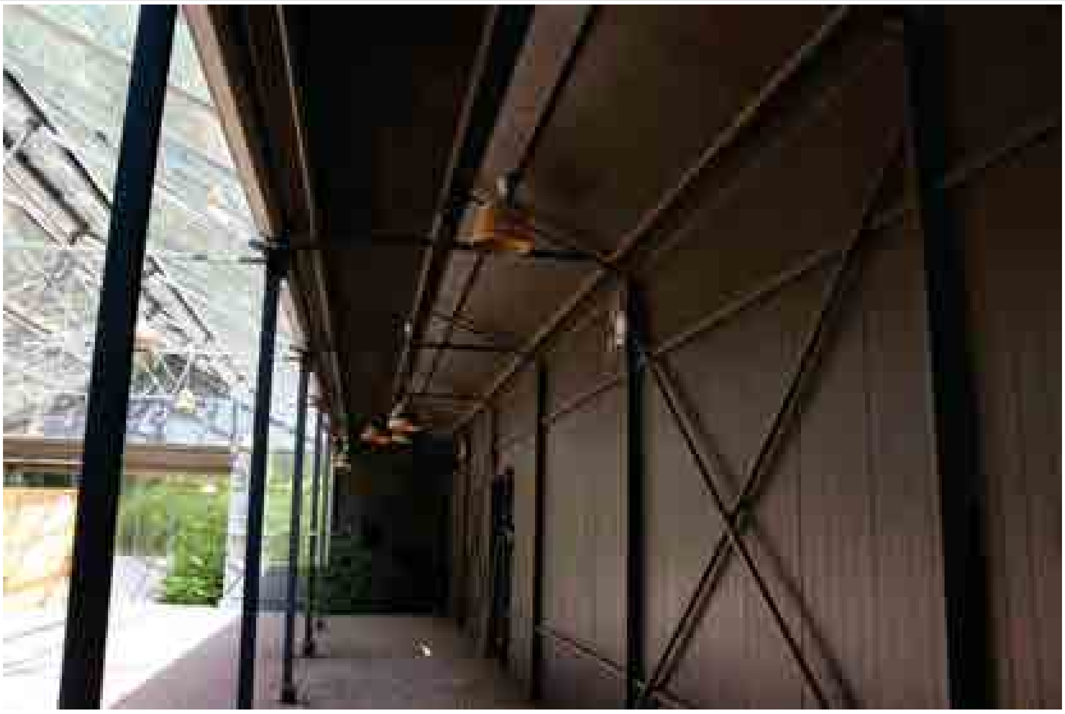
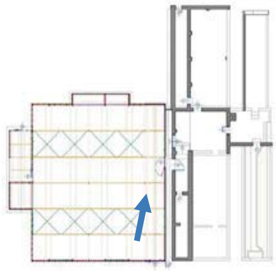
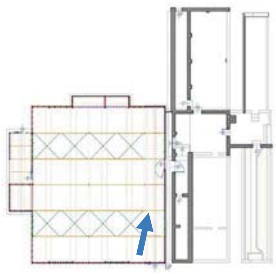


*Photo no. 349 -*

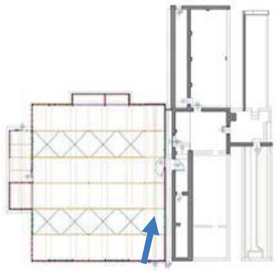


Ground floor plan

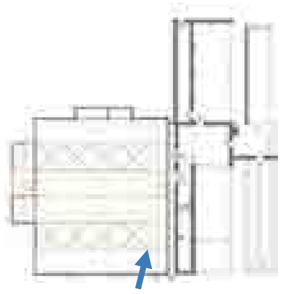
*Photo no. 350 -*



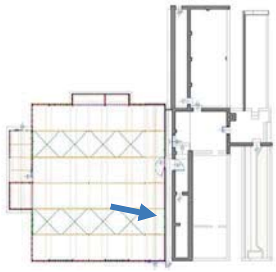
Ground floor plan

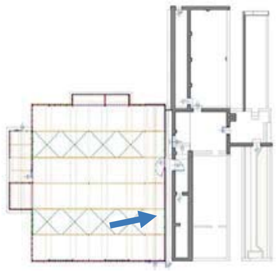
| *Photo no. 351 -* | Ground floor plan |
|---|---|

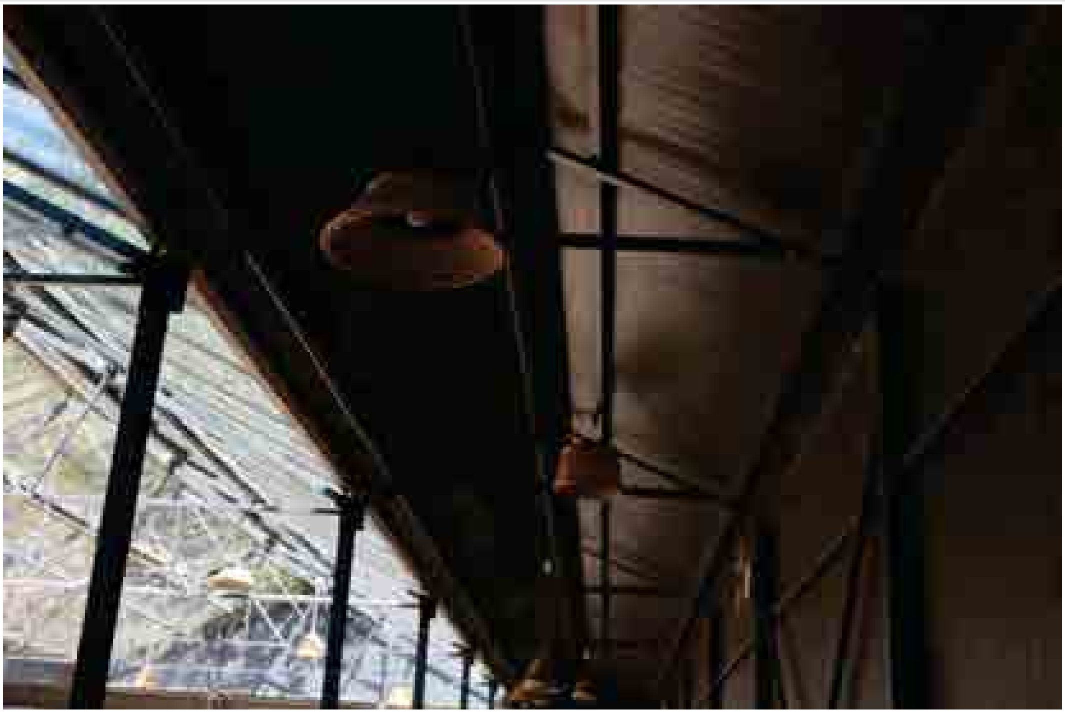
| *Photo no. 352 -* | Ground floor plan |
|---|---|



| | |
|---|---|
| *Photo no. 353 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 354 -* | Ground floor plan |



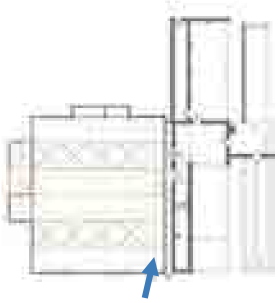


*Photo no. 355 -* Ground floor plan

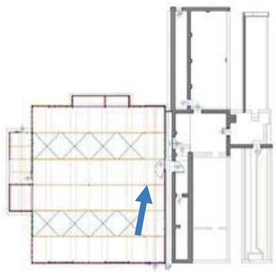


*Photo no. 356 -* Ground floor plan

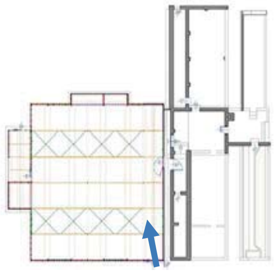


*Photo no. 357 -* Ground floor plan

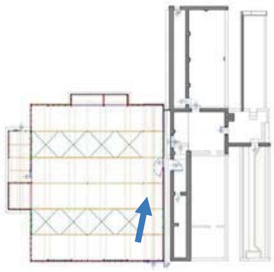
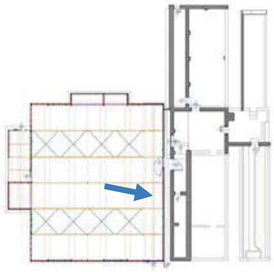
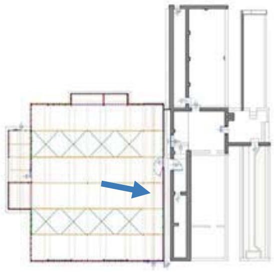


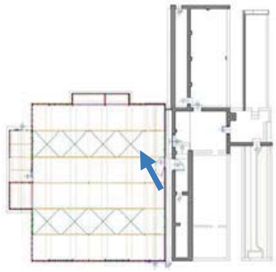
*Photo no. 358 -* Ground floor plan



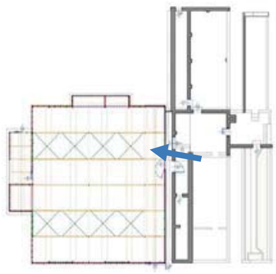
| *Photo no. 359 -* | Ground floor plan |
|---|---|



| *Photo no. 360 -* | Ground floor plan |
|---|---|



| *Photo no. 361 -* | Ground floor plan |
|---|---|



| *Photo no. 362 -* | Ground floor plan |
|---|---|

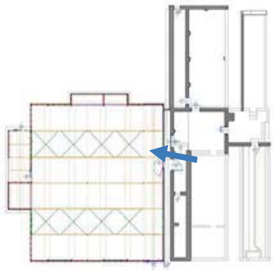


*Photo no. 363 -* Ground floor plan

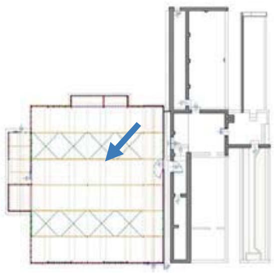
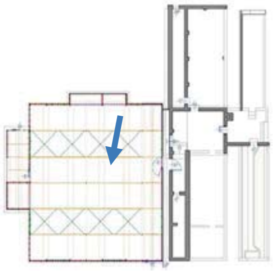
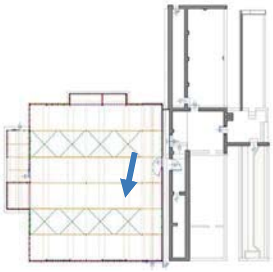


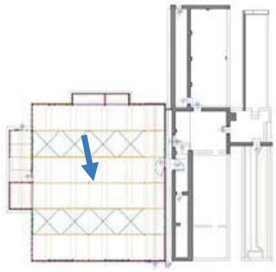
*Photo no. 364 -* Ground floor plan



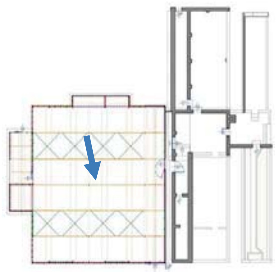
| *Photo no. 365 -* | Ground floor plan |
|---|---|



| *Photo no. 366 -* | Ground floor plan |
|---|---|



| *Photo no. 367 -* | Ground floor plan |
|---|---|



| *Photo no. 368 -* | Ground floor plan |
|---|---|

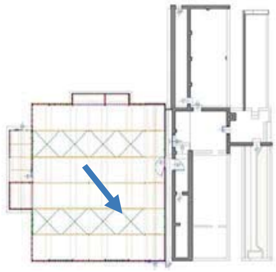


*Photo no. 369 -*

Ground floor plan

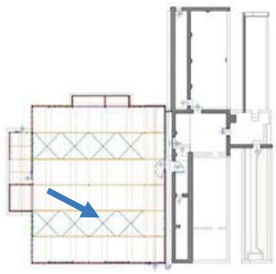


*Photo no. 370 -*

Ground floor plan

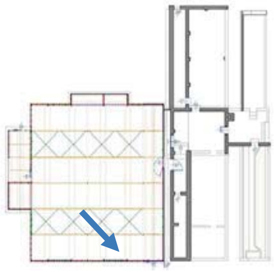


*Photo no. 371 -*

Ground floor plan

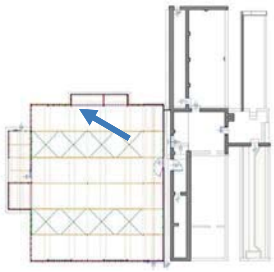
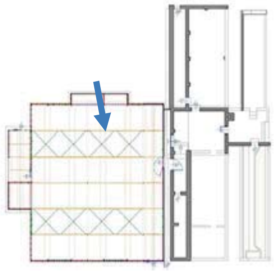


*Photo no. 372 -*

Ground floor plan

| | |
|---|---|
|  |  |
| *Photo no. 373 -* | Ground floor plan |

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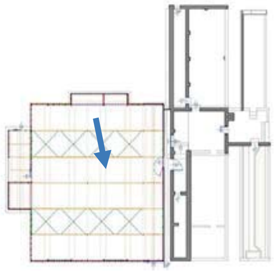
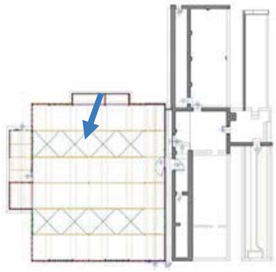
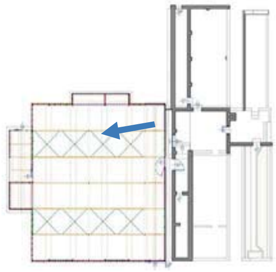
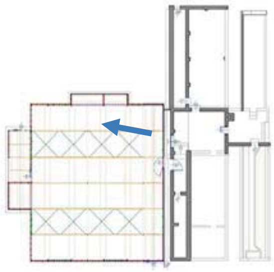
|---|---|
|  |  |
| *Photo no. 374 -* | Ground floor plan |

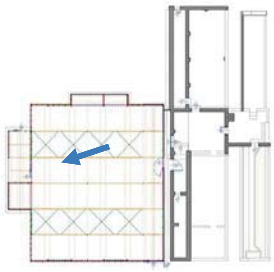
| *Photo no. 375 -* | Ground floor plan |
|---|---|

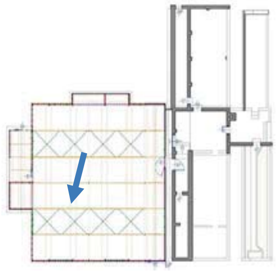
| *Photo no. 376 -* | Ground floor plan |
|---|---|



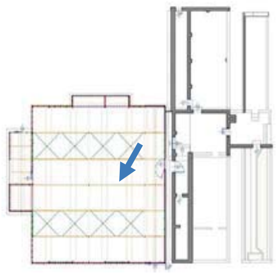
| | |
|---|---|
| *Photo no. 377 -* | Ground floor plan |



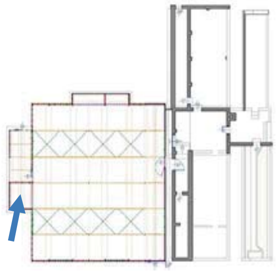
| | |
|---|---|
| *Photo no. 378 -* | Ground floor plan |



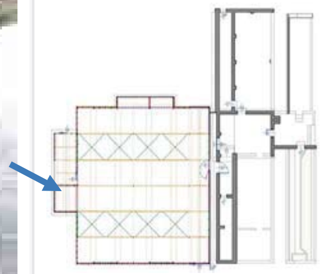
| *Photo no. 379 -* | Ground floor plan |
|---|---|



| *Photo no. 380 -* | Ground floor plan |
|---|---|



| *Photo no. 381 -* | Ground floor plan |
|---|---|



| *Photo no. 382 -* | Ground floor plan |
|---|---|

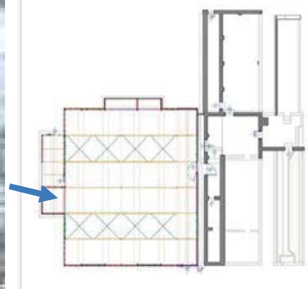



*Photo no. 383 -* Ground floor plan

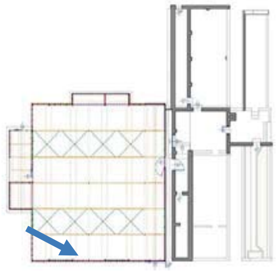



*Photo no. 384 -* Ground floor plan

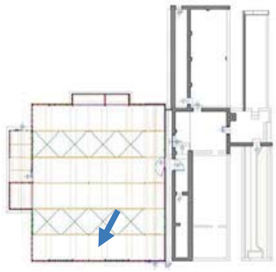


*Photo no. 385 -* Ground floor plan

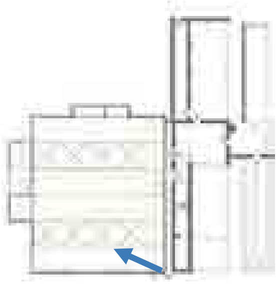


*Photo no. 386 -* Ground floor plan

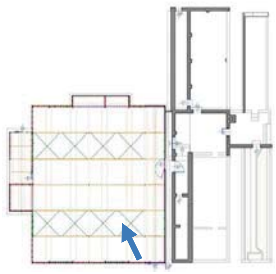


*Photo no. 387 -* Ground floor plan

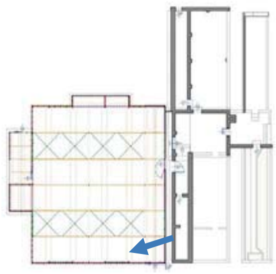


*Photo no. 388 -* Ground floor plan

*Photo no. 389 -* Ground floor plan

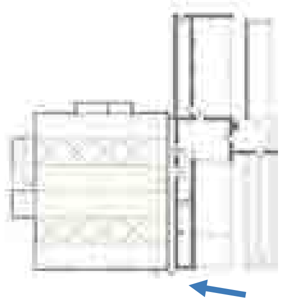
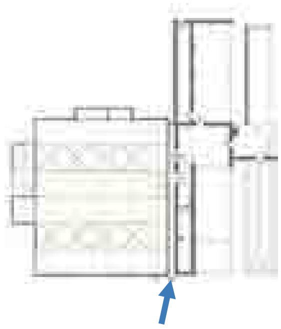
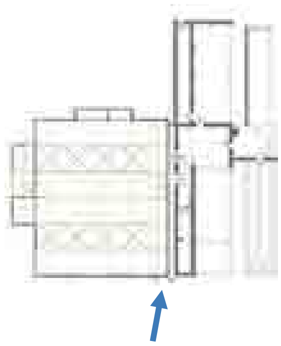
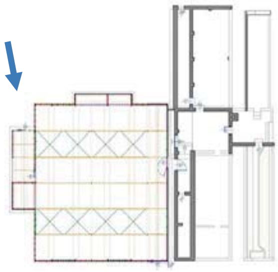


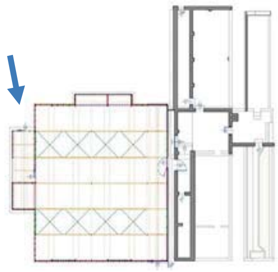
*Photo no. 390 -* Ground floor plan

| | |
|---|---|
|  |  |
| *Photo no. 391 -* | Ground floor plan |

| | |
|---|---|
|  |  |
| *Photo no. 392 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 393 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 394 -* | Ground floor plan |

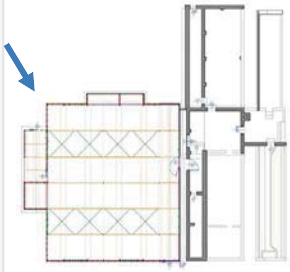


*Photo no. 395 -* Ground floor plan

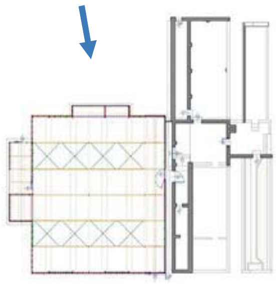


*Photo no. 396 -* Ground floor plan

## Annex B.14 – Building C26

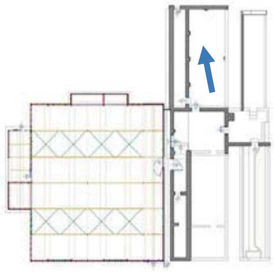


*Photo no. 397 -* Ground floor plan

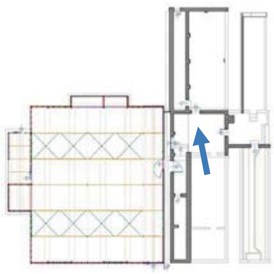
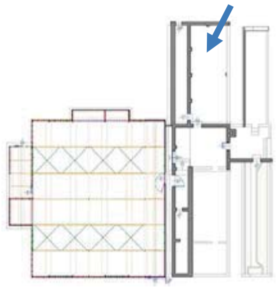
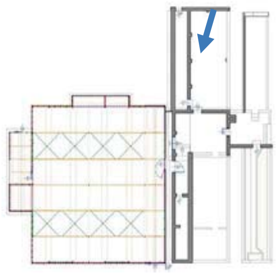


*Photo no. 398 -* Ground floor plan



| *Photo no. 399 -* | Ground floor plan |
|---|---|



| *Photo no. 400 -* | Ground floor plan |
|---|---|

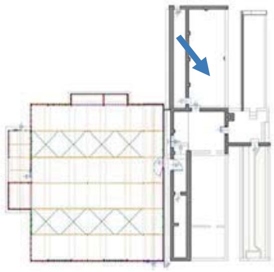


*Photo no. 401 -*

Ground floor plan

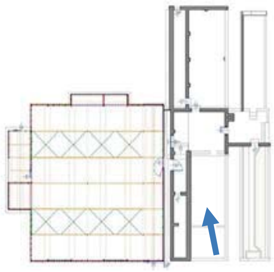


*Photo no. 402 -*

Ground floor plan

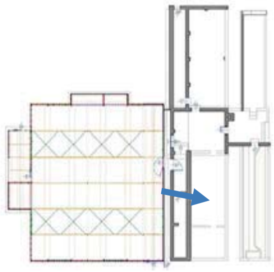


*Photo no. 403 -* Ground floor plan

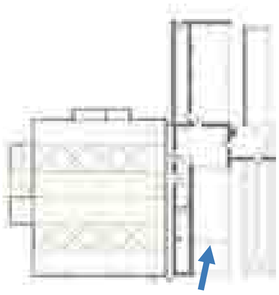
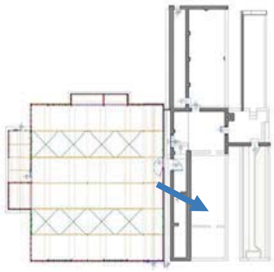
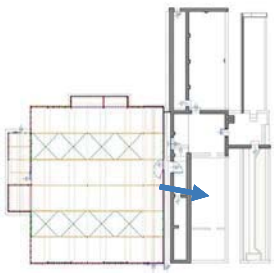


*Photo no. 404 -* Ground floor plan



| | |
|---|---|
| *Photo no. 405 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 406 -* | Ground floor plan |

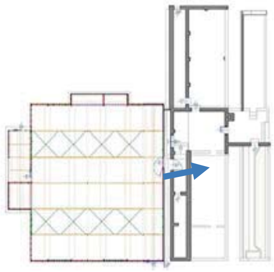


*Photo no. 407 -* Ground floor plan

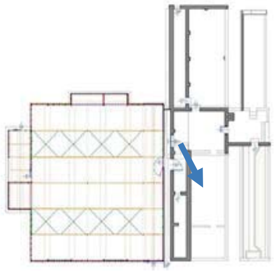
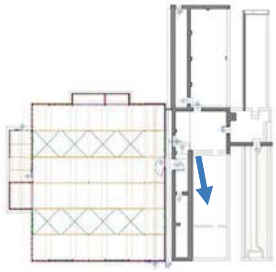
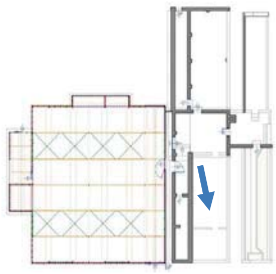


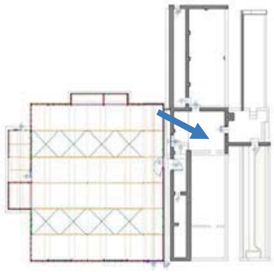
*Photo no. 408 -* Ground floor plan



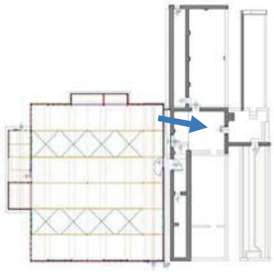
| | |
|---|---|
| *Photo no. 409 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 410 -* | Ground floor plan |



| *Photo no. 411 -* | Ground floor plan |
|---|---|



| *Photo no. 412 -* | Ground floor plan |
|---|---|

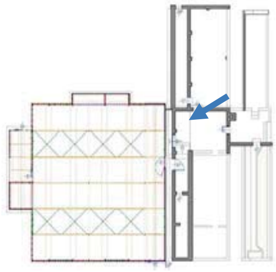


*Photo no. 413 -*

Ground floor plan

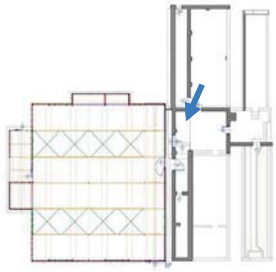


*Photo no. 414 -*

Ground floor plan

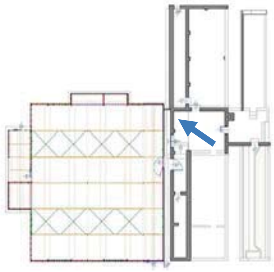


*Photo no. 415 -*

Ground floor plan

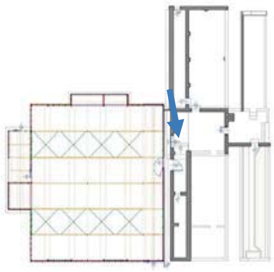
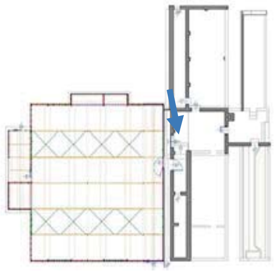
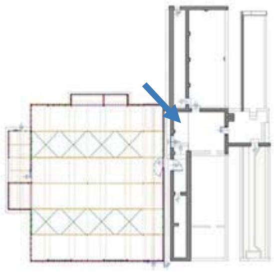


*Photo no. 416 -*

Ground floor plan



| *Photo no. 417 -* | Ground floor plan |
|---|---|



| *Photo no. 418 -* | Ground floor plan |
|---|---|

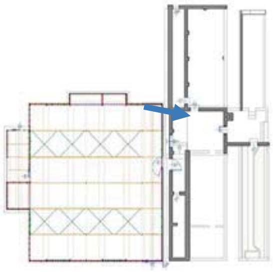


*Photo no. 419 -* Ground floor plan

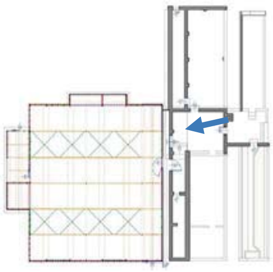
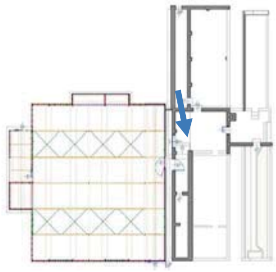
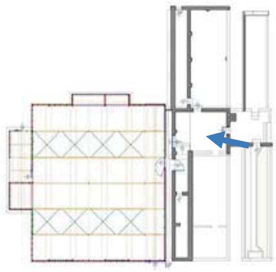


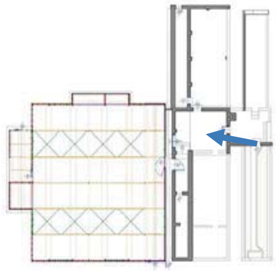
*Photo no. 420 -* Ground floor plan



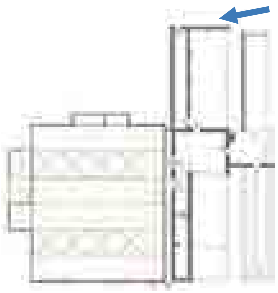
| | |
|---|---|
| *Photo no. 421 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 422 -* | Ground floor plan |



| *Photo no. 423 -* | Ground floor plan |
|---|---|



| *Photo no. 424 -* | Ground floor plan |
|---|---|

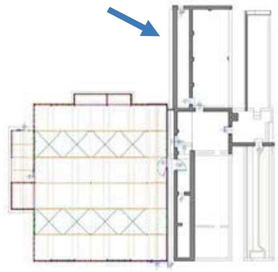



*Photo no. 425 -* | Ground floor plan

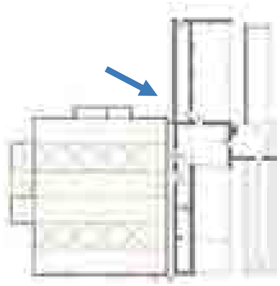



*Photo no. 426 -* | Ground floor plan

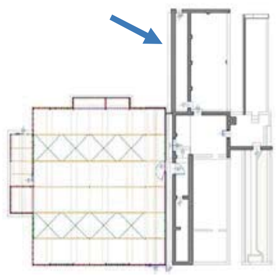


*Photo no. 427 -* Ground floor plan

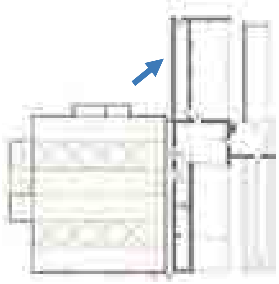


*Photo no. 428 -* Ground floor plan

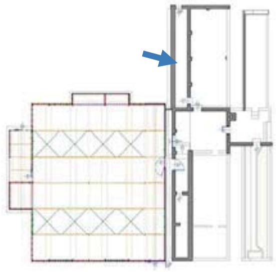


*Photo no. 429 -*

Ground floor plan

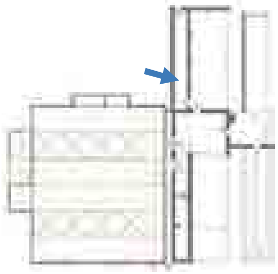
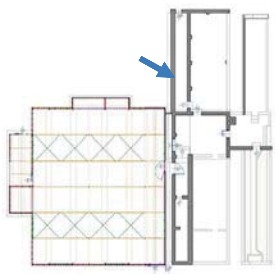
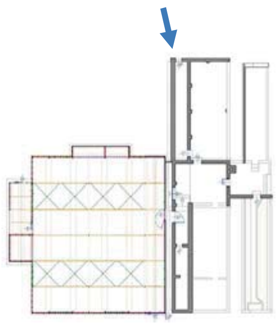


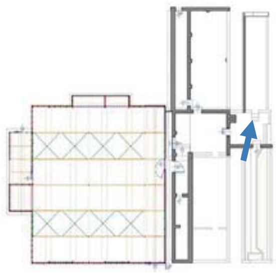
*Photo no. 430 -*

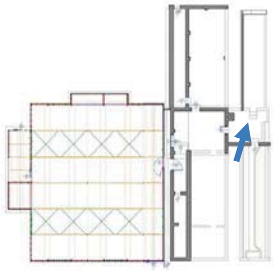
Ground floor plan

| | |
|---|---|
|  |  |
| *Photo no. 431 -* | Ground floor plan |

| | |
|---|---|
|  |  |
| *Photo no. 432 -* | Ground floor plan |

| | |
|---|---|
|  |  |
| *Photo no. 433 -* | Ground floor plan |

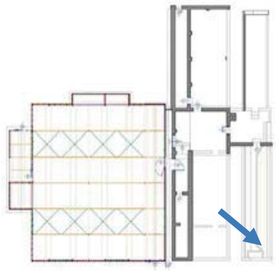
| | |
|---|---|
|  |  |
| *Photo no. 434 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 435* - | Ground floor plan |

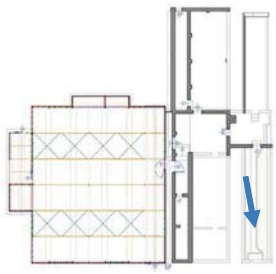
## Annex B.15 – Building C27

*Photo no. 436 -*

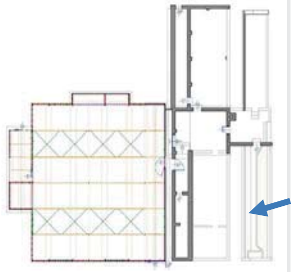


Ground floor plan

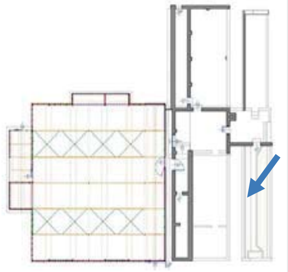
*Photo no. 437 -*



Ground floor plan



| *Photo no. 438* - | Ground floor plan |
|---|---|



| *Photo no. 439* - | Ground floor plan |
|---|---|

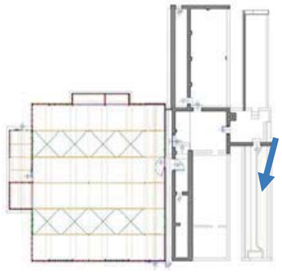


*Photo no. 440 -*

Ground floor plan

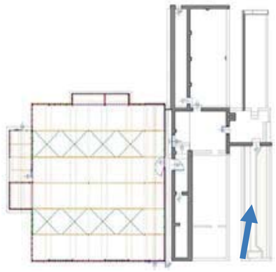
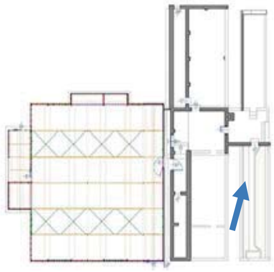


*Photo no. 441 -*

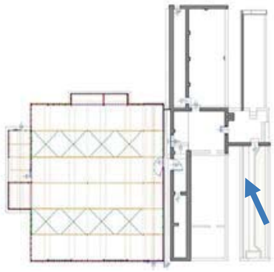
Ground floor plan

*Photo no. 442 -*



Ground floor plan

*Photo no. 443 -*



Ground floor plan

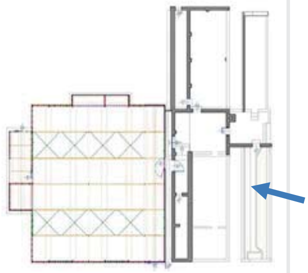


*Photo no. 444 -* Ground floor plan

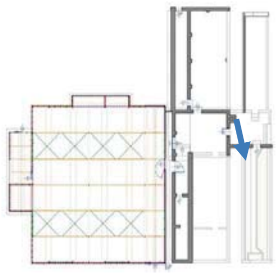
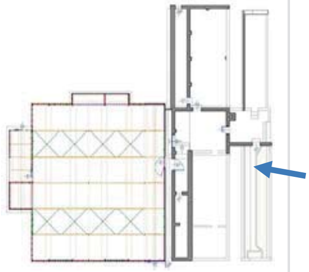
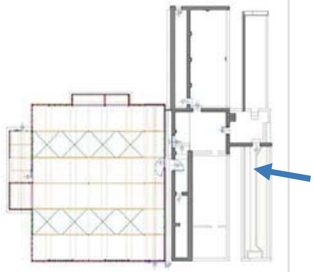


*Photo no. 445 -* Ground floor plan



| *Photo no. 446 -* | Ground floor plan |
|---|---|



| *Photo no. 447 -* | Ground floor plan |
|---|---|

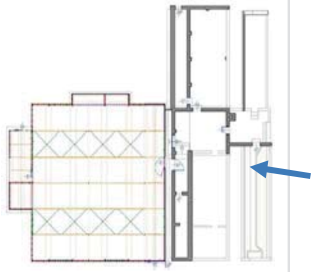


*Photo no. 448 -*

Ground floor plan

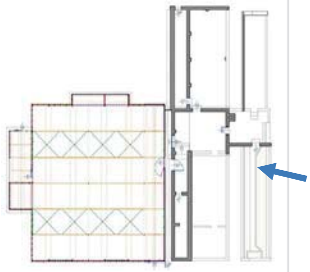
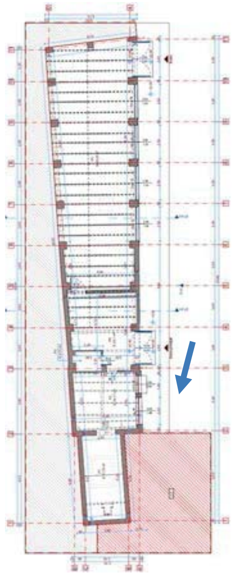


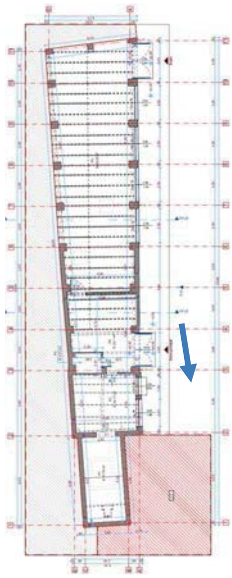
*Photo no. 449 -*

Ground floor plan

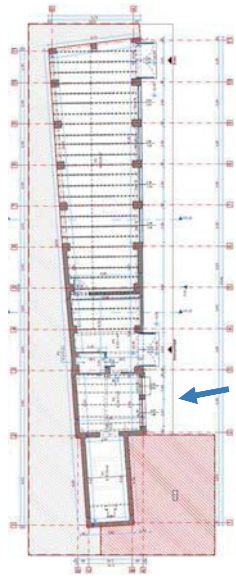
## Annex B.16 – Building C28 – C29



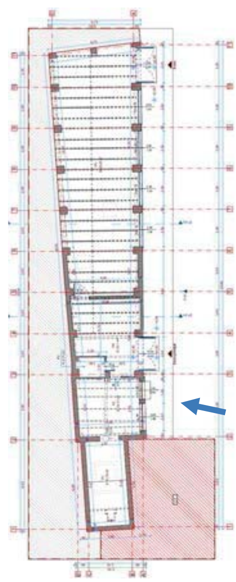
| *Photo no. 450 -* | Ground floor plan |
|---|---|



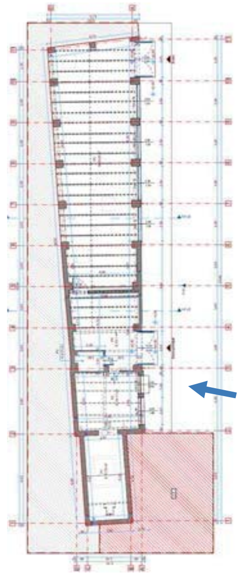
| *Photo no. 451 -* | Ground floor plan |
|---|---|



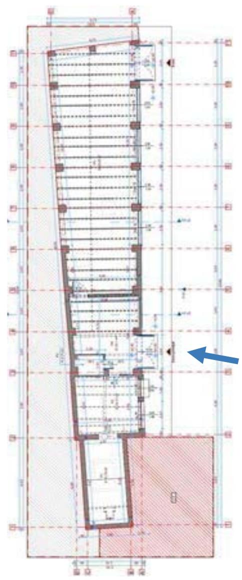
| Photo no. 452 - | Ground floor plan |
|---|---|



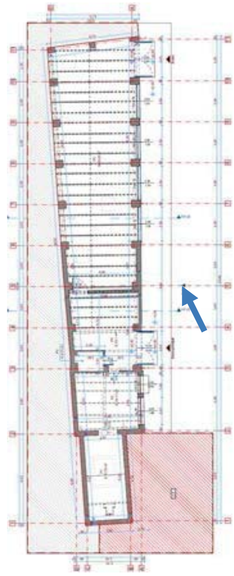
| Photo no. 453 - | Ground floor plan |
|---|---|



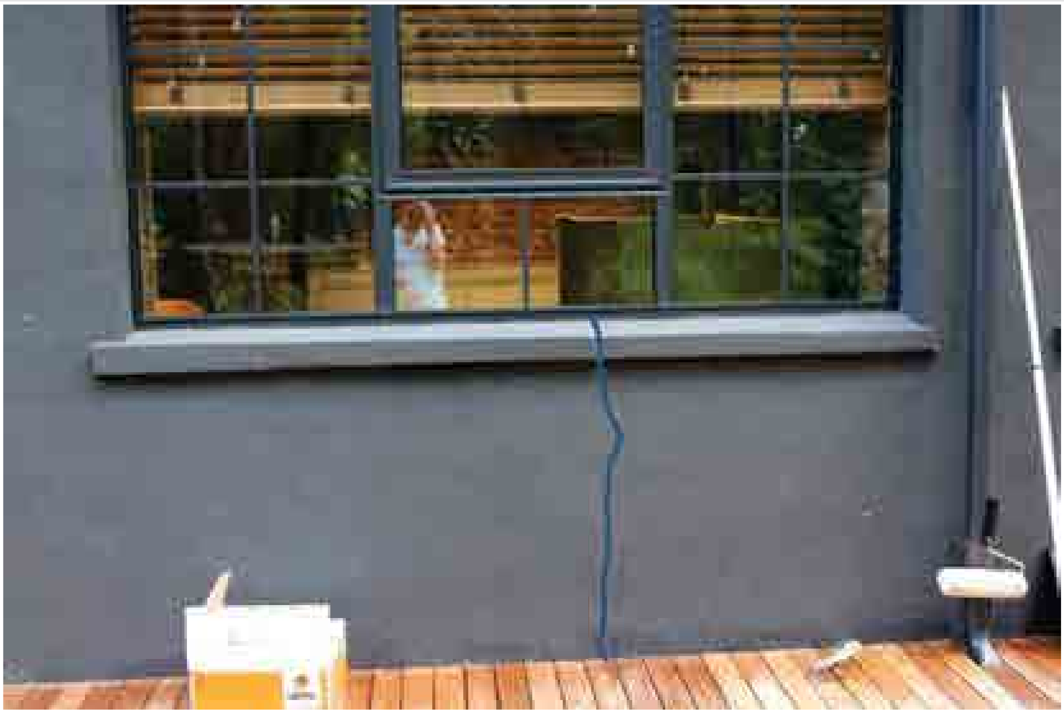
| *Photo no. 454 -* | Ground floor plan |
|---|---|

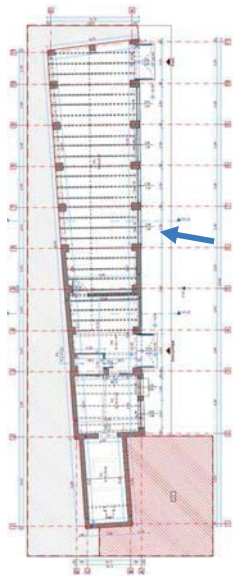


| *Photo no. 455 -* | Ground floor plan |
|---|---|

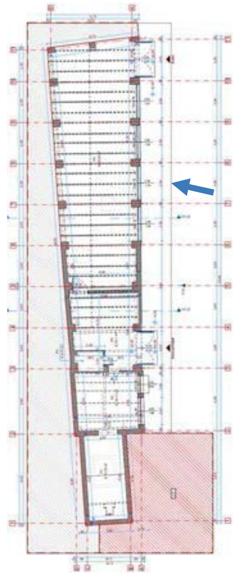


| | |
|---|---|
| *Photo no. 456 -* | Ground floor plan |

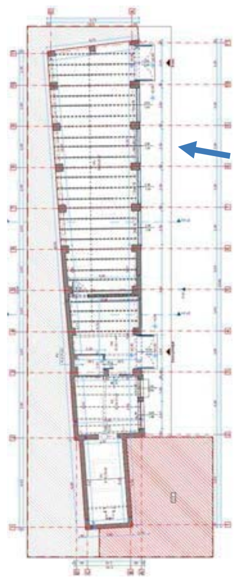




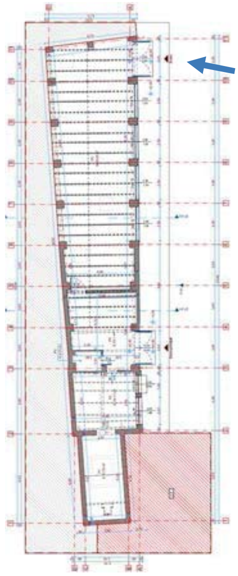
| | |
|---|---|
| *Photo no. 457 -* | Ground floor plan |



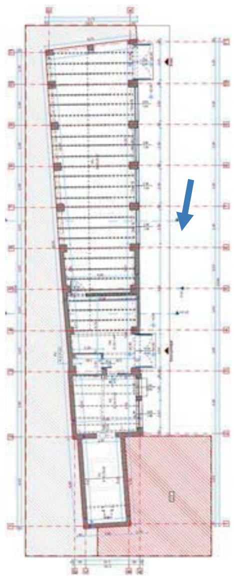
| | |
|---|---|
| *Photo no. 458 -* | Ground floor plan |



| | |
|---|---|
| *Photo no. 459 -* | Ground floor plan |



| *Photo no. 460 -* | Ground floor plan |
|---|---|



| *Photo no. 461 -* | Ground floor plan |
|---|---|

*Photo no. 462 -*

Ground floor plan

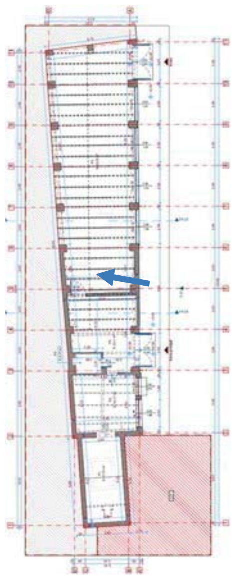
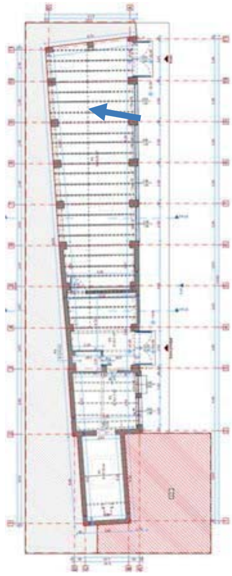
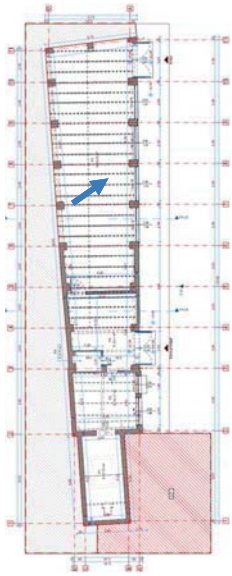
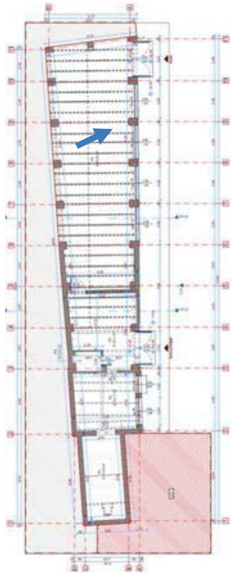


*Photo no. 463 -*

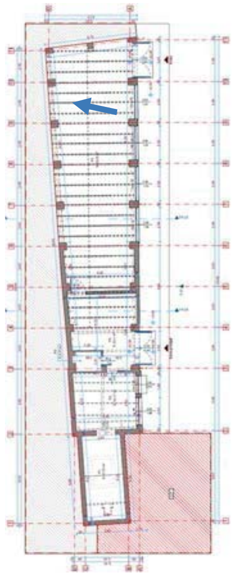
Ground floor plan

| | |
|---|---|
|  |  |
| *Photo no. 464 -* | Ground floor plan |

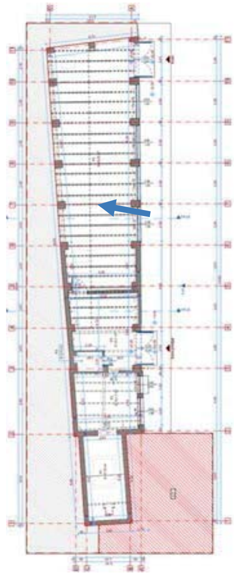
| | |
|---|---|
|  |  |
| *Photo no. 465 -* | Ground floor plan |



| *Photo no. 466 -* | Ground floor plan |
|---|---|



| *Photo no. 467 -* | Ground floor plan |
|---|---|

| | |
|---|---|
|  |  |
| *Photo no. 468 -* | Ground floor plan |

# ANNEX C – CALCULATION SUMMARY

## Annex C1 – Calculation summary - existing version – Building C7

## Annex C2 – Calculation summary - existing version – Building C8

## Annex C3 – Calculation summary - existing version – Building C10

## Annex C4 – Calculation summary - existing version – Building C13

## Annex C5 – Calculation summary - existing version – Building C17

## Annex C6 – Calculation summary - existing version – Building C20

## Annex C7 – Calculation summary - existing version – Building C24; C26; C27

## Annex C8 – Calculation summary - existing version – Building C25

## Annex C9 – Calculation summary - existing version – Building C28-29

# ANNEX C1 – CALCULATION BRIEF - EXISTING VARIANT – BUILDING C7

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of CR6-2013 (Annex A1), for the existing structure. The degrees of insurance for each masonry strip were established for shear force, axial force and bending moment, and a degree of insurance for each main direction of arrangement of the structural elements (longitudinal and transverse). The minimum degree of insurance was calculated for each strip and a degree of insurance, in each direction (longitudinal and transverse) and, informatively, a degree of insurance for the entire structure.

From the linear static analyses, using the response spectrum corresponding to the location, the effective values of the efforts for each wall resulted.

The ratio between the minimum effort capable in a section and the external one, represents the degree of insurance of the element (masonry strip).

The total degree of insurance resulting reflects the degree of insurance of the structure under the assumption that the floors are rigid enough to transmit horizontal loads from one strip to another, making them work together in the event of an earthquake. The slats for which the degree of insurance resulted with a value of zero are subjected to tension, an effort that they cannot take over.

# 1. MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered;*

- *the level of knowledge: KL3 – limited knowledge*
- *trust factor: CF = 1,35*

For the calculation in the linear elastic domain, considering the behavior factor q (reduced spectrum), the design strengths of the masonry for evaluating the capacity to resist bending with axial force and shear are taken as follows:

| Masonry resistances | | | | |
|---|---|---|---|---|
| Compression | | | | |
| Type | Investment | | | Standard |
| Compressive strength of the masonry element | $f_b$= | 10.00 | [N/mm2] | acc. SR EN 771 |
| Average compressive strength of mortar | $f_m$ = | 4.00 | [N/mm2] | acc. SR EN 998-2:2004 |
| K coefficient for solid ceramic bricks | K= | 0.55 | table 4.1, code CR6-2013 | |
| | constant that depends on the type of masonry element and the type of mortar | | | |
| Characteristic compressive strength | $f_k$ = | 3.80 | [N/mm2] | acc. SR EN 1052-1 |
| Trust factor | CF = | 1.35 | - | acc. 4.1 P100-3/2019 |
| Partial trust coefficient | $\gamma_M$ = | 3.00 | - | acc. D.3.3.1.2.(7) P100-3/2019 |
| Design resistance value for walls subjected to shear force | | | | |
| for horizontal joint sliding failure (fvd): | | | | |
| Resistance to sliding failure in horizontal joint | $f_{vd}$ = | $f_{vk}/(\gamma_M\ CF)$ | | acc. D.3.4.1.3.1.P100-3/2019 |
| Characteristic breaking strength | $f_{vk}$ = | $f_{yko} + * 0.7 \cdot \sigma d$ | | acc. 4.3.a CR6-2013 |
| The initial characteristic unit shear strength | $f_{vk0}$ = | 0.045 | [N/mm2] | |
| for breakage in the scale under the effect of main tensile stresses. (ftd) : | | $f_{td} = 0{,}04 \times f_m / (\gamma_M \times CF)$ | | |
| Breakage in the scale strength | $f_{td}$ = | 0.049 | [N/mm2] | acc. D.3.4.1.3.1.P100-3/2019 |
| Longitudinal modulus of elasticity of masonry | $E_z$ = | 3800 | [N/mm2] | acc. tab. 4.9 CR6-2013 |
| Transverse modulus of elasticity of masonry | $G_z$ = | 1520 | [N/mm2] | acc. rel. 4.9 CR6-2013 |

## 1.2 Geometric features

There are no original plans of the building, nor information on the building's behavior in 20th-century earthquakes.

The compositional characteristics were determined according to the construction survey.

# 2. LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

## 2.1 Permanent loads

Permanent floor loads above ground floor

| Wooden floors | | | | |
|---|---|---|---|---|
| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m2) |
| 1 | Floor | - | - | 0.30 |
| 2 | Wooden cabinet | 0.05 | 6 | 0.30 |
| 3 | Wooden beams | - | - | 0.50 |
| 4 | Filler | 0.03 | 16 | 0.48 |
| 5 | Wooden toe board | 0.03 | 6 | 0.18 |
| 6 | Plaster | 0.02 | 19 | 0.38 |
| **Total loads** | | | | **2.14** |

Permanent loads at roof level

| Item. no. | Load name | Standard load on surface [kN/m2] |
|---|---|---|
| 1 | Cover (sheeting+cladding) | 0.30 |
| 2 | Roof framing (plywood+props+struts+clamps+rafters) | 0.40 |
| **Total loads** | | **0.70** |

Load from the weight of masonry elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of solid brick masonry elements* | - | 18.0 | 18.0 |
| Total loads | | | | 18.0 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

## 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| $\gamma$ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m$^2$ |
| Total loads | **$p_{1,k}$ = 1.60 kN/m$^2$** |

Figure no. 1 - Zoning map of the characteristic value of the given snow load

(IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A ([]):
  - category A: $q_k = 1.50$ kN/m$^2$
- for building attics:
  - non-passable attics : $q_k = 0.75$ kN/m$^2$

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of “modal analysis with response spectrum” using a linear elastic model.  From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used
– using the combinations below:

$$E_{Edx} \text{ "+" } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ "+" } E_{Edy}$$

where:

$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction $FF0 and FI0( and the corresponding accidental eccentricity ($F_{xi}$ si $F_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

ei – accidental eccentricity of the mass at level i

$F_{xi}$ – static equivalent horizontal seismic force applied at level i

The accidental eccentricity is calculated with the expression: e0

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table No. 1 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j\geq 1} G_{k,j} + P + A_{ED} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$ |
| | Frequent grouping | $\sum_{j\geq 1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |

| | | |
|---|---|---|
| | Quasi-permanent grouping | $\sum_{j>1} G_{k,j} + P + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004. *Table no. 2 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3. LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

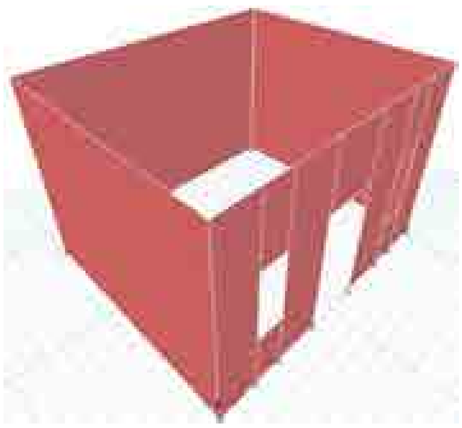


*Figure no. 2 - Calculation model – Spatial modeling – existing situation*

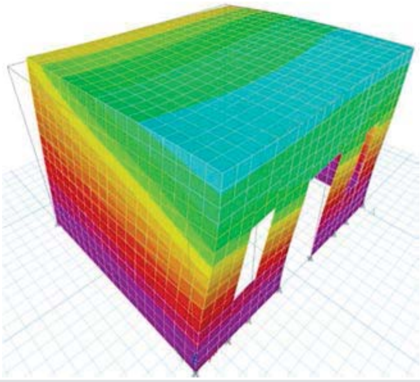
## 3.2 Analysis of natural vibration modes

In the calculation, 12 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

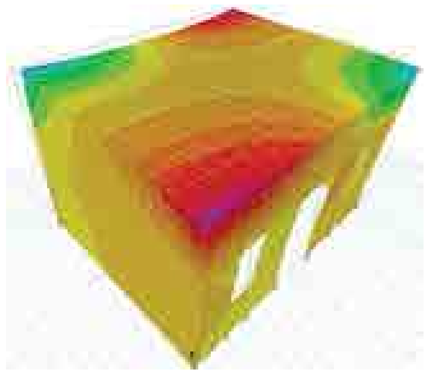
In the calculation, 12 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

*Table no. 3 -*

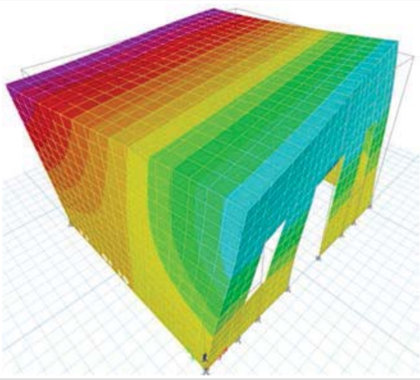
| Case | Mode | Period | Ux | Uy | Sum UX | Sum UY |
|---|---|---|---|---|---|---|
| Modal | 1 | 0.044 | 0.951 | 0.000 | 0.9505 | 0.0002 |
| Modal | 2 | 0.041 | 0.000 | 1.000 | 0.9507 | 0.9872 |
| Modal | 3 | 0.024 | 0.052 | 0.000 | 0.9953 | 0.9872 |
| Modal | 4 | 0.011 | 0.000 | 0.997 | 0.9953 | 0.9995 |
| Modal | 5 | 0.011 | 0.003 | 0.000 | 0.9956 | 0.9995 |
| Modal | 6 | 0.01 | 0.007 | 0.772 | 0.9956 | 0.9995 |
| Modal | 7 | 0.01 | 0.802 | 0.000 | 0.9995 | 0.9995 |
| Modal | 8 | 0.009 | 0.001 | 0.912 | 0.9995 | 0.9995 |
| Modal | 9 | 0.008 | 0.000 | 0.003 | 0.9995 | 0.9995 |
| Modal | 10 | 0.008 | 0.000 | 0.964 | 0.9995 | 0.9998 |
| Modal | 11 | 0.008 | 0.004 | 0.000 | 0.9996 | 0.9998 |
| Modal | 12 | 0.007 | 0.000 | 0.000 | 0.9996 | 0.9998 |



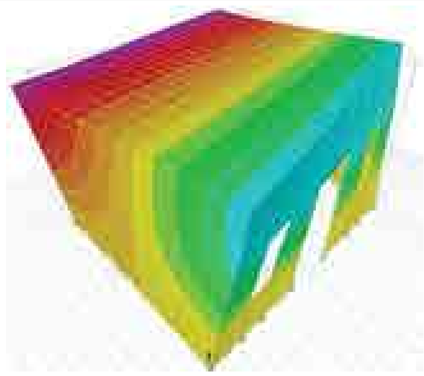
*Vibration mode 1 (3D view)*



*Vibration mode 2 (3D view)*



*Vibration mode 3 (3D view)*



*Vibration mode 4 (3D view)*

## 3.3 Analysis of forces in the slats

### 3.3.1 Calculation scheme - slats numbering

*Figure no. 1 - Ground floor beams – X; Y direction*

# 4. DEGREE OF STRUCTURE INSURANCE – QUANTITATIVE EVALUATION

## 4.1 Seismic assessment for the effects of in-plane wall action

The capable efforts in the beams were calculated with the relations from the normative P100-3/2019 Annex D.

### 4.1.1 Determining the degree of insurance for the entire building, on the slats in each direction

$$R_{3i} = \frac{V_{cap,i}}{F_{b,i}}$$

$V_{cap,i}$ - is the shear force capable of wall "i" (the smaller of $V_{fd}$ and $V_{ff}$).

*Table no. 2 - Degree of insurance on the shoulder straps in the longitudinal and transverse direction*

| Degree of insurance on the slats in the longitudinal direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| L01 | 0.25 | 0.71 | -21.35 | 0.10 | 1.89 | 2.79 | 10.75 | D | 1.89 | 0.00 | > 1.0 |
| L02 | 0.25 | 0.84 | -29.17 | 0.56 | 3.00 | 3.82 | 13.40 | D | 3.00 | 0.00 | > 1.0 |
| L03 | 0.25 | 0.86 | -30.23 | 3.02 | 3.18 | 3.96 | 13.78 | D | 3.18 | 0.00 | > 1.0 |
| L04 | 0.25 | 0.74 | -22.23 | 4.91 | 2.05 | 2.91 | 11.20 | D | 2.05 | 0.00 | 0.42 |
| L05 | 0.25 | 5.15 | -153.14 | 0.39 | 98.62 | 51.84 | 116.47 | F | 51.84 | 0.00 | > 1.0 |

| Degree of insurance on the slats in the transverse direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| T01 | 0.25 | 4.35 | -129.21 | 47.01 | 70.30 | 41.13 | 98.34 | F | 41.13 | 0.00 | 0.87 |
| T02 | 0.25 | 4.35 | -129.06 | 46.87 | 70.23 | 41.10 | 98.30 | F | 41.10 | 0.00 | 0.88 |

*Table no. 3 - Degree of insurance by structure*

| Direction | Investment |
|---|---|
| Ground floor - Longitudinal | **0.53** |
| Ground floor - Transversal | **0.88** |

| | |
|---|---|
| **Technical expert:** | eng. IOAN ROTĂRESCU |
| **Signature:** | |

# ANNEX C2 – CALCULATION BRIEF - EXISTING VARIANT – BUILDING C8

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of CR6-2013 (Annex A1), for the existing structure. The degrees of insurance for each masonry strip were established for shear force, axial force and bending moment, and a degree of insurance for each main direction of arrangement of the structural elements (longitudinal and transverse). The minimum degree of insurance was calculated for each strip and a degree of insurance, in each direction (longitudinal and transverse) and, informatively, a degree of insurance for the entire structure.

From the linear static analyses, using the response spectrum corresponding to the location, the effective values of the efforts for each wall resulted.

The ratio between the minimum effort capable in a section and the external one, represents the degree of insurance of the element (masonry strip).

The total degree of insurance resulting reflects the degree of insurance of the structure under the assumption that the floors are rigid enough to transmit horizontal loads from one strip to another, making them work together in the event of an earthquake. The slats for which the degree of insurance resulted with a value of zero are subjected to tension, an effort that they cannot take over.

# 1. MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered; - the level of knowledge: KL3 – limited knowledge*

*- trust factor: CF = 1,35*

For the calculation in the linear elastic domain, considering the behavior factor q (reduced spectrum), the design strengths of the masonry for evaluating the capacity to resist bending with axial force and shear are taken as follows:

| Masonry resistances | | | | |
|---|---|---|---|---|
| Compression | | | | |
| Type | Investment | | | Standard |
| Compressive strength of the masonry element | $f_b$= | 10.00 | [N/mm2] | acc. SR EN 771 |
| Average compressive strength of mortar | $f_m$ = | 4.00 | [N/mm2] | acc. SR EN 998-2:2004 |
| K coefficient for solid ceramic bricks | K= | 0.55 | table 4.1, code CR6-2013 | |
| | constant that depends on the type of masonry element and the type of mortar | | | |
| Characteristic compressive strength | $f_k$ = | 3.80 | [N/mm2] | acc. SR EN 1052-1 |
| Trust factor | CF = | 1.35 | - | acc. 4.1 P100-3/2019 |
| Partial trust coefficient | $\gamma_M$ = | 3.00 | - | acc. D.3.3.1.2.(7) P100-3/2019 |
| Design resistance value for walls subjected to shear force | | | | |
| for horizontal joint sliding failure (fvd): | | | | |
| Resistance to sliding failure in horizontal joint | $f_{vd}$ = | $f_{vk}/(\gamma_M \cdot CF)$ | | acc. D.3.4.1.3.1.P100-3/2019 |
| Characteristic breaking strength | $f_{vk}$ = | $f_{vko} + 0.7 \cdot \sigma_d$ | | acc. 4.3.a CR6-2013 |
| The initial characteristic unit shear strength | $f_{vk0}$ = | 0,045 | [N/mm2] | |
| for breakage in the scale under the effect of main tensile stresses. (ftd) : | $f_{td}$ = 0,04 x $f_m$ / ($\gamma_M$ x CF) | | | |
| Breakage in the scale strength | $f_{td}$ = | 0.049 | [N/mm2] | acc. D.3.4.1.3.1.P100-3/2019 |
| Longitudinal modulus of elasticity of masonry | $E_z$ = | 3800 | [N/mm2] | acc. tab. 4.9 CR6-2013 |
| Transverse modulus of elasticity of masonry | $G_z$ = | 1520 | [N/mm2] | acc. rel. 4.9 CR6-2013 |

## 1.2 Geometric features

There are no original plans of the building, nor information on the building's behavior in 20th-century earthquakes.

The compositional characteristics were determined according to the construction survey.

## 2. LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

### 2.1 Permanent loads

Permanent floor loads above ground floor

| Wooden floors | | | | |
|---|---|---|---|---|
| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m2) |
| 1 | Floor | - | - | 0.30 |
| 2 | Wooden cabinet | 0.05 | 6 | 0.30 |
| 3 | Wooden beams | - | - | 0.50 |
| 4 | Filler | 0.03 | 16 | 0.48 |
| 5 | Wooden toe board | 0.03 | 6 | 0.18 |
| 6 | Plaster | 0.02 | 19 | 0.38 |
| **Total loads** | | | | **2.14** |

Permanent loads at roof level

| Item. no. | Load name | Standard load on surface [kN/m2] |
|---|---|---|
| 1 | Cover (sheeting+cladding) | 0.30 |
| 2 | Roof framing (plywood+props+struts+clamps+rafters) | 0.40 |
| **Total loads** | | **0.70** |

Load from the weight of masonry elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of solid brick masonry elements* | - | 18.0 | 18.0 |
| Total loads | | | | 18.0 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

### 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| $\gamma$ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 3 - Zoning map of the characteristic value of the given snow load

(IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A ([]):
  - category A: $q_k = 1.50$ kN/m$^2$
- for building attics:
  - non-passable attics : $q_k = 0.75$ kN/m$^2$

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of "modal analysis with response spectrum" using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used
– using the combinations below:

$$E_{Edx} \text{ " + " } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ " + " } E_{Edy}$$

where:
$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction $FF0 and FI0( and the corresponding accidental eccentricity ($F_{xi}$ si $F_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i

$F_{xi}$ – static equivalent horizontal seismic force applied at level i

The accidental eccentricity is calculated with the expression: e0

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table No. 4 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j\geq 1} G_{k,j} + P + A_{ED} + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$ |
| | Frequent grouping | $\sum_{j\geq 1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |

| | | |
|---|---|---|
| | Quasi-permanent grouping | $\sum_{j\geq 1} G_{k,j} + P + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004. *Table no. 5 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3. LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

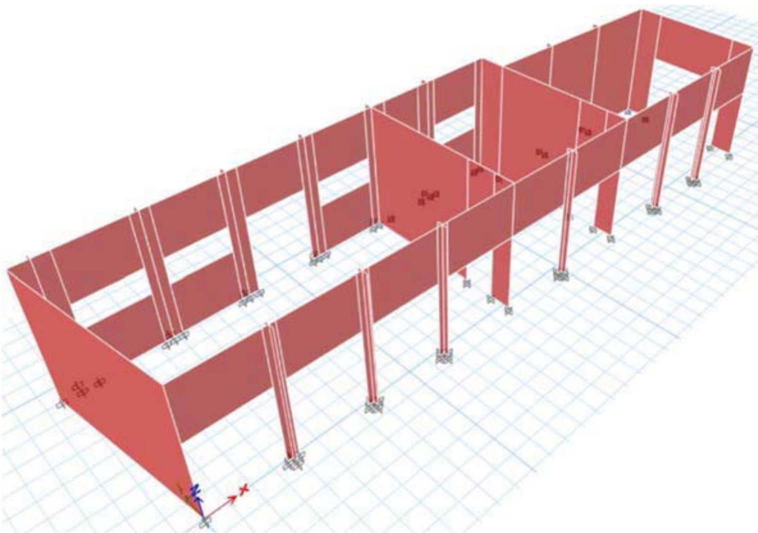


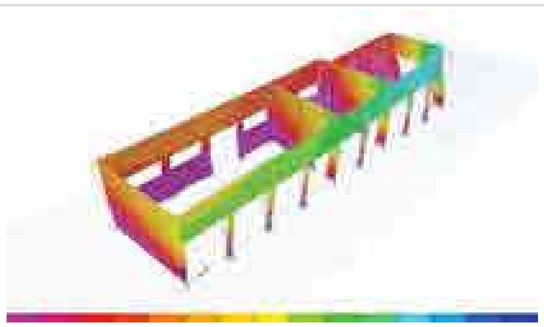
*Figure no. 4 - Calculation model – Spatial modeling – existing situation*

## 3.2 Analysis of natural vibration modes

In the calculation, 12 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

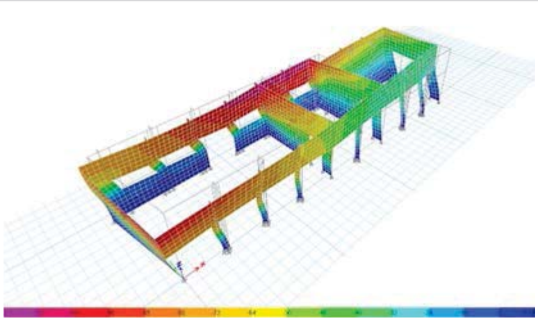
*Table no. 6 -*

| Case | Mode | Period | Ux | Uy | Sum UX | Sum UY |
|---|---|---|---|---|---|---|
| Modal | 1 | 0.086 | 0.336 | 0.202 | 0.3277 | 0.1967 |
| Modal | 2 | 0.062 | 0.199 | 0.801 | 0.5155 | 0.9519 |
| Modal | 3 | 0.053 | 0.883 | 0.057 | 0.9805 | 0.9817 |
| Modal | 4 | 0.035 | 0.037 | 0.003 | 0.9851 | 0.9821 |
| Modal | 5 | 0.024 | 0.000 | 0.002 | 0.9852 | 0.9864 |
| Modal | 6 | 0.024 | 0.004 | 0.002 | 0.9959 | 0.9917 |
| Modal | 7 | 0.021 | 0.003 | 0.021 | 0.996 | 0.9922 |
| Modal | 8 | 0.018 | 0.000 | 0.025 | 0.996 | 0.9962 |
| Modal | 9 | 0.016 | 0.000 | 0.009 | 0.996 | 0.9986 |
| Modal | 10 | 0.016 | 0.001 | 0.001 | 0.9967 | 0.9991 |
| Modal | 11 | 0.014 | 0.000 | 0.000 | 0.9972 | 0.9991 |
| Modal | 12 | 0.014 | 0.000 | 0.000 | 0.9985 | 0.9991 |

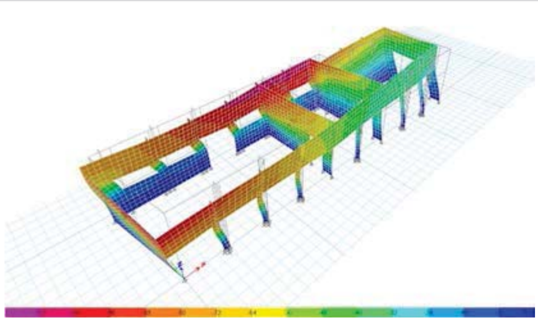


*Vibration mode 1 (3D view)*

*Vibration mode 2 (3D view)*



*Vibration mode 3 (3D view)*



*Vibration mode 4 (3D view)*

## 3.3 Analysis of forces in the slats

### 3.3.1 Calculation scheme - slats numbering

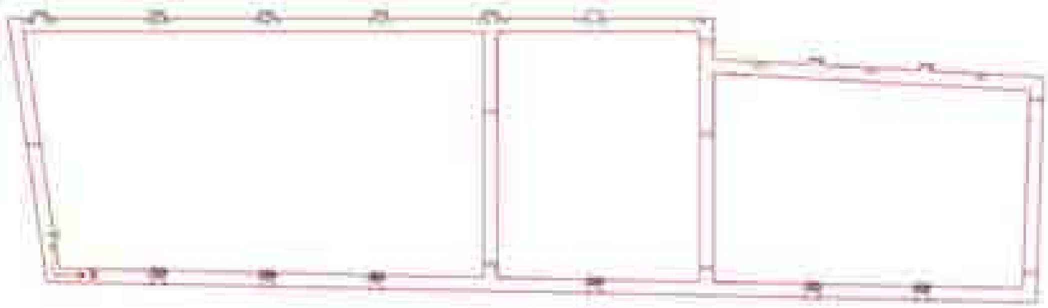


*Figure no. 2 - Ground floor beams – X; Y direction*

# 4. DEGREE OF STRUCTURE INSURANCE – QUANTITATIVE EVALUATION

## 4.1 Seismic assessment for the effects of in-plane wall action

The capable efforts in the beams were calculated with the relations from the normative P100-3/2019 Annex D.

### 4.1.1 Determining the degree of insurance for the entire building, on the slats in each direction

$$R_{3i} = \frac{V_{cap,i}}{F_{b,i}}$$

$V_{cap,i}$ - is the shear force capable of wall "i" (the smaller of $V_{fd}$ and $V_{ff}$).

*Table no. 4 - Degree of insurance on the shoulder straps in the longitudinal and transverse direction*

| Degree of insurance on the slats in the longitudinal direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| L01 | 0.25 | 0.27 | -26.76 | 0.41 | 0.64 | 3.50 | 6.63 | D | 0.64 | 0.00 | > 1 |
| L02 | 0.25 | 0.27 | -31.74 | 0.64 | 0.67 | 4.15 | 7.16 | D | 0.67 | 0.00 | > 1 |
| L03 | 0.25 | 0.27 | -28.78 | 0.79 | 0.65 | 3.77 | 6.85 | D | 0.65 | 0.00 | 0.83 |
| L04 | 0.25 | 0.27 | -23.39 | 0.80 | 0.60 | 3.06 | 6.25 | D | 0.60 | 0.00 | 0.75 |
| L05 | 0.25 | 0.27 | -25.32 | 0.79 | 0.62 | 3.31 | 6.47 | D | 0.62 | 0.00 | 0.79 |
| L06 | 0.25 | 0.27 | -25.34 | 0.99 | 0.62 | 3.32 | 6.47 | D | 0.62 | 0.00 | 0.63 |
| L07 | 0.25 | 6.06 | -177.2 | 101.71 | 134.62 | 62.69 | 136.27 | F | 62.69 | 0.00 | 0.62 |
| L08 | 0.25 | 0.81 | -21.86 | 2.11 | 2.25 | 2.86 | 11.82 | D | 2.25 | 0.00 | > 1 |
| L09 | 0.25 | 0.43 | -7.88 | 3.24 | 0.44 | 1.03 | 5.53 | D | 0.44 | 0.00 | 0.14 |
| L10 | 0.25 | 0.47 | -11.14 | 1.67 | 0.66 | 1.46 | 6.51 | D | 0.66 | 0.00 | 0.40 |
| L11 | 0.25 | 0.42 | -7.65 | 2.97 | 0.42 | 1.00 | 5.35 | D | 0.42 | 0.00 | 0.14 |
| L12 | 0.25 | 0.63 | -21.69 | 4.80 | 1.67 | 2.84 | 9.99 | D | 1.67 | 0.00 | 0.35 |
| L13 | 0.25 | 0.56 | -14.77 | 3.49 | 1.06 | 1.93 | 8.13 | D | 1.06 | 0.00 | 0.30 |
| L14 | 0.25 | 0.27 | -5.06 | 2.85 | 0.18 | 0.66 | 3.45 | D | 0.18 | 0.00 | 0.06 |

| Degree of insurance on the slats in the transverse direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| T01 | 0.25 | 4.61 | -155.83 | 64.77 | 88.33 | 49.04 | 109.23 | F | 49.04 | 0.00 | 0.76 |
| T02 | 0.25 | 0.24 | -8.05 | 1.11 | 0.24 | 1.05 | 3.82 | D | 0.24 | 0.00 | 0.22 |
| T03 | 0.25 | 0.27 | -26.76 | 3.51 | 0.64 | 3.50 | 6.63 | D | 0.64 | 0.00 | 0.18 |
| T04 | 0.25 | 0.25 | -25.75 | 2.53 | 0.54 | 3.37 | 6.18 | D | 0.54 | 0.00 | 0.21 |
| T05 | 0.25 | 0.27 | -31.74 | 5.29 | 0.67 | 4.15 | 7.16 | D | 0.67 | 0.00 | 0.13 |
| T06 | 0.25 | 0.25 | -39.10 | 4.88 | 0.54 | 5.12 | 7.48 | D | 0.54 | 0.00 | 0.11 |
| T07 | 0.25 | 0.27 | -28.78 | 3.56 | 0.65 | 3.77 | 6.85 | D | 0.65 | 0.00 | 0.18 |
| T08 | 0.25 | 0.25 | -27.01 | 3.74 | 0.54 | 3.53 | 6.31 | D | 0.54 | 0.00 | 0.15 |
| T09 | 0.25 | 0.58 | -49.48 | 1.15 | 2.74 | 6.47 | 13.33 | D | 2.74 | 0.00 | > 1.0 |
| T10 | 0.25 | 3.58 | -180.40 | 37.39 | 73.69 | 46.60 | 98.93 | F | 46.60 | 0.00 | > 1.0 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| T11 | 0.25 | 0.59 | -50.06 | 2.48 | 2.81 | 6.55 | 13.50 | D | 2.81 | 0.00 | > 1.0 |
| T12 | 0.25 | 3.40 | -151.42 | 62.25 | 60.43 | 38.08 | 85.85 | F | 38.08 | 0.00 | 0.61 |
| T13 | 0.25 | 0.27 | -25.32 | 1.29 | 0.62 | 3.31 | 6.47 | D | 0.62 | 0.00 | 0.48 |
| T14 | 0.25 | 0.24 | -13.84 | 1.60 | 0.36 | 1.81 | 4.62 | D | 0.36 | 0.00 | 0.22 |
| T15 | 0.25 | 0.27 | -25.34 | 1.21 | 0.62 | 3.32 | 6.47 | D | 0.62 | 0.00 | 0.52 |
| T16 | 0.25 | 0.24 | -14.50 | 1.95 | 0.38 | 1.90 | 4.77 | D | 0.38 | 0.00 | 0.19 |
| T17 | 0.25 | 0.67 | -52.41 | 10.25 | 3.52 | 6.86 | 14.88 | D | 3.52 | 0.00 | 0.34 |
| T18 | 0.25 | 0.73 | -32.74 | 19.96 | 2.79 | 4.28 | 12.81 | D | 2.79 | 0.00 | 0.14 |
| T19 | 0.25 | 0.27 | -23.39 | 0.66 | 0.60 | 3.06 | 6.25 | D | 0.60 | 0.00 | 0.91 |

*Table no. 5 - Degree of insurance by structure*

| Direction | Investment |
|---|---|
| Ground floor - Longitudinal | **0.54** |
| Ground floor - Transversal | **0.61** |

| **Technical expert:** | eng. IOAN ROTĂRESCU |
|---|---|
| **Signature:** | |

## ANNEX C3 – CALCULATION BRIEF - EXISTING VARIANT – BUILDING C10

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of SR EN 1993 , for the existing structure.

# 1 MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered;*

*- the level of knowledge: KL3 – limited knowledge*

*- trust factor: CF = 1,35*

| Type | Investment | Standard |
|---|---|---|
| Minimum yield strength | fy=235 [N/mm2] | acc. tab.3.1, SR EN1993-1-1/2006 |
| Ultimate tensile strength | fu=360 [N/mm2] | acc. tab.3.1, SR EN1993-1-1/2006 |
| Poisson's ratio in the elastic range | n=0.3 | acc. 3.2.6, SR EN1993-1-1/2006 |
| Longitudinal modulus of elasticity | E=210000 [N/mm2] | acc. 3.2.6, SR EN1993-1-1/2006 |
| Partial safety factor | $\gamma_{M0}$=1.00 | acc. 6.1, SR EN1993-1-1/2006 |
| Partial safety factor | $\gamma_{M1}$=1.00 | acc. 6.1, SR EN1993-1-1/2006 |
| Partial safety factor | $\gamma_{M2}$=1.25 | acc. 6.1, SR EN1993-1-1/2006 |

## 1.2 Geometric features

The compositional characteristics were determined according to the construction survey.

# 2 LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

## 2.1 Permanent loads

Load from the weight of metal elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of steel elements* | - | 78.5 | 78.5 |
| Total loads | | | | 78.5 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

## 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| γ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |

| | |
|---|---|
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 5 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A ([]):

  - category A: $q_k$ = 1.50 kN/m$^2$

***- Wind loads***

According to CR 1-1-4 2012 “Evaluation of wind action on buildings” the wind load is calculated according to the expression:

| $w_e = \gamma I_w \; c_f \; q_p(z_e)$ | |
|---|---|
| $\gamma_{Iw}$ | importance factor |
| $q_b$ | dynamic wind pressure value on site |
| $z_e$ | reference height for external action |
| r | air density |
| vb | reference wind speed on site |
| $k_r(z_0)$ | Land factor |
| $z_0$ | roughness length |
| $c_r(z)$ | roughness factor |
| $v_m(z)$ | average wind speed at a height z=ze |
| √β | proportionality factor |
| $I_v(z)$ | wind turbulence intensity |
| $c_{pv}(z)$ | gust factor for average wind speed |
| $v_p(z)$ | peak wind speed at height z=ze produced by gusts |
| $k_r(z_0)^2$ | Land factor |
| $c_r(z)^2$ | roughness factor for dynamic wind pressure at height ze |
| $c_{pq}(z)$ | gust factor for average dynamic wind pressure at height ze |
| $q_p(z)$ | peak dynamic wind pressure at height ze |
| $c_f$ -1 | force coefficient (for headwind) |

| | |
|---|---|
| $c_f$ -2 | force coefficient (for wind at 450) |

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of "modal analysis with response spectrum" using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{ik}\right)^2 / \sum m_i \cdot s_{ik}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used
– using the combinations below:

$$E_{Edx} \text{ "+" } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ "+" } E_{Edy}$$

where:

$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as product of the horizontal forces in each horizontal direction ($F_{xi}$ si $F_{yi}$) and the corresponding accidental eccentricity ($e_{xi}$ si $e_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i

$F_{xi}$ – static equivalent horizontal seismic force applied at level i

The accidental eccentricity is calculated with the expression:

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table No. 7 - Load combination

| | | |
|---|---|---|
| SLU | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j>1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j>1} G_{k,j} + P + A_{ED} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| SLS | Characteristic grouping | $\sum_{j>1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$ |
| | Frequent grouping | $\sum_{j>1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| | Quasi-permanent grouping | $\sum_{j>1} G_{k,j} + P + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004.

*Table no. 8 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3 LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.

- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

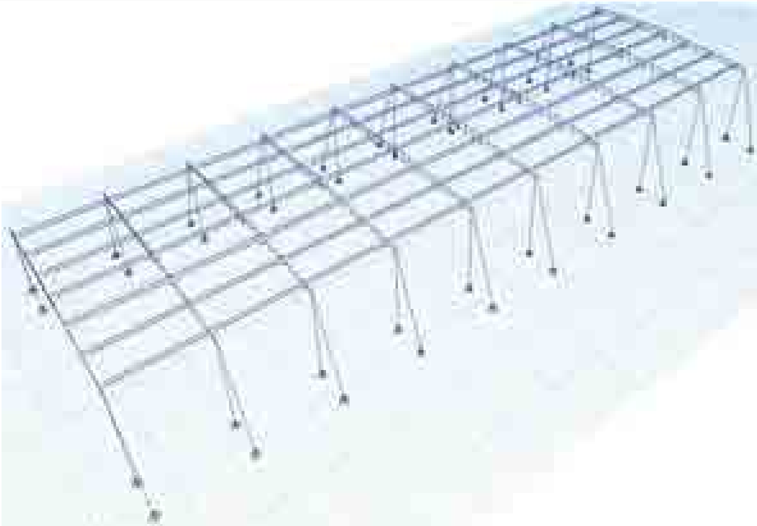


*Figure no. 1 - Calculation model – Spatial modeling – existing situation*

## 3.2 Calculation scheme

For each element, the capable axial force and the design buckling resistance are determined, these efforts are compared with the effective efforts resulting from the linear static analysis, resulting in the efficiency of the bars.

The design value of the tensile stress NEd in each cross-section must satisfy the following condition:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$$

$N_{Ed}$ – axial force calculation of tensile stress;

$N_{t,Rd}$ – axial tensile force;

For sections with holes, the design value Nt,Rd of the tensile strength shall be taken equal to the lower of the values below:

a) the design value of the plastic resistance in the gross cross-section

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$$

b) the design value of the ultimate resistance of the net cross-section, in the direction of the fixing holes

$$N_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2}}$$

A – the gross area of the cross-section;

$A_{net}$ – net area, in front of the fixing holes, of the cross-section;

$f_y$ – minimum yield strength of the steel;

$f_u$ – ultimate tensile strength of steel.

The design value of the compressive stress NEd in each cross-section must satisfy the following condition:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0$$

$N_{Ed}$ – axial force calculation of the compressive stress;

$N_{c,Rd}$ – axial force capable of compression;

The design value of the cross-sectional resistance to uniform compression is determined as follows:

$$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \quad q$$

Compression bars must be checked for buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

$N_{Ed}$ – axial force calculation of the compressive stress;

$N_{b,Rd}$ – design resistance of the compressed bar to buckling;

The design buckling resistance of a compressed bar is equal to:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

In the case of concentrically compressed bars, the value of x must be calculated, taking into account the corresponding buckling curve, using the following relationship:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \; dar \; \chi \leq 1.0$$

$$\Phi = 0.5 \cdot \left[1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

α – imperfection factor;

$N_{cr}$ – critical axial elastic buckling stress, corresponding to the considered buckling mode, calculated based on the characteristics of the gross cross-section.

## 4. DEGREE OF INSURANCE

The R3 coefficient for the entire building, for each type of section and in each group, is presented in the following table:

| **Load grouping** | **Section Type** | | |
|---|---|---|---|
| | **1 (4 L40X5)** | **2 (2 L40X5)** | **3 ( 8 mm bar)** |
| | **Insurance level by section type** | | |
| G1 - P | 0.93 | 1.20 | 1.56 |
| G2 - Z | **0.60** | 1.05 | 0.70 |
| G3-V | **0.75** | 1.30 | 1.47 |
| GS-S | **0.55** | 1.42 | 1.71 |

| | |
|---|---|
| **Technical expert:** | eng. DANIEL DIACONU |
| **Signature:** | |

# ANNEX C4 – CALCULATION BRIEF - EXISTING VARIANT – BUILDING C13

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of SR EN 1992 , (with additional annexes) for the existing structure. A degree of insurance was established for each column, for shear force, axial force and bending moment, and, for information, a degree of insurance for the entire structure.

From the linear static analyses, using the response spectrum corresponding to the location, the effective values of the efforts for each wall resulted.

# 1. MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

Given the limited level of inspection and testing, it was considered;

| Structural element | Material | Type | Characteristic resistance | Calculation resistance |
|---|---|---|---|---|
| Poles | concrete | monolithic | $f_{ck}$=12 N/mm2 | $f_{cdm}$= 8.5 N/mm2 |
| | longitudinal reinforcement | | $f_{yk}$=345 N/mm2 | $f_{ydm}$=261 N/mm2 |
| | transverse reinforcement | | $f_{yk}$=255 N/mm2 | $f_{ydm}$=193 N/mm2 |
| Beams | concrete | monolithic | $f_{ck}$=12 N/mm2 | $f_{cdm}$=8.5 N/mm2 |
| | longitudinal reinforcement | | $f_{yk}$=355 N/mm2 | $f_{ydm}$=268 N/mm2 |
| | transverse reinforcement | | $f_{yk}$=255 N/mm2 | $f_{ydm}$=193 N/mm2 |
| Slabs | concrete | monolithic | $f_{ck}$=12 N/mm2 | $f_{cdm}$=8.5 N/mm2 |
| | reinforcement | | $f_{yk}$=255 N/mm2 | $f_{ydm}$=193 N/mm2 |

## 1.2 Geometric features

There are no original plans of the building, nor information on the building's behavior in 20th-century earthquakes.

The compositional characteristics were determined according to the construction survey.

# 2. LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

## 2.1 Permanent loads

Permanent terrace slab loads

| Reinforced concrete slabs 15 cm | | | | |
|---|---|---|---|---|
| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m2) |
| 1 | Waterproofing and thermal insulation layers | 0.20 | 9 | 1.80 |
| 2 | Reinforced concrete slab | 0.15 | 25 | - |
| 3 | Plaster | 0.02 | 19 | 0.38 |
| | | | Total loads | 2.20 |

Load from the weight of reinforced concrete elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of reinforced concrete elements* | - | 25 | 25 |
| | | | Total loads | 25 |

| | | | | |
|---|---|---|---|---|
| 2 | Weight of plain concrete elements* | - | 24 | 24 |
| | | | Total loads | 24 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

## 2.2 Variable loads

### *- snow load (according to the code CR 1-1-3-2012)*

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $s_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| $\gamma$ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1 |
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 2 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

### *- useful loads*

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category C
  - subcategoria C: $q_k = 2$ kN/m$^2$
- terasa:
  - non-passable terrace: $q_k = 0.75$ kN/m$^2$

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of "modal analysis with response spectrum" using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.0 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 2.50 |
| - fraction of critical damping: **5%** (acc. P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with an effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining maximum stresses and displacements, among which we mention the method used

– using the combinations below:

$$E_{Edx} \text{ "+" } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ "+" } E_{Edy}$$

where:

$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction ($F_{xi}$ si $F_{yi}$) and the corresponding accidental eccentricity ($e_{xi}$ si $e_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i

$F_{xi}$ – static equivalent horizontal seismic force applied at level i.

The accidental eccentricity is calculated with the expression:

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table no. 9 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j\geq1}\gamma_{G,j}G_{k,j} + \gamma_P P + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j\geq1}G_{k,j} + P + A_{Ed} + \sum_{i\geq1}\Psi_{2,i}Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j\geq1}G_{k,j} + P + Q_{k,1} + \sum_{i>1}\Psi_{0,i}Q_{k,i}$ |
| | Frequent grouping | $\sum_{j\geq1}G_{k,j} + P + \Psi_{1,1}Q_{k,1} + \sum_{i>1}\Psi_{2,i}Q_{k,i}$ |
| | Quasi-permanent grouping | $\sum_{j\geq1}G_{k,j} + P + \sum_{i\geq1}\Psi_{2,i}Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004. *Table no. 10 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3. LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations

- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

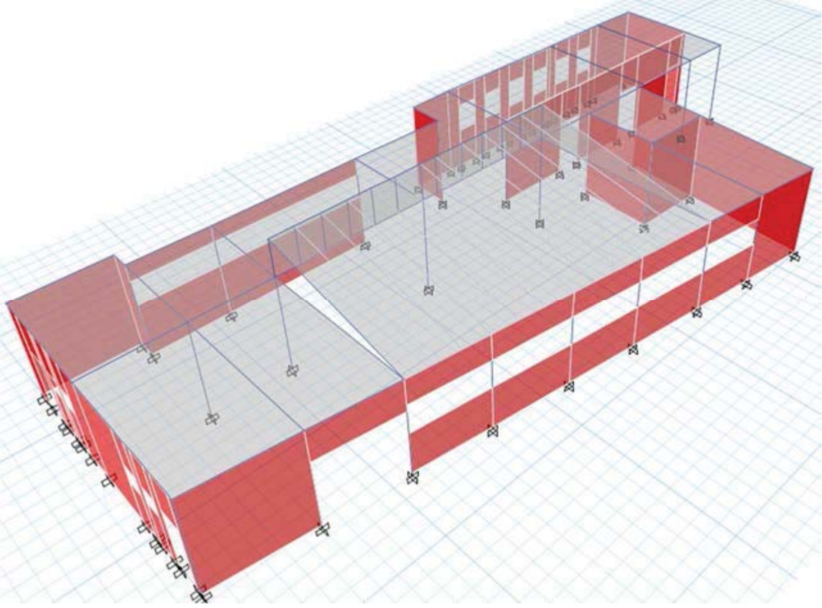


*Figure no. 3 - Calculation model – Spatial modeling – existing situation*

## 3.2 Analysis of natural vibration modes

In the calculation, 9 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

*Table no. 11 -*

| Case | Mode | Period | Ux | Uy | Sum UX | Sum UY |
|---|---|---|---|---|---|---|
| Modal | 1 | 0,092 | 0.0000 | 0.7270 | 0.0000 | 0.7270 |
| Modal | 2 | 0,061 | 0.9316 | 0.0018 | 0.9316 | 0.7288 |
| Modal | 3 | 0,054 | 0.0049 | 0.0063 | 0.9365 | 0.7351 |
| Modal | 4 | 0,048 | 0.0012 | 0.2043 | 0.9377 | 0.9394 |
| Modal | 5 | 0,042 | 0.0182 | 0.0431 | 0.9560 | 0.9826 |
| Modal | 6 | 0,036 | 0.0014 | 0.0005 | 0.9573 | 0.9831 |
| Modal | 7 | 0,033 | 0.0124 | 0.0016 | 0.9698 | 0.9847 |
| Modal | 8 | 0.03 | 0.0201 | 0.0013 | 0.9899 | 0.9860 |
| Modal | 9 | 0,029 | 0.0015 | 0.0020 | 0.9914 | 0.9880 |

*Vibration mode 1 (3D view)*

*Vibration mode 2 (3D view)*

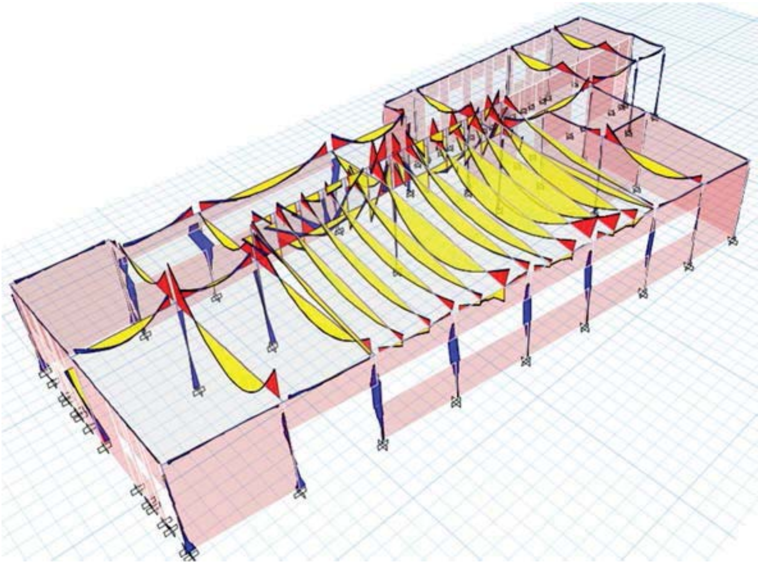
*Vibration mode 3 (3D view)*

*Vibration mode 4 (3D view)*

## 3.3 Effort analysis

### 3.3.1 Effort diagrams

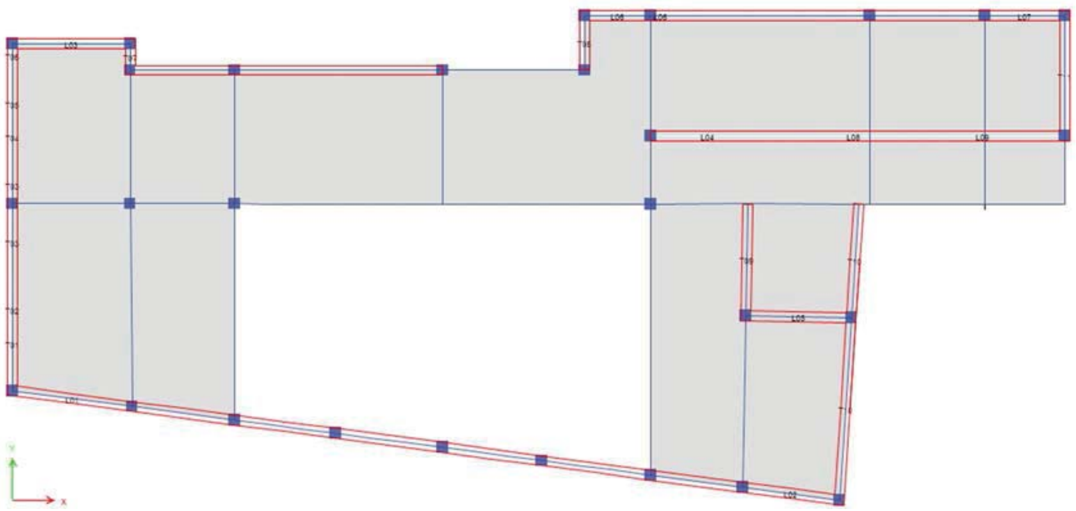
Axial stress diagram in columns



Bending moment stress diagram in columns

## 4. DEGREE OF STRUCTURE INSURANCE – QUANTITATIVE EVALUATION

### 4.1 Layout and naming of poles and walls



Tabelul nr. 12 - Degree of structural seismic insurance of the poles

| Degrees of column insurance in longitudinal direction X | | | | | |
|---|---|---|---|---|---|
| Pole | b pole | h pole | Degree of insurance | | |
| | [mm] | [mm] | $N_x$ | $M_x$ | $V_x$ |
| C1 | 350 | 350 | 1.64 | 0.81 | 0.77 |
| C2 | 350 | 350 | 1.18 | 0.59 | 0.94 |
| C3 | 350 | 350 | 1.10 | 0.61 | 0.91 |
| C4 | 350 | 350 | 1.31 | 0.86 | 0.72 |
| C5 | 350 | 350 | 1.18 | 0.60 | 1.00 |
| C6 | 350 | 350 | 0.86 | 1.25 | 0.54 |
| C7 | 350 | 350 | 0.85 | 1.24 | 0.54 |
| C8 | 350 | 350 | 1.39 | 0.69 | 0.86 |
| C9 | 350 | 350 | 1.44 | 0.59 | 1.00 |

| | | | | | |
|---|---|---|---|---|---|
| C10 | 350 | 350 | 0.86 | 0.68 | 1.00 |
| C11 | 350 | 350 | 0.99 | 0.65 | 1.00 |
| C12 | 350 | 350 | 1.45 | 0.59 | 1.00 |
| C13 | 350 | 350 | 1.63 | 0.60 | 1.00 |
| C14 | 350 | 350 | 1.16 | 0.63 | 1.00 |
| C15 | 350 | 350 | 0.91 | 0.68 | 1.00 |
| C16 | 350 | 350 | 1.44 | 0.60 | 1.00 |
| C17 | 350 | 350 | 1.62 | 0.59 | 1.00 |
| C18 | 350 | 350 | 1.03 | 0.65 | 1.00 |
| C19 | 350 | 350 | 0.80 | 0.68 | 1.00 |
| C20 | 350 | 350 | 1.37 | 0.59 | 1.00 |
| C22 | 350 | 350 | 1.41 | 0.71 | 0.86 |
| C23 | 350 | 350 | 0.92 | 1.27 | 0.54 |
| C24 | 350 | 350 | 0.76 | 1.21 | 0.53 |
| C25 | 350 | 350 | 1.28 | 0.71 | 0.83 |
| C26 | 350 | 350 | 1.64 | 0.82 | 0.77 |
| C27 | 350 | 350 | 1.31 | 0.59 | 0.98 |
| C28 | 350 | 350 | 1.08 | 0.61 | 0.90 |
| C29 | 350 | 350 | 1.42 | 0.84 | 0.74 |
| C30 | 350 | 350 | 1.45 | 0.59 | 1.00 |
| C31 | 350 | 350 | 1.45 | 0.59 | 1.00 |
| C32 | 350 | 350 | 0.85 | 1.24 | 0.54 |
| C33 | 350 | 350 | 0.91 | 0.68 | 1.00 |
| C34 | 350 | 350 | 0.76 | 1.21 | 0.53 |
| **General degree of insurance** | | | **1.19** | **0.76** | **0.86** |

The deggree of insurance on the walls structure, in each of the two directions, resulted according to the table below:

| **Degree of insurance of poles on the structure** | |
|---|---|
| Department | Value |
| R3 | **0.86** |

| | |
|---|---|
| **Technical expert:** | eng. IOAN ROTĂRESCU |
| **Signature:** | |

# ANNEX C5 – CALCULATION BRIEF - EXISTING VARIANT – BUILDING C17

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of CR6-2013 (Annex A1), for the existing structure. The degrees of insurance for each masonry strip were established for shear force, axial force and bending moment, and a degree of insurance for each main direction of arrangement of the structural elements (longitudinal and transverse). The minimum degree of insurance was calculated for each strip and a degree of insurance, in each direction (longitudinal and transverse) and, informatively, a degree of insurance for the entire structure.

From the linear static analyses, using the response spectrum corresponding to the location, the effective values of the efforts for each wall resulted.

The ratio between the minimum effort capable in a section and the external one, represents the degree of insurance of the element (masonry strip).

The total degree of insurance resulting reflects the degree of insurance of the structure under the assumption that the floors are rigid enough to transmit horizontal loads from one strip to another, making them work together in the event of an earthquake. The slats for which the degree of insurance resulted with a value of zero are subjected to tension, an effort that they cannot take over.

# 1. MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered;*

*- the level of knowledge: KL3 – limited knowledge*

*- trust factor: CF = 1,35*

For the calculation in the linear elastic domain, considering the behavior factor q (reduced spectrum), the design strengths of the masonry for evaluating the capacity to resist bending with axial force and shear are taken as follows:

| Masonry resistances | | | | |
|---|---|---|---|---|
| Compression | | | | |
| Type | Investment | | | Standard |
| Compressive strength of the masonry element | $f_b$= | 10.00 | [N/mm2] | acc. SR EN 771 |
| Average compressive strength of mortar | $f_m$ = | 4.00 | [N/mm2] | acc. SR EN 998-2:2004 |
| K coefficient for solid ceramic bricks | K= | 0.55 | table 4.1, code CR6-2013 | |
| | constant that depends on the type of masonry element and the type of mortar | | | |
| Characteristic compressive strength | $f_k$ = | 3.80 | [N/mm2] | acc. SR EN 1052-1 |
| Trust factor | CF = | 1.35 | - | acc. 4.1 P100-3/2019 |
| Partial trust coefficient | $\gamma_M$ = | 3.00 | - | acc. D.3.3.1.2.(7) P100-3/2019 |
| Design resistance value for walls subjected to shear force | | | | |
| for horizontal joint sliding failure (fvd): | | | | |
| Resistance to sliding failure in horizontal joint | $f_{vd}$ = | f"#/$γ% · CF( | | acc. D.3.4.1.3.1.P100-3/2019 |
| Characteristic breaking strength | $f_{vk}$ = | f"#) * 0.7 · σ. | | acc. 4.3.a CR6-2013 |
| The initial characteristic unit shear strength | $f_{vk0}$ = | 0,045 | [N/mm2] | |
| for breakage in the scale under the effect of main tensile stresses. (ftd) : | | $f_{td}$ = 0,04 x $f_m$ / ($\gamma_M$ x CF) | | |
| Breakage in the scale strength | $f_{td}$ = | 0.049 | [N/mm2] | acc. D.3.4.1.3.1.P100-3/2019 |
| Longitudinal modulus of elasticity of masonry | $E_z$ = | 3800 | [N/mm2] | acc. tab. 4.9 CR6-2013 |
| Transverse modulus of elasticity of masonry | $G_z$ = | 1520 | [N/mm2] | acc. rel. 4.9 CR6-2013 |

## 1.2 Geometric features

There are no original plans of the building, nor information on the building's behavior in 20th-century earthquakes.

The compositional characteristics were determined according to the construction survey.

# 2. LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

## 2.1 Permanent loads

Permanent floor loads above ground floor

| Wooden floors | | | | |
|---|---|---|---|---|
| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m2) |
| 1 | Floor | - | - | 0.30 |
| 2 | Wooden cabinet | 0.05 | 6 | 0.30 |
| 3 | Wooden beams | - | - | 0.50 |
| 4 | Filler | 0.03 | 16 | 0.48 |
| 5 | Wooden toe board | 0.03 | 6 | 0.18 |
| 6 | Plaster | 0.02 | 19 | 0.38 |
| **Total loads** | | | | **2.14** |

Permanent loads at roof level

| Item. no. | Load name | Standard load on surface [kN/m2] |
|---|---|---|
| 1 | Cover (sheeting+cladding) | 0.30 |
| 2 | Roof framing (plywood+props+struts+clamps+rafters) | 0.40 |
| **Total loads** | | **0.70** |

Load from the weight of masonry elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of solid brick masonry elements* | - | 18.0 | 18.0 |
| Total loads | | | | 18.0 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

## 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| γ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 4 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A:
  - category A: $q_k$ = 1.50 kN/m$^2$
- for building attics:
  - non-passable attics : $q_k$ = 0.75 kN/m$^2$

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of “modal analysis with response spectrum” using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used

– using the combinations below:

$$E_{Edx} \text{ " + " } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ " + " } E_{Edy}$$

where:

$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction ($F_{xi}$ si $F_{yi}$) and the corresponding accidental eccentricity ($e_{xi}$ si $e_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i

$F_{xi}$ – static equivalent horizontal seismic force applied at level i

The accidental eccentricity is calculated with the expression:

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table no. 13 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j\geq1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j\geq1} G_{k,j} + P + A_{ED} + \sum_{i\geq1} \Psi_{2,i} Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j\geq1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$ |
| | Frequent grouping | $\sum_{j\geq1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |

| | | |
|---|---|---|
| | Quasi-permanent grouping | $\sum_{j\geq1} G_{k,j} + P + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004.

*Table no. 14 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3. LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

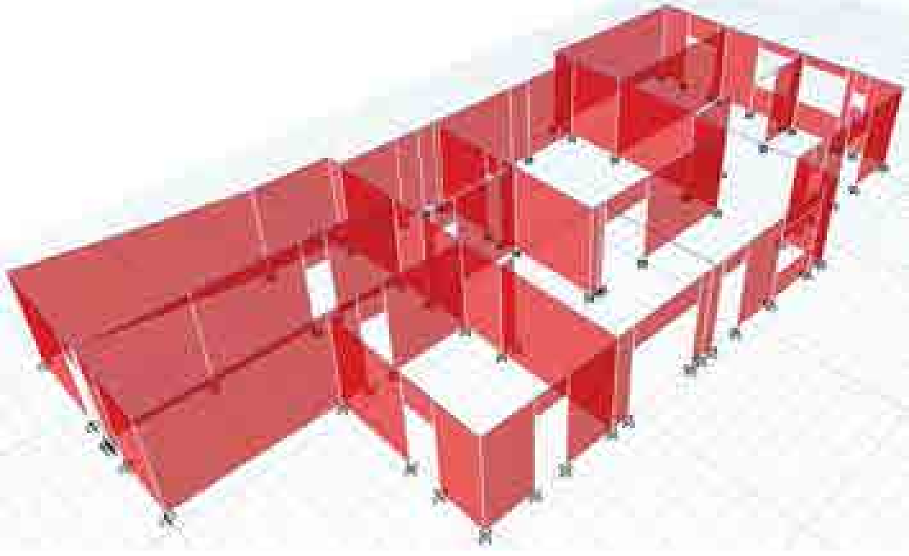


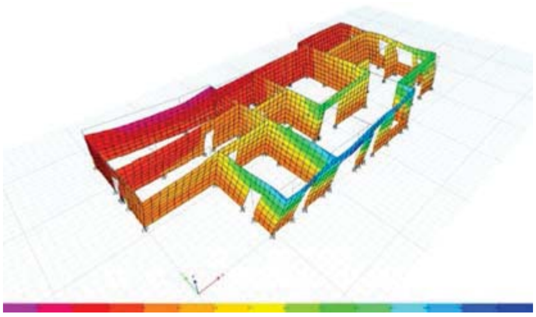
*Figure no. 5 - Calculation model – Spatial modeling – existing situation*

## 3.2 Analysis of natural vibration modes

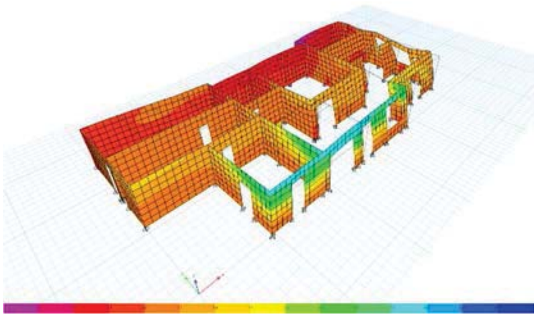
In the calculation, 12 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

*Table no. 15 -*

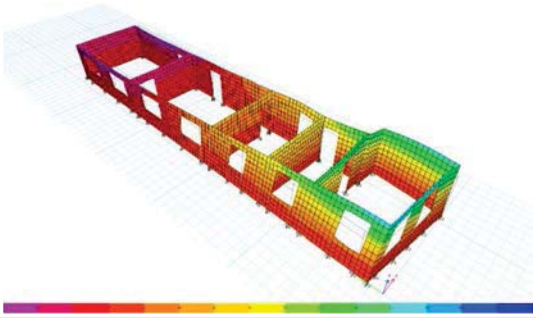
| Case | Mode | Period | Ux | Uy | Sum UX | Sum UY |
|---|---|---|---|---|---|---|
| Modal | 1 | 0,052 | 0,004 | 0,835 | 0.0044 | 0.8352 |
| Modal | 2 | 0,048 | 0,064 | 0,143 | 0.0681 | 0.9781 |
| Modal | 3 | 0.04 | 0,856 | 0,002 | 0.9239 | 0.9805 |
| Modal | 4 | 0,034 | 0,001 | 0,005 | 0.9244 | 0.9854 |
| Modal | 5 | 0,028 | 0,003 | 0,005 | 0.9272 | 0.9902 |
| Modal | 6 | 0,026 | 0,055 | 0,001 | 0.9819 | 0.9911 |
| Modal | 7 | 0,025 | 0,003 | 0,000 | 0.9847 | 0.9911 |
| Modal | 8 | 0,023 | 0,003 | 0,002 | 0.9876 | 0.9929 |
| Modal | 9 | 0,021 | 0,001 | 0,000 | 0.9886 | 0.9929 |
| Modal | 10 | 0.02 | 0,000 | 0,000 | 0.9888 | 0.9929 |
| Modal | 11 | 0.02 | 0,001 | 0,001 | 0.9902 | 0.9937 |
| Modal | 12 | 0,019 | 0,000 | 0,000 | 0.9903 | 0.9939 |



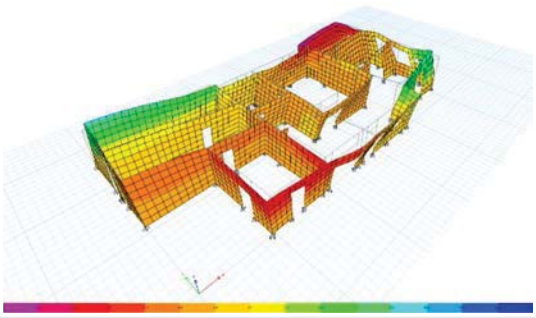
*Vibration mode 1 (3D view)*



*Vibration mode 2 (3D view)*



*Vibration mode 3 (3D view)*



*Vibration mode 4 (3D view)*

## 3.3 Analysis of forces in the slats

### 3.3.1 Calculation scheme - slats numbering

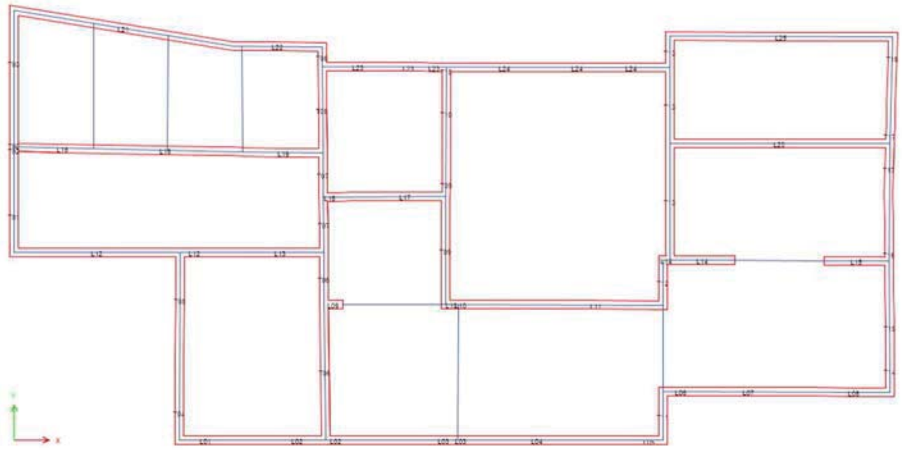


*Figure no. 3 - Ground floor beams – X; Y direction*

# 4. DEGREE OF STRUCTURE INSURANCE – QUANTITATIVE EVALUATION

## 4.1 Seismic assessment for the effects of in-plane wall action

The capable efforts in the beams were calculated with the relations from the normative P100-3/2019 Annex D.

### 4.1.1 Determining the degree of insurance for the entire fall, on the slats in each direction

$$R_{3i} = \frac{V_{cap,i}}{F_{b,i}}$$

$V_{cap,i}$ - is the shear force capable of wall "i" (the smaller of $V_{fd}$ and $V_{ff}$).

*Table no. 6 - Degree of insurance on the shoulder straps in the longitudinal and transverse direction*

| Degree of insurance on the slats in the longitudinal direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| L01 | 0.25 | 1.58 | -52.82 | 12.06 | 11.73 | 6.91 | 24.82 | F | 6.91 | 0.00 | 0.57 |
| L02 | 0.25 | 2.29 | -99.03 | 27.41 | 30.59 | 12.96 | 43.95 | F | 12.96 | 0.00 | 0.47 |
| L03 | 0.25 | 0.97 | -78.29 | 11.49 | 8.57 | 10.25 | 21.82 | D | 8.57 | 0.00 | 0.75 |
| L04 | 0.25 | 1.75 | -93.16 | 7.89 | 21.08 | 12.19 | 33.01 | F | 12.19 | 0.00 | > 1.0 |
| L05 | 0.25 | 0.82 | -30.11 | 2.65 | 3.43 | 3.94 | 13.36 | D | 3.43 | 0.00 | > 1.0 |
| L06 | 0.25 | 1.06 | -39.45 | 10.83 | 5.82 | 5.16 | 17.38 | F | 5.16 | 0.00 | 0.48 |
| L07 | 0.25 | 1.28 | -58.53 | 18.56 | 10.02 | 7.66 | 22.71 | F | 7.66 | 0.00 | 0.41 |
| L08 | 0.25 | 2.10 | -80.25 | 47.01 | 23.29 | 10.50 | 35.32 | F | 10.50 | 0.00 | 0.22 |
| L09 | 0.25 | 0.55 | -22.75 | 0.67 | 1.69 | 2.98 | 9.34 | D | 1.69 | 0.00 | > 1.0 |
| L10 | 0.25 | 0.55 | -50.61 | 0.90 | 2.90 | 6.62 | 13.02 | D | 2.90 | 0.00 | > 1.0 |
| L11 | 0.25 | 3.92 | -211.39 | 78.52 | 106.34 | 56.73 | 111.26 | F | 56.73 | 0.00 | 0.72 |
| L12 | 0.25 | 5.71 | -192.93 | 64.06 | 155.10 | 64.31 | 135.26 | F | 64.31 | 0.00 | > 1.0 |
| L13 | 0.25 | 2.56 | -111.29 | 48.56 | 38.50 | 14.56 | 55.25 | F | 14.56 | 0.00 | 0.30 |
| L14 | 0.25 | 2.11 | -109.43 | 28.98 | 29.93 | 14.32 | 40.10 | F | 14.32 | 0.00 | 0.49 |
| L15 | 0.25 | 1.91 | -95.29 | 34.93 | 23.80 | 12.47 | 35.02 | F | 12.47 | 0.00 | 0.36 |
| L16 | 0.25 | 0.40 | -14.24 | 2.07 | 0.80 | 1.86 | 6.47 | D | 0.80 | 0.00 | 0.39 |
| L17 | 0.25 | 2.38 | -109.80 | 10.74 | 34.88 | 14.37 | 48.86 | F | 14.37 | 0.00 | > 1.0 |
| L18 | 0.25 | 6.02 | -308.98 | 68.98 | 241.74 | 92.72 | 167.50 | F | 92.72 | 0.00 | > 1.0 |
| L19 | 0.25 | 2.28 | -100.45 | 44.66 | 30.86 | 13.15 | 44.04 | F | 13.15 | 0.00 | 0.29 |
| L20 | 0.25 | 6.43 | -282.16 | 110.65 | 244.38 | 88.88 | 168.23 | F | 88.88 | 0.00 | 0.80 |
| L21 | 0.25 | 6.50 | -224.28 | 66.09 | 204.64 | 75.95 | 155.18 | F | 75.95 | 0.00 | > 1.0 |
| L22 | 0.25 | 2.61 | -81.95 | 52.50 | 30.41 | 10.72 | 50.66 | F | 10.72 | 0.00 | 0.20 |
| L23 | 0.25 | 3.63 | -106.12 | 49.68 | 55.27 | 30.00 | 81.59 | F | 30.00 | 0.00 | 0.60 |
| L24 | 0.25 | 6.55 | -226.41 | 109.16 | 207.91 | 76.69 | 156.39 | F | 76.69 | 0.00 | 0.70 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| L25 | 0.25 | 6.52 | -195.18 | 70.07 | 182.22 | 69.38 | 147.87 | F | 69.38 | | |

| Degree of insurance on the slats in the transverse direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| T01 | 0.25 | 2.12 | -63.29 | 18.33 | 38.45 | 8.28 | 32.88 | F | 8.28 | 0.00 | 0.45 |
| T02 | 0.25 | 0.30 | -13.66 | 0.33 | 1.09 | 1.79 | 5.31 | D | 1.09 | 0.00 | > 1.0 |
| T03 | 0.25 | 3.10 | -96.51 | 40.64 | 85.14 | 12.63 | 71.23 | F | 12.63 | 0.00 | 0.31 |
| T04 | 0.25 | 1.58 | -56.17 | 7.26 | 24.80 | 7.35 | 25.43 | F | 7.35 | 0.00 | > 1.0 |
| T05 | 0.25 | 2.82 | -115.11 | 31.03 | 88.75 | 15.06 | 65.39 | F | 15.06 | 0.00 | 0.49 |
| T06 | 0.25 | 5.51 | -247.51 | 85.34 | 365.22 | 75.67 | 145.40 | F | 75.67 | 0.00 | 0.89 |
| T07 | 0.25 | 2.90 | -111.02 | 41.57 | 88.92 | 14.53 | 67.38 | F | 14.53 | 0.00 | 0.35 |
| T08 | 0.25 | 3.12 | -104.30 | 42.88 | 91.83 | 13.65 | 73.64 | F | 13.65 | 0.00 | 0.32 |
| T09 | 0.25 | 3.84 | -268.80 | 59.64 | 245.45 | 69.33 | 121.67 | F | 69.33 | 0.00 | > 1.0 |
| T10 | 0.25 | 2.49 | -110.29 | 38.22 | 73.81 | 14.43 | 52.52 | F | 14.43 | 0.00 | 0.38 |
| T11 | 0.25 | 1.45 | -62.15 | 4.85 | 24.32 | 8.13 | 25.02 | F | 8.13 | 0.00 | > 1.0 |
| T12 | 0.25 | 1.33 | -73.23 | 15.15 | 24.88 | 9.58 | 25.39 | F | 9.58 | 0.00 | 0.63 |
| T13 | 0.25 | 6.55 | -288.13 | 133.92 | 508.23 | 90.91 | 171.55 | F | 90.91 | 0.00 | 0.68 |
| T14 | 0.25 | 1.25 | -44.56 | 14.81 | 15.49 | 5.83 | 20.09 | F | 5.83 | 0.00 | 0.39 |
| T15 | 0.25 | 0.32 | -7.26 | 3.96 | 0.69 | 0.95 | 4.41 | D | 0.69 | 0.00 | 0.17 |
| T16 | 0.25 | 0.43 | -14.60 | 2.58 | 1.79 | 1.91 | 6.84 | D | 1.79 | 0.00 | 0.69 |
| T17 | 0.25 | 1.98 | -70.77 | 21.17 | 39.14 | 9.26 | 31.93 | F | 9.26 | 0.00 | 0.44 |
| T18 | 0.25 | 1.23 | -37.88 | 17.38 | 13.34 | 4.96 | 18.82 | F | 4.96 | 0.00 | 0.29 |

*Table no. 7 - Degree of insurance by structure*

| Direction | Investment |
|---|---|
| Ground floor - Longitudinal | **0.61** |
| Ground floor - Transversal | **0.45** |

| | |
|---|---|
| **Technical expert:** | eng. IOAN ROTĂRESCU |
| **Signature:** | |

# ANNEX C6 – CALCULATION SUMMARY - EXISTING VARIANT – BUILDING C20

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of CR6-2013 (Annex A1), for the existing structure. The degrees of insurance for each masonry strip were established for shear force, axial force and bending moment, and a degree of insurance for each main direction of arrangement of the structural elements (longitudinal and transverse). The minimum degree of insurance was calculated for each strip and a degree of insurance, in each direction (longitudinal and transverse) and, informatively, a degree of insurance for the entire structure.

From the linear static analyses, using the response spectrum corresponding to the location, the effective values of the efforts for each wall resulted.

The ratio between the minimum effort capable in a section and the external one, represents the degree of insurance of the element (masonry strip).

The total degree of insurance resulting reflects the degree of insurance of the structure under the assumption that the floors are rigid enough to transmit horizontal loads from one strip to another, making them work together in the event of an earthquake. The slats for which the degree of insurance resulted with a value of zero are subjected to tension, an effort that they cannot take over.

# 1. MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered;*

*- the level of knowledge: KL3 – limited knowledge*

*- trust factor: CF = 1,35*

For the calculation in the linear elastic domain, considering the behavior factor q (reduced spectrum), the design strengths of the masonry for evaluating the capacity to resist bending with axial force and shear are taken as follows:

| Masonry resistances | | | | |
|---|---|---|---|---|
| Compression | | | | |
| Type | Investment | | | Standard |
| Compressive strength of the masonry element | $f_b$= | 10.00 | [N/mm2] | acc. SR EN 771 |
| Average compressive strength of mortar | $f_m$ = | 4.00 | [N/mm2] | acc. SR EN 998-2:2004 |
| K coefficient for solid ceramic bricks | K= | 0.55 | table 4.1, code CR6-2013 | |
| | constant that depends on the type of masonry element and the type of mortar | | | |
| Characteristic compressive strength | $f_k$ = | 3.80 | [N/mm2] | acc. SR EN 1052-1 |
| Trust factor | CF = | 1.35 | - | acc. 4.1 P100-3/2019 |
| Partial trust coefficient | $\gamma_M$ = | 3.00 | - | acc. D.3.3.1.2.(7) P100-3/2019 |
| Design resistance value for walls subjected to shear force | | | | |
| for horizontal joint sliding failure (fvd): | | | | |
| Resistance to sliding failure in horizontal joint | $f_{vd}$ = | $f_{vk}/(\gamma_M \cdot CF)$ | | acc. D.3.4.1.3.1.P100-3/2019 |
| Characteristic breaking strength | $f_{vk}$ = | $f_{vk0} + 0.7 \cdot \sigma_d$ | | acc. 4.3.a CR6-2013 |
| The initial characteristic unit shear strength | $f_{vk0}$ = | 0,045 | [N/mm2] | |
| for breakage in the scale under the effect of main tensile stresses. (ftd) : | | $f_{td} = 0{,}04 \times f_m / (\gamma_M \times CF)$ | | |
| Breakage in the scale strength | $f_{td}$ = | 0.049 | [N/mm2] | acc. D.3.4.1.3.1.P100-3/2019 |
| Longitudinal modulus of elasticity of masonry | $E_z$ = | 3800 | [N/mm2] | acc. tab. 4.9 CR6-2013 |
| Transverse modulus of elasticity of masonry | $G_z$ = | 1520 | [N/mm2] | acc. rel. 4.9 CR6-2013 |

## 1.2 Geometric features

There are no original plans of the building, nor information on the building's behavior in 20th-century earthquakes.

The compositional characteristics were determined according to the construction survey.

## 2. LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

### 2.1 Permanent loads

Permanent floor loads above ground floor

| Wooden floors | | | | |
|---|---|---|---|---|
| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m2) |
| 1 | Floor | - | - | 0.30 |
| 2 | Wooden cabinet | 0.05 | 6 | 0.30 |
| 3 | Wooden beams | - | - | 0.50 |
| 4 | Filler | 0.03 | 16 | 0.48 |
| 5 | Wooden toe board | 0.03 | 6 | 0.18 |
| 6 | Plaster | 0.02 | 19 | 0.38 |
| **Total loads** | | | | **2.14** |

Permanent loads at roof level

| Item. no. | Load name | Standard load on surface [kN/m2] |
|---|---|---|
| 1 | Cover (sheeting+cladding) | 0.30 |
| 2 | Roof framing (plywood+props+struts+clamps+rafters) | 0.40 |
| **Total loads** | | **0.70** |

Load from the weight of masonry elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of solid brick masonry elements* | - | 18.0 | 18.0 |
| Total loads | | | | 18.0 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

### 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| γ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 1 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A ([]):
  - category A: $q_k$ = 1.50 kN/m$^2$
- for building attics:
  - non-passable attics : $q_k$ = 0.75 kN/m$^2$

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of "modal analysis with response spectrum" using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used
– using the combinations below:

$$E_{Edx} \text{ "+" } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ "+" } E_{Edy}$$

where:
$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction ($F_{xi}$ si $F_{yi}$) and the corresponding accidental eccentricity ($e_{xi}$ si $e_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i
$F_{xi}$ – static equivalent horizontal seismic force applied at level i

The accidental eccentricity is calculated with the expression: $e_0$

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table no. 16 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j\geq1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j\geq1} G_{k,j} + P + A_{ED} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j\geq1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$ |
| | Frequent grouping | $\sum_{j\geq1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |

| | | |
|---|---|---|
| | Quasi-permanent grouping | $\sum_{j\geq1} G_{k,j} + P + \sum_{i\geq1} \Psi_{2,i} Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004. *Table no. 17 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3. LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

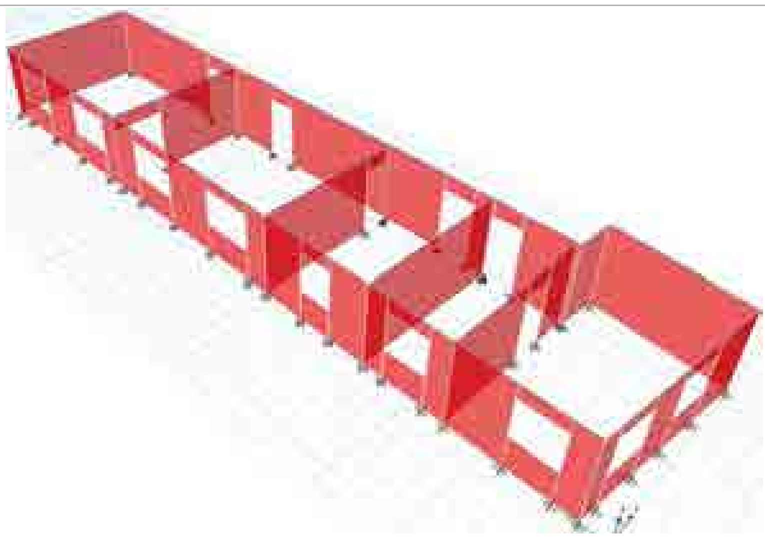


*Figure no. 2 - Calculation model – Spatial modeling – existing situation*

## 3.2 Analysis of natural vibration modes

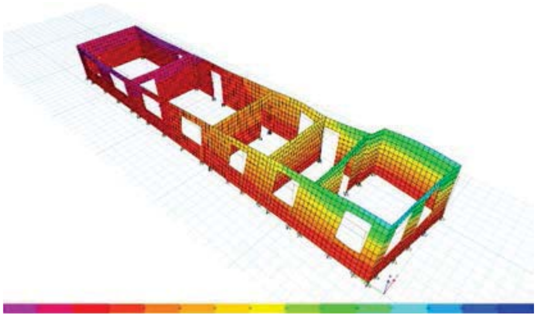
In the calculation, 12 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

*Table no. 18 -*

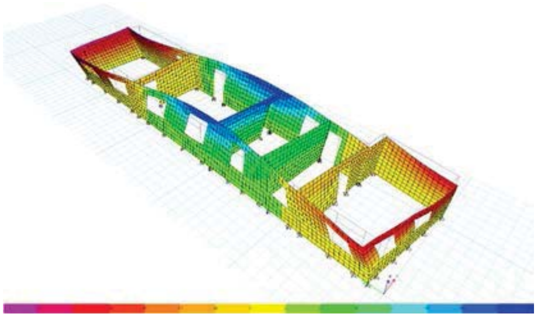
| Case | Mode | Period | Ux | Uy | Sum UX | Sum UY |
|---|---|---|---|---|---|---|
| Modal | 1 | 0,044 | 0,705 | 0,000 | 0.7053 | 0 |
| Modal | 2 | 0,035 | 0,021 | 0,877 | 0.7258 | 0.8766 |
| Modal | 3 | 0,034 | 0,213 | 0,108 | 0.9391 | 0.9845 |
| Modal | 4 | 0,028 | 0,027 | 0,002 | 0.9663 | 0.9866 |
| Modal | 5 | 0,022 | 0,005 | 0,000 | 0.9709 | 0,987 |
| Modal | 6 | 0,021 | 0,025 | 0,006 | 0.9953 | 0.9929 |
| Modal | 7 | 0,017 | 0,001 | 0,000 | 0.9962 | 0.9932 |
| Modal | 8 | 0,015 | 0,000 | 0,002 | 0.9963 | 0.9952 |
| Modal | 9 | 0,013 | 0,000 | 0,001 | 0.9963 | 0.9958 |
| Modal | 10 | 0,013 | 0,001 | 0,000 | 0.9968 | 0.9958 |
| Modal | 11 | 0,012 | 0,001 | 0,000 | 0.9974 | 0.9958 |
| Modal | 12 | 0,011 | 0,000 | 0,000 | 0.9974 | 0.9961 |

*Vibration mode 1 (3D view)*

*Vibration mode 2 (3D view)*



*Vibration mode 3 (3D view)*



*Vibration mode 4 (3D view)*

## 3.3 Analysis of forces in the slats

### 3.3.1 Calculation scheme - slats numbering

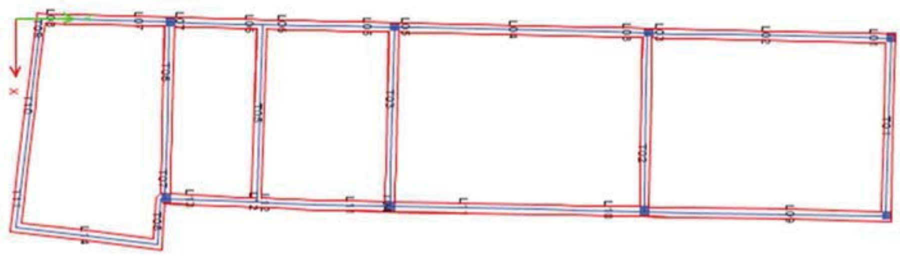


*Figure no. 4 - Ground floor beams – X; Y direction*

# 4. DEGREE OF STRUCTURE INSURANCE – QUANTITATIVE EVALUATION

## 4.1 Seismic assessment for the effects of in-plane wall action

The capable efforts in the beams were calculated with the relations from the normative P100-3/2019 Annex D.

### 4.1.1 Determining the degree of insurance for the entire building, on the slats in each direction

$$R_{3i} = \frac{V_{cap,i}}{F_{b,i}}$$

$V_{cap,i}$ - is the shear force capable of wall "i" (the smaller of $V_{fd}$ and $V_{ff}$).

*Table no. 8 - Degree of insurance on the shoulder straps in the longitudinal and transverse direction*

| Degree of insurance on the slats in the longitudinal direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| L01 | 0.25 | 0.77 | -10.99 | 7.76 | 1.60 | 1.44 | 9.21 | F | 1.44 | 0.00 | 0.19 |
| L02 | 0.25 | 1.65 | -68.78 | 9.52 | 19.13 | 9.00 | 28.20 | F | 9.00 | 0.00 | 0.95 |
| L03 | 0.25 | 1.71 | -33.28 | 10.11 | 10.54 | 4.35 | 22.97 | F | 4.35 | 0.00 | 0.43 |
| L04 | 0.25 | 1.70 | -70.63 | 8.25 | 20.25 | 9.24 | 29.60 | F | 9.24 | 0.00 | > 1.0 |
| L05 | 0.25 | 2.00 | -39.51 | 12.66 | 14.63 | 5.17 | 31.59 | F | 5.17 | 0.00 | 0.41 |
| L06 | 0.25 | 1.74 | -43.00 | 12.04 | 13.53 | 5.63 | 25.58 | F | 5.63 | 0.00 | 0.47 |
| L07 | 0.25 | 2.15 | -46.59 | 20.26 | 18.40 | 6.10 | 37.58 | F | 6.10 | 0.00 | 0.30 |
| L08 | 0.25 | 0.62 | -17.05 | 2.67 | 1.91 | 2.23 | 9.13 | D | 1.91 | 0.00 | 0.71 |
| L09 | 0.25 | 4.91 | -151.00 | 64.88 | 131.13 | 49.87 | 112.41 | F | 49.87 | 0.00 | 0.77 |
| L10 | 0.25 | 1.76 | -40.66 | 23.35 | 13.06 | 5.32 | 25.70 | F | 5.32 | 0.00 | 0.23 |
| L11 | 0.25 | 5.89 | -171.59 | 74.90 | 179.85 | 60.41 | 132.22 | F | 60.41 | 0.00 | 0.81 |
| L12 | 0.25 | 0.60 | -20.52 | 3.43 | 2.14 | 2.69 | 9.52 | D | 2.14 | 0.00 | 0.63 |
| L13 | 0.25 | 1.44 | -32.80 | 15.98 | 8.60 | 4.29 | 19.78 | F | 4.29 | 0.00 | 0.27 |
| L14 | 0.25 | 3.72 | -95.57 | 40.15 | 64.21 | 28.41 | 79.97 | F | 28.41 | 0.00 | 0.71 |

| Degree of insurance on the slats in the transverse direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| T01 | 0.25 | 4.63 | -108.62 | 2.24 | 183.30 | 38.30 | 96.54 | F | 38.30 | 38.30 | 0.00 |
| T02 | 0.25 | 3.00 | -151.39 | 10.90 | 147.12 | 19.81 | 82.94 | F | 19.81 | 19.81 | 0.00 |
| T03 | 0.25 | 3.77 | -151.18 | 4.15 | 193.76 | 41.69 | 95.28 | F | 41.69 | 41.69 | 0.00 |
| T04 | 0.25 | 0.14 | -1.03 | 0.08 | 0.06 | 0.14 | 1.47 | D | 0.06 | 0.06 | 0.00 |
| T05 | 0.25 | 4.64 | -162.34 | 4.54 | 261.63 | 50.74 | 111.34 | F | 50.74 | 50.74 | 0.00 |
| T06 | 0.25 | 2.67 | -94.42 | 3.71 | 87.38 | 12.36 | 64.30 | F | 12.36 | 12.36 | 0.00 |
| T07 | 0.25 | 1.03 | -25.33 | 2.44 | 9.46 | 3.31 | 14.54 | F | 3.31 | 3.31 | 0.00 |
| T08 | 0.25 | 1.20 | -27.42 | 5.55 | 12.04 | 3.59 | 16.55 | F | 3.59 | 3.59 | 0.00 |
| T09 | 0.25 | 0.56 | -15.84 | 0.60 | 3.19 | 2.07 | 8.33 | F | 2.07 | 2.07 | 0.00 |
| T10 | 0.25 | 1.05 | -37.77 | 0.49 | 13.71 | 4.94 | 16.97 | F | 4.94 | 4.94 | 0.00 |
| T11 | 0.25 | 1.46 | -40.03 | 4.51 | 20.89 | 5.24 | 21.36 | F | 5.24 | 5.24 | 0.00 |

*Table no. 9 - Degree of insurance by structure*

| Direction | Investment |
|---|---|
| Ground floor - Longitudinal | **0.53** |
| Ground floor - Transversal | **1.00** |

| **Technical expert:** | eng. IOAN ROTĂRESCU |
|---|---|
| **Signature:** | |

# ANNEX C7 – CALCULATION SUMMARY- EXISTING VARIANT – BUILDING C24, C26, C27

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of CR6-2013 (Annex A1), for the existing structure. The degrees of insurance for each masonry strip were established for shear force, axial force and bending moment, and a degree of insurance for each main direction of arrangement of the structural elements (longitudinal and transverse). The minimum degree of insurance was calculated for each strip and a degree of insurance, in each direction (longitudinal and transverse) and, informatively, a degree of insurance for the entire structure.

From the linear static analyses, using the response spectrum corresponding to the location, the effective values of the efforts for each wall resulted.

The ratio between the minimum effort capable in a section and the external one, represents the degree of insurance of the element (masonry strip).

The total degree of insurance resulting reflects the degree of insurance of the structure under the assumption that the floors are rigid enough to transmit horizontal loads from one strip to another, making them work together in the event of an earthquake. The slats for which the degree of insurance resulted with a value of zero are subjected to tension, an effort that they cannot take over.

# 1. MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered; - the level of knowledge: KL3 – limited knowledge*

*- trust factor: CF = 1,35*

For the calculation in the linear elastic domain, considering the behavior factor q (reduced spectrum), the design strengths of the masonry for evaluating the capacity to resist bending with axial force and shear are taken as follows:

| Masonry resistances | | | | |
|---|---|---|---|---|
| Compression | | | | |
| Type | Investment | | | Standard |
| Compressive strength of the masonry element | $f_b$= | 10.00 | [N/mm2] | acc. SR EN 771 |
| Average compressive strength of mortar | $f_m$ = | 4.00 | [N/mm2] | acc. SR EN 998-2:2004 |
| K coefficient for solid ceramic bricks | K= | 0.55 | table 4.1, code CR6-2013 | |
| | constant that depends on the type of masonry element and the type of mortar | | | |
| Characteristic compressive strength | $f_k$ = | 3.80 | [N/mm2] | acc. SR EN 1052-1 |
| Trust factor | CF = | 1.35 | - | acc. 4.1 P100-3/2019 |
| Partial trust coefficient | $\gamma_M$ = | 3.00 | - | acc. D.3.3.1.2.(7) P100-3/2019 |
| Design resistance value for walls subjected to shear force | | | | |
| for horizontal joint sliding failure (fvd): | | | | |
| Resistance to sliding failure in horizontal joint | $f_{vd}$ = | $f_{vk}/(\gamma_M \cdot CF)$ | | acc. D.3.4.1.3.1.P100-3/2019 |
| Characteristic breaking strength | $f_{vk}$ = | $f_{vko} + 0.7 \cdot \sigma_d$ | | acc. 4.3.a CR6-2013 |
| The initial characteristic unit shear strength | $f_{vk0}$ = | 0,045 | [N/mm2] | |
| for breakage in the scale under the effect of main tensile stresses. (ftd) : | $f_{td}$ = 0,04 x $f_m$ / ($\gamma_M$ x CF) | | | |
| Breakage in the scale strength | $f_{td}$ = | 0.049 | [N/mm2] | acc. D.3.4.1.3.1.P100-3/2019 |
| Longitudinal modulus of elasticity of masonry | $E_z$ = | 3800 | [N/mm2] | acc. tab. 4.9 CR6-2013 |
| Transverse modulus of elasticity of masonry | $G_z$ = | 1520 | [N/mm2] | acc. rel. 4.9 CR6-2013 |

## 1.2 Geometric features

There are no original plans of the building, nor information on the building's behavior in 20th-century earthquakes.

The compositional characteristics were determined according to the construction survey.

## 2. LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

### 2.1 Permanent loads

Permanent floor loads above ground floor

| Wooden floors | | | | |
|---|---|---|---|---|
| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m2) |
| 1 | Floor | - | - | 0.30 |
| 2 | Wooden cabinet | 0.05 | 6 | 0.30 |
| 3 | Wooden beams | - | - | 0.50 |
| 4 | Filler | 0.03 | 16 | 0.48 |
| 5 | Wooden toe board | 0.03 | 6 | 0.18 |
| 6 | Plaster | 0.02 | 19 | 0.38 |
| **Total loads** | | | | **2.14** |

Permanent loads at roof level

| Item. no. | Load name | Standard load on surface [kN/m2] |
|---|---|---|
| 1 | Cover (sheeting+cladding) | 0.30 |
| 2 | Roof framing (plywood+props+struts+clamps+rafters) | 0.40 |
| **Total loads** | | **0.70** |

Load from the weight of masonry elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of solid brick masonry elements* | - | 18.0 | 18.0 |
| Total loads | | | | 18.0 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

### 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| γ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 3 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A:
    - category A: $q_k$ = 1.50 kN/m$^2$
- for building attics:
    - non-passable attics : $q_k$ = 0.75 kN/m$^2$

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of "modal analysis with response spectrum" using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used

– using the combinations below:

$$E_{Edx} \text{ "+" } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ "+" } E_{Edy}$$

where:

$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction ($F_{xi}$ si $F_{yi}$) and the corresponding accidental eccentricity ($e_{xi}$ si $e_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i

$F_{xi}$ – static equivalent horizontal seismic force applied at level i

The accidental eccentricity is calculated with the expression: $e_0$

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table no. 19 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j>1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j>1} G_{k,j} + P + A_{ED} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j>1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$ |
| | Frequent grouping | $\sum_{j>1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |

| | | |
|---|---|---|
| | Quasi-permanent grouping | $\sum_{j\geq 1} G_{k,j} + P + \sum_{i>1} \psi_{2,i} Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004.

*Table no. 20 -*

| | Surname | type | Permanent | Useful – | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3. LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

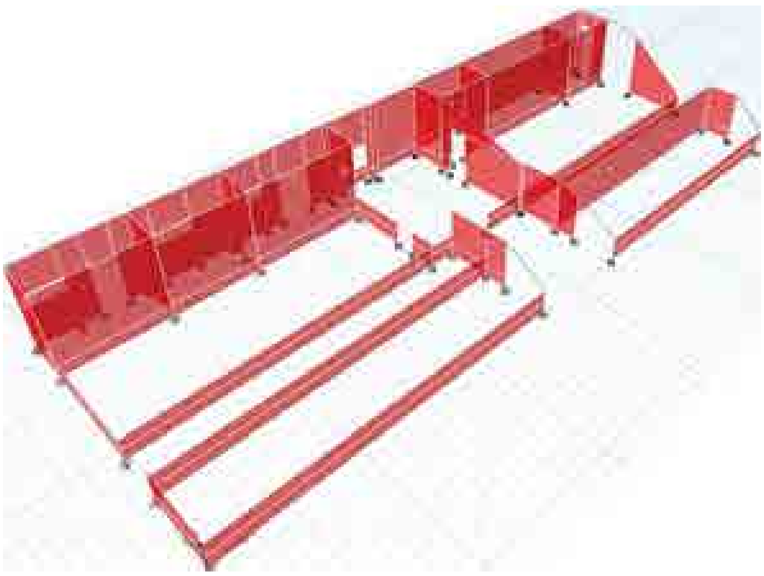


*Figure no. 4 - Calculation model – Spatial modeling – existing situation*

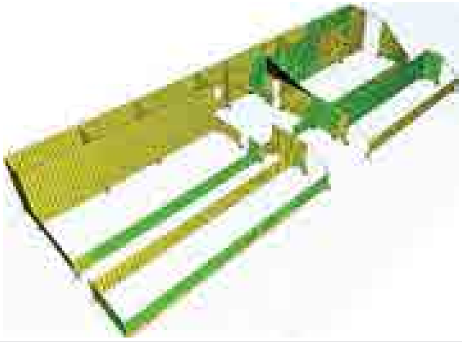
## 3.2 Analysis of natural vibration modes

In the calculation, 12 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

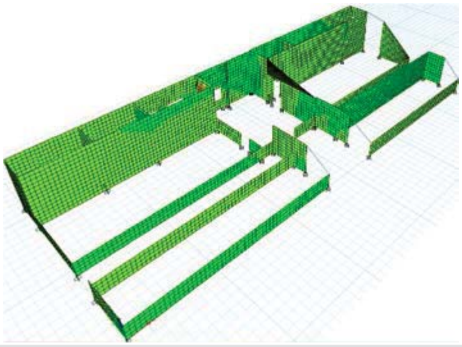
*Table no. 21 -*

| Case | Mode | Period | Ux | Uy | Sum UX | Sum UY |
|---|---|---|---|---|---|---|
| Modal | 1 | 0.021 | 0 | 0.999 | 0.0000 | 0.0393 |
| Modal | 2 | 0.02 | 0 | 0.002 | 0.0001 | 0.041 |
| Modal | 3 | 0.02 | 0 | 0.585 | 0.0001 | 0.0924 |
| Modal | 4 | 0.019 | 0 | 0.002 | 0.0001 | 0.1099 |
| Modal | 5 | 0.019 | 0 | 0.006 | 0.0001 | 0.1231 |
| Modal | 6 | 0.018 | 0 | 0.007 | 0.0014 | 0.1625 |
| Modal | 7 | 0.018 | 0 | 0.028 | 0.0014 | 0.165 |
| Modal | 8 | 0.018 | 0.008 | 0.007 | 0.0111 | 0.1736 |
| Modal | 9 | 0.017 | 0 | 0.001 | 0.0112 | 0.1741 |
| Modal | 10 | 0.017 | 0.001 | 0.877 | 0.0114 | 0.3324 |
| Modal | 11 | 0.017 | 0 | 0.905 | 0.0114 | 0.39 |
| Modal | 12 | 0.017 | 0 | 0.316 | 0.0114 | 0.4241 |

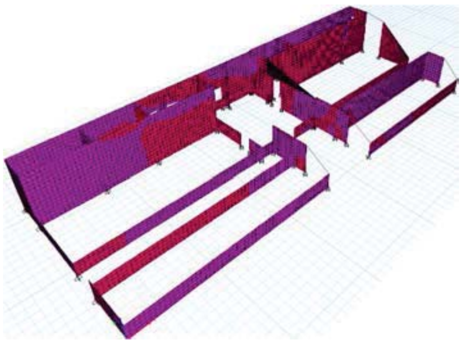
*Vibration mode 1 (3D view)*



*Vibration mode 2 (3D view)*



*Vibration mode 1 (3D view)*



*Vibration mode 2 (3D view)*

## 3.3 Analysis of forces in the slats

### 3.3.1 Calculation scheme - slats numbering

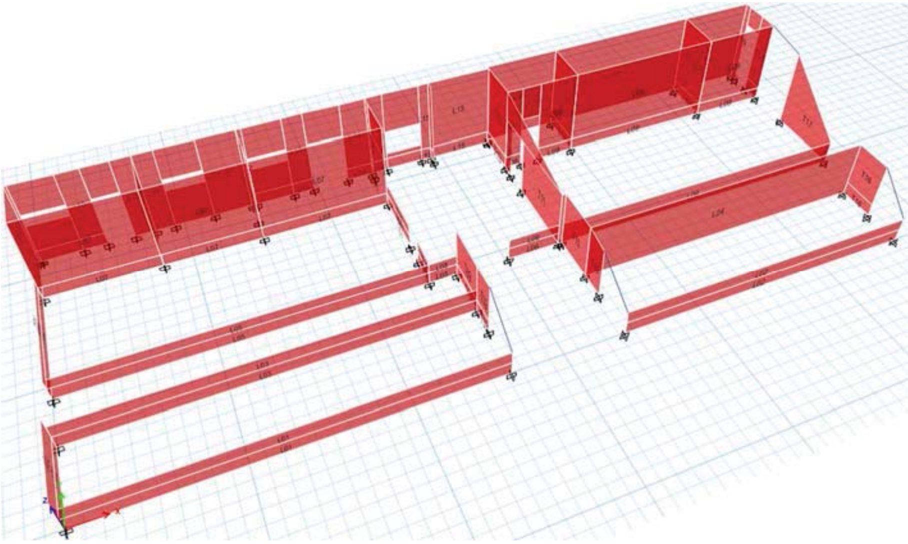


*Figure no. 5 - Ground floor beams – X; Y direction*

# 4. DEGREE OF STRUCTURE INSURANCE – QUANTITATIVE EVALUATION

## 4.1 Seismic assessment for the effects of in-plane wall action

Eforturile capabile din șpaleți au fost calculate cu relațiile din normativul *P100-3/2019 Annex D*.

### 4.1.1 Determining the degree of insurance for the entire building, on the slats in each direction

$$R_{3i} = \frac{V_{cap,i}}{F_{b,i}}$$

$V_{cap,i}$ - is the shear force capable of wall "i" (the smaller of $V_{fd}$ and $V_{ff}$).

*Table no. 10 - Degree of insurance on the slats in the longitudinal and transverse direction*

| Degree of insurance on the slats in the longitudinal direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| L01 | 0.25 | 13.19 | -41.74 | 1.81 | 271.96 | 68.41 | 180.53 | F | 68.41 | 0.00 | > 1.0 |
| L02 | 0.25 | 11.02 | -39.95 | 0.22 | 217.00 | 57.05 | 153.03 | F | 57.05 | 0.00 | > 1.0 |
| L03 | 0.25 | 13.19 | -38.37 | 3.80 | 250.21 | 67.63 | 179.02 | F | 67.63 | 0.00 | > 1.0 |
| L04 | 0.25 | 11.00 | -140.72 | 39.14 | 320.11 | 80.12 | 191.98 | F | 80.12 | 0.00 | > 1.0 |
| L05 | 0.25 | 13.19 | -30.01 | 0.66 | 217.98 | 65.71 | 175.22 | F | 65.71 | 0.00 | > 1.0 |
| L06 | 0.25 | 13.62 | -60.10 | 19.72 | 447.04 | 74.77 | 193.81 | F | 74.77 | 0.00 | > 1.0 |
| L07 | 0.25 | 12.17 | -247.33 | 140.63 | 397.08 | 110.56 | 242.40 | F | 110.56 | 0.00 | 0.79 |
| L08 | 0.25 | 0.64 | -13.75 | 10.26 | 1.16 | 1.80 | 8.65 | D | 1.16 | 0.00 | 0.11 |
| L09 | 0.25 | 10.30 | -202.72 | 116.98 | 276.29 | 90.86 | 203.17 | F | 90.86 | 0.00 | 0.78 |
| L10 | 0.25 | 0.81 | -18.70 | 27.22 | 1.98 | 2.45 | 11.20 | D | 1.98 | 0.00 | 0.07 |
| L11 | 0.25 | 1.41 | -34.29 | 14.90 | 6.27 | 4.49 | 19.84 | F | 4.49 | 0.00 | 0.30 |
| L12 | 0.25 | 1.57 | -35.62 | 8.38 | 7.31 | 4.66 | 21.59 | F | 4.66 | 0.00 | 0.56 |
| L13 | 0.25 | 0.76 | -19.40 | 19.34 | 1.90 | 2.54 | 10.87 | D | 1.90 | 0.00 | 0.10 |
| L14 | 0.25 | 1.60 | -46.00 | 6.24 | 9.37 | 6.02 | 23.83 | F | 6.02 | 0.00 | 0.97 |
| L15 | 0.25 | 2.84 | -66.77 | 49.15 | 24.72 | 8.74 | 48.20 | F | 8.74 | 0.00 | 0.18 |
| L16 | 0.25 | 11.70 | -262.85 | 108.78 | 401.92 | 111.73 | 240.51 | F | 111.73 | 0.00 | > 1.0 |

| Degree of insurance on the slats in the transverse direction | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| T01 | 0.25 | 2.75 | -13.23 | 0.00 | 12.75 | 1.73 | 39.61 | F | 1.73 | 0.00 | 0 |
| T02 | 0.25 | 4.37 | -12.86 | 2.52 | 30.88 | 14.53 | 59.39 | F | 14.53 | 0.00 | > 1.0 |
| T03 | 0.25 | 1.88 | -39.82 | 5.31 | 9.81 | 5.21 | 25.24 | F | 5.21 | 0.00 | 0.98 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| T04 | 0.25 | 1.88 | -43.05 | 6.95 | 10.53 | 5.63 | 25.89 | F | 5.63 | 0.00 | 0.81 |
| T05 | 0.25 | 0.96 | -24.20 | 5.17 | 2.99 | 3.17 | 13.63 | D | 2.99 | 0.00 | 0.58 |
| T06 | 0.25 | 1.11 | -3.03 | 0.08 | 1.67 | 0.40 | 14.98 | F | 0.40 | 0.00 | > 1.0 |
| T07 | 0.25 | 2.46 | -10.69 | 0.16 | 12.93 | 1.40 | 34.94 | F | 1.40 | 0.00 | > 1.0 |
| T08 | 0.25 | 0.29 | -8.02 | 1.07 | 0.29 | 1.05 | 4.21 | D | 0.29 | 0.00 | 0.27 |
| T09 | 0.25 | 2.75 | -26.29 | 1.46 | 16.59 | 3.44 | 44.80 | F | 3.44 | 0.00 | > 1.0 |
| T10 | 0.25 | 1.78 | -37.49 | 0.99 | 13.35 | 4.91 | 27.75 | F | 4.91 | 0.00 | > 1.0 |
| T11 | 0.25 | 3.09 | -47.47 | 0.50 | 30.04 | 6.21 | 56.66 | F | 6.21 | 0.00 | > 1.0 |
| T12 | 0.25 | 0.51 | -12.48 | 1.12 | 1.26 | 1.63 | 7.19 | D | 1.26 | 0.00 | > 1.0 |
| T13 | 0.25 | 1.70 | -35.24 | 7.72 | 7.86 | 4.61 | 22.67 | F | 4.61 | 0.00 | 0.60 |
| T14 | 0.25 | 1.81 | -34.85 | 5.94 | 8.36 | 4.56 | 23.63 | F | 4.56 | 0.00 | 0.77 |
| T15 | 0.25 | 1.81 | -34.38 | 8.80 | 8.25 | 4.50 | 23.53 | F | 4.50 | 0.00 | 0.51 |
| T16 | 0.25 | 1.38 | -17.14 | 7.19 | 4.88 | 2.24 | 15.90 | F | 2.24 | 0.00 | 0.31 |
| T17 | 0.25 | 0.00 | -35.56 | 1.57 | 0.74 | 4.65 | 0.44 | F | 0.44 | 0.00 | 0.28 |
| T18 | 0.25 | 0.76 | -17.18 | 3.64 | 1.71 | 2.25 | 10.44 | D | 1.71 | 0.00 | 0.47 |
| T19 | 0.25 | 0.30 | -5.52 | 0.78 | 0.22 | 0.72 | 3.86 | D | 0.22 | 0.00 | 0.28 |

*Table no. 11 - Degree of insurance by structure*

| Direction | Investment |
|---|---|
| Ground floor - Longitudinal | **0.74** |
| Ground floor - Transversal | **0.61** |

| | |
|---|---|
| **Technical expert:** | ING. IOAN ROTARESCU |
| **Signature:** | |

# ANNEX C8 – CALCULATION BRIEF - EXISTING VARIANT – BUILDING C25

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of SR EN 1993 , for the existing structure.

# 1 MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered;*

*- the level of knowledge: KL3 – limited knowledge*

*- trust factor: CF = 1,35*

| Type | Investment | Standard |
|---|---|---|
| Minimum yield strength | fy=235 [N/mm2] | acc. tab.3.1, SR EN1993-1-1/2006 |
| Ultimate tensile strength | fu=360 [N/mm2] | acc. tab.3.1, SR EN1993-1-1/2006 |
| Poisson's ratio in the elastic range | n=0.3 | acc. 3.2.6, SR EN1993-1-1/2006 |
| Longitudinal modulus of elasticity | E=210000 [N/mm2] | acc. 3.2.6, SR EN1993-1-1/2006 |
| Partial safety factor | $\gamma M0=1.00$ | acc. 6.1, SR EN1993-1-1/2006 |
| Partial safety factor | $\gamma M1=1.00$ | acc. 6.1, SR EN1993-1-1/2006 |
| Partial safety factor | $\gamma M2=1.25$ | acc. 6.1, SR EN1993-1-1/2006 |

## 1.2 Geometric features

The compositional characteristics were determined according to the construction survey.

# 2 LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

## 2.1 Permanent loads

Load from the weight of metal elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of steel elements* | - | 78.5 | 78.5 |
| Total loads | | | | 78.5 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

## 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| $\gamma$ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |

| | |
|---|---|
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 6 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A

- category A: $q_k$ = 1.50 kN/m$^2$

***- Wind loads***

According to CR 1-1-4 2012 "Evaluation of wind action on buildings" the wind load is calculated according to the expression:

| $w_e = \gamma_{Iw}\, c_f\, q_p(z_e)$ | |
|---|---|
| $\gamma_{Iw}$ | importance factor |
| $q_b$ | dynamic wind pressure value on site |
| $z_e$ | reference height for external action |
| r | air density |
| vb | reference wind speed on site |
| $k_r(z_0)$ | Land factor |
| $z_0$ | roughness length |
| $c_r(z)$ | roughness factor |
| $v_m(z)$ | average wind speed at a height z=ze |
| √β | proportionality factor |
| $I_v(z)$ | wind turbulence intensity |
| $c_{pv}(z)$ | gust factor for average wind speed |
| $v_p(z)$ | peak wind speed at height z=ze produced by gusts |
| $k_r(z_0)^2$ | Land factor |
| $c_r(z)^2$ | roughness factor for dynamic wind pressure at height ze |
| $c_{pq}(z)$ | gust factor for average dynamic wind pressure at height ze |
| $q_p(z)$ | peak dynamic wind pressure at height ze |
| $c_f$ -1 | force coefficient (for headwind) |
| $c_f$ -2 | force coefficient (for wind at 450) |

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of “modal analysis with response spectrum” using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used
– using the combinations below:

$$E_{Edx} \text{ "+" } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ "+" } E_{Edy}$$

where:
$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction ($F_{xi}$ si $F_{yi}$) and the corresponding accidental eccentricity ($e_{xi}$ si $e_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i
$F_{xi}$ – static equivalent horizontal seismic force applied at level i.

The accidental eccentricity is calculated with the expression:

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table no. 22 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j\geq 1}\gamma_{G,j}G_{k,j} + \gamma_P P + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\Psi_{Q,i}Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j\geq 1}G_{k,j} + P + A_{ED} + \sum_{i\geq 1}\Psi_{2,i}Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j\geq 1}G_{k,j} + P + Q_{k,1} + \sum_{i>1}\Psi_{0,i}Q_{k,i}$ |
| | Frequent grouping | $\sum_{j\geq 1}G_{k,j} + P + \Psi_{1,1}Q_{k,1} + \sum_{i>1}\Psi_{2,i}Q_{k,i}$ |
| | Quasi-permanent grouping | $\sum_{j\geq 1}G_{k,j} + P + \sum_{i\geq 1}\Psi_{2,i}Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004.

*Table no. 23 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3 LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

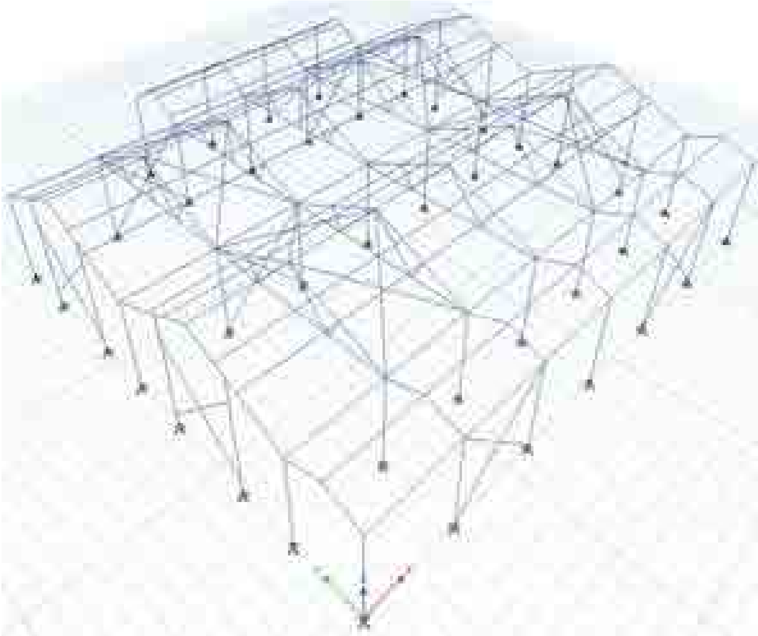


*Figure no. 6 - Calculation model – Spatial modeling – existing situation*

## 3.2 Calculation scheme

For each element, the capable axial force and the design buckling resistance are determined, these efforts are compared with the effective efforts resulting from the linear static analysis, resulting in the efficiency of the bars.

The design value of the tensile stress NEd in each cross-section must satisfy the following condition:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$$

$N_{Ed}$ – axial force calculation of tensile stress;

$N_{t,Rd}$ – axial tensile force;

For sections with holes, the design value Nt,Rd of the tensile strength shall be taken equal to the lower of the values below:

a) the design value of the plastic resistance in the gross cross-section

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$$

b) the design value of the ultimate resistance of the net cross-section, in the direction of the fixing holes

$$N_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2}}$$

A – the gross area of the cross-section;

$A_{net}$ – net area, in front of the fixing holes, of the cross-section;

$f_y$ – minimum yield strength of the steel;

$f_u$ – ultimate tensile strength of steel.

The design value of the compressive stress NEd in each cross-section must satisfy the following condition::

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0$$

$N_{Ed}$ – axial force calculation of the compressive stress;

$N_{c,Rd}$ – axial force capable of compression;

The design value of the cross-sectional resistance to uniform compression is determined as follows:

$$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$$

Compression bars must be checked for buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

$N_{Ed}$ – axial force calculation of the compressive stress;

$N_{b,Rd}$ – design resistance of the compressed bar to buckling;

The design buckling resistance of a compressed bar is equal to:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

In the case of concentrically compressed bars, the value of x must be calculated, taking into account the corresponding buckling curve, using the following relationship:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \; dar \; \chi \leq 1.0$$

$$\Phi = 0.5 \cdot \left[1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$$

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

α – imperfection factor;

$N_{cr}$ – critical axial elastic buckling stress, corresponding to the considered buckling mode, calculated based on the characteristics of the gross cross-section.

## 4. DEGREE OF INSURANCE

The R3 coefficient for the entire building, for each type of section and in each group, is presented in the following table:

| Load grouping | Section Type | | |
|---|---|---|---|
| | 1 (RHS 60x30x4) | 2 (RHS 30x30x3) | 3 (RHS 20x20x3) |
| | Insurance level by section type | | |
| G1 - P | **> 1,00** | 1.19 | 1.00 |
| G2 - Z | **0.65** | 0.95 | 0.65 |
| G3-V | **0.73** | 1.34 | 1.42 |
| GS-S | **0.60** | 1.35 | 1.50 |

| | |
|---|---|
| **Technical expert:** | ENG.DANIELDIACONU |
| **Signature:** | |

# ANNEX C9 – CALCULATION BRIEF - EXISTING VARIANT – BUILDING C27, C28

The calculation was carried out according to the provisions of the code P100-3/2019 and according to the provisions of CR6-2013 (Annex A1), for the existing structure. The degrees of insurance for each masonry strip were established for shear force, axial force and bending moment, and a degree of insurance for each main direction of arrangement of the structural elements (longitudinal and transverse). The minimum degree of insurance was calculated for each strip and a degree of insurance, in each direction (longitudinal and transverse) and, informatively, a degree of insurance for the entire structure.

From the linear static analyses, using the response spectrum corresponding to the location, the effective values of the efforts for each wall resulted.

The ratio between the minimum effort capable in a section and the external one, represents the degree of insurance of the element (masonry strip).

The total degree of insurance resulting reflects the degree of insurance of the structure under the assumption that the floors are rigid enough to transmit horizontal loads from one strip to another, making them work together in the event of an earthquake. The slats for which the degree of insurance resulted with a value of zero are subjected to tension, an effort that they cannot take over.

# 1. MATERIAL CHARACTERISTICS

## 1.1 Resistances (calculated values)

*Given the limited level of inspection and testing, it was considered;*

*- the level of knowledge: KL3 – limited knowledge*

*- trust factor: CF = 1,35*

For the calculation in the linear elastic domain, considering the behavior factor q (reduced spectrum), the design strengths of the masonry for evaluating the capacity to resist bending with axial force and shear are taken as follows:

| Masonry resistances | | | | |
|---|---|---|---|---|
| Compression | | | | |
| Type | Investment | | | Standard |
| Compressive strength of the masonry element | $f_b$= | 10.00 | [N/mm2] | acc. SR EN 771 |
| Average compressive strength of mortar | $f_m$ = | 4.00 | [N/mm2] | acc. SR EN 998-2:2004 |
| K coefficient for solid ceramic bricks | K= | 0.55 | table 4.1, code CR6-2013 | |
| | constant that depends on the type of masonry element and the type of mortar | | | |
| Characteristic compressive strength | $f_k$ = | 3.80 | [N/mm2] | acc. SR EN 1052-1 |
| Trust factor | CF = | 1.35 | - | acc. 4.1 P100-3/2019 |
| Partial trust coefficient | $\gamma_M$ = | 3.00 | - | acc. D.3.3.1.2.(7) P100-3/2019 |
| Design resistance value for walls subjected to shear force | | | | |
| for horizontal joint sliding failure (fvd): | | | | |
| Resistance to sliding failure in horizontal joint | $f_{vd}$ = | $f_{vk}/(\gamma_M \cdot CF)$ | | acc. D.3.4.1.3.1.P100-3/2019 |
| Characteristic breaking strength | $f_{vk}$ = | $f_{vk0} + 0.7 \cdot \sigma_d$ | | acc. 4.3.a CR6-2013 |
| The initial characteristic unit shear strength | $f_{vk0}$ = | 0,045 | [N/mm2] | |
| for breakage in the scale under the effect of main tensile stresses. (ftd) : | | $f_{td} = 0,04 \times f_m / (\gamma_M \times CF)$ | | |
| Breakage in the scale strength | $f_{td}$ = | 0.049 | [N/mm2] | acc. D.3.4.1.3.1.P100-3/2019 |
| Longitudinal modulus of elasticity of masonry | $E_z$ = | 3800 | [N/mm2] | acc. tab. 4.9 CR6-2013 |
| Transverse modulus of elasticity of masonry | $G_z$ = | 1520 | [N/mm2] | acc. rel. 4.9 CR6-2013 |

## 1.2 Geometric features

There are no original plans of the building, nor information on the building's behavior in 20th-century earthquakes.

The compositional characteristics were determined according to the construction survey.

## 2. LOADS

The values of standard loads are established based on the Eurocode SR EN 1991-1-1-2004.

### 2.1 Permanent loads

Permanent floor loads above ground floor

| Wooden floors | | | | |
|---|---|---|---|---|
| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m2) |
| 1 | Floor | - | - | 0.30 |
| 2 | Wooden cabinet | 0.05 | 6 | 0.30 |
| 3 | Wooden beams | - | - | 0.50 |
| 4 | Filler | 0.03 | 16 | 0.48 |
| 5 | Wooden toe board | 0.03 | 6 | 0.18 |
| 6 | Plaster | 0.02 | 19 | 0.38 |
| **Total loads** | | | | **2.14** |

Permanent loads at roof level

| Item. no. | Load name | Standard load on surface [kN/m2] |
|---|---|---|
| 1 | Cover (sheeting+cladding) | 0.30 |
| 2 | Roof framing (plywood+props+struts+clamps+rafters) | 0.40 |
| **Total loads** | | **0.70** |

Load from the weight of masonry elements:

| Item. no. | Element name | Thickness (m) | Technical weight (kN/m3) | Standard load (kN/m3) |
|---|---|---|---|---|
| 1 | Weight of solid brick masonry elements* | - | 18.0 | 18.0 |
| Total loads | | | | 18.0 |

* the weight of structural elements is generated automatically by the calculation program depending on the dimensions and technical weight of the materials

### 2.2 Variable loads

***- snow load (according to the code CR 1-1-3-2012)***

The action of snow on constructions **is considered as a variable action and in some cases of exceptional snow accumulation as an accidental action.**

The characteristic value of the snow load for the persistent/transient design situation is determined as follows:

| Loading from packed and unpacked snow – $S_k = \mu_i \cdot C_e \cdot C_t \cdot s_k$ | |
|---|---|
| γ – importance-exposure factor for snow action acc.to CR1-1-3/2012 | 1.00 |
| $\mu_i$ – the shape coefficient of the given snow load acc.to CR1-1-3/2012 | 0.8 |
| $C_e$ – exposure coefficient acc.to CR1-1-3/2012 | 1.0 |
| $C_t$ – thermal coefficient acc.to CR1-1-3/2012 | 1.0 |
| $s_k$ –the characteristic value of the load given by the snow on the ground acc.to CR1-1-3/2012 | 2.00 kN/m2 |
| Total loads | **$p_{1,k}$ = 1.60 kN/m2** |

Figure no. 1 - Zoning map of the characteristic value of the given snow load (IMR = 50 years)

***- useful loads***

When designing, for areas expected to be subjected to different load categories, the most critical load cases must be considered.

According to SR EN 1991-1-1-2004, tabel 6.1, the useful load on the current floors is:

- category A:
  - category A: $q_k = 1.50$ kN/m$^2$
- for building attics:
  - non-passable attics : $q_k = 0.75$ kN/m$^2$

## 2.3 Exceptional loads

The determination of the loads from the seismic action is done by the method of “modal analysis with response spectrum” using a linear elastic model. From the previously calculated vibration modes, the program generates the values of the equivalent seismic forces and applies them as static forces on the structure; after which, the values thus calculated by summing the results belonging to the vibration modes determine the maximum stress resulting from the seismic effect.

For the evaluation of the seismic loads, the following were considered:

| $F_b = \gamma_I \cdot S_d(T_k) \cdot \left(\sum m_i \cdot s_{i,k}\right)^2 / \sum m_i \cdot s_{i,k}^2$ | |
|---|---|
| $\gamma_I$ – important factor, according to tab. 4.3, P100-1/2013 | 1.00 |
| $a_g$ – ground acceleration for design, according to fig. 3.1, P100-1/2013 | 0,30 g |
| $T_c$ – the corner period, according to fig. 3.2, P100-1/2013 | 1,60 sec |
| q – factorul de comportare, according to Annex D, P100-3/2019 | 1.50 |
| - fraction of critical damping: **8%** (acc P100-3/2019); | |

The distribution of seismic forces at the level, for simplified calculation, results from the relationship:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i}$$

The calculation will consider eigenmodes with a significant contribution to the total seismic response. This condition is met if:

- the sum of the effective modal masses for the considered eigenmodes represents at least 90% of the total mass of the structure
- all eigenmodes with effective modal mass greater than 5% of the total mass were considered in the calculation

*The seismic safety assessment of the constructions was made considering the seismic action applied successively on both main directions of the building.*

**The combination of the effects of the components of the seismic action**

The program applies four methods for determining the maximum stresses and displacements, of which we mention the method used

– using the combinations below:

$$E_{Edx} \text{ "+" } 0{,}30\, E_{Edy}$$
$$0{,}30\, E_{Edx} \text{ "+" } E_{Edy}$$

where:

$E_{Edx(y)}$ – the effects of the action due to the application of seismic motion in the direction of the horizontal axis x, respectively y

**Effects of accidental torsion**

The torsion effect produced by an accidental eccentricity can be considered by introducing a torsional moment at each level.

Torsional effects were considered through the torsional moments ($M_{xi}$ si $M_{yi}$) around the vertical axis, according to P100-1/2013 – 4.5.3.3.3. They are determined as the product of the horizontal forces in each horizontal direction ($F_{xi}$ si $F_{yi}$) and the corresponding accidental eccentricity ($e_{xi}$ si $e_{yi}$).

$$M_{xi} = F_{xi} \cdot e_{xi}$$

$e_i$ – accidental eccentricity of the mass at level i

$F_{xi}$ – static equivalent horizontal seismic force applied at level i

The accidental eccentricity is calculated with the expression:

$$e_i = \pm 0{,}05 \cdot L_i$$

## 2.4 Load groups

In the calculation of the structure, it is necessary to take into account the most unfavorable load combinations, whose simultaneous action is practically achievable.

Table no. 24 - Load combination

| | | |
|---|---|---|
| **SLU** | Action groups for permanent or transient design situations (fundamental groups) | $\sum_{j\geq1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{Q,i} Q_{k,i}$ |
| | Action groups for seismic design situations | $\sum_{j\geq1} G_{k,j} + P + A_{ED} + \sum_{i\geq1} \Psi_{2,i} Q_{k,i}$ |
| **SLS** | Characteristic grouping | $\sum_{j\geq1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$ |
| | Frequent grouping | $\sum_{j\geq1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}$ |

| | | |
|---|---|---|
| | Quasi-permanent grouping | $\sum_{j\geq1} G_{k,j} + P + \sum_{i\geq1} \Psi_{2,i} Q_{k,i}$ |
| "+" means "in combination with" | | |

## 2.5 Load combinations

Load groups are calculated in accordance with the standard SR EN 1990:2004.

*Table no. 25 -*

| | Surname | type | Permanent | Useful – current | Useful – attic | Snow | Earthquake | SX | SY |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fundamental 1 | SLU | 1.35 | 1.50 | 1.05 | 1.35 | 0.00 | 0.00 | 0.00 |
| 2 | SLU 1 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 1.00 | 0.30 |
| 3 | SLU 2 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | 0.30 | 1.00 |
| 4 | SLU 3 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -1.00 | 0.30 |
| 5 | SLU 4 | SLU | 1.00 | 0.30 | 0.30 | 0.40 | 1.00 | -0.30 | 1.00 |
| 6 | SLS 1 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 7 | SLS 2 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |
| 8 | SLS 3 | SLS | 1.00 | 0.30 | 0.30 | 0.40 | 0.60 | 0.00 | 0.00 |

# 3. LINEAR STATIC ANALYSIS

Analytical evaluation by going through the following steps:

- modeling the structure for calculations (using the material characteristics established according to the particularities of each material)
- evaluation of loads, seismic forces and relevant load combinations
- establishing the calculation method depending on the composition of the existing construction and the purpose pursued by the respective evaluation.
- checking the structural elements from the point of view of stability, resistance, rigidity and ductility.

## 3.1 Calculation model

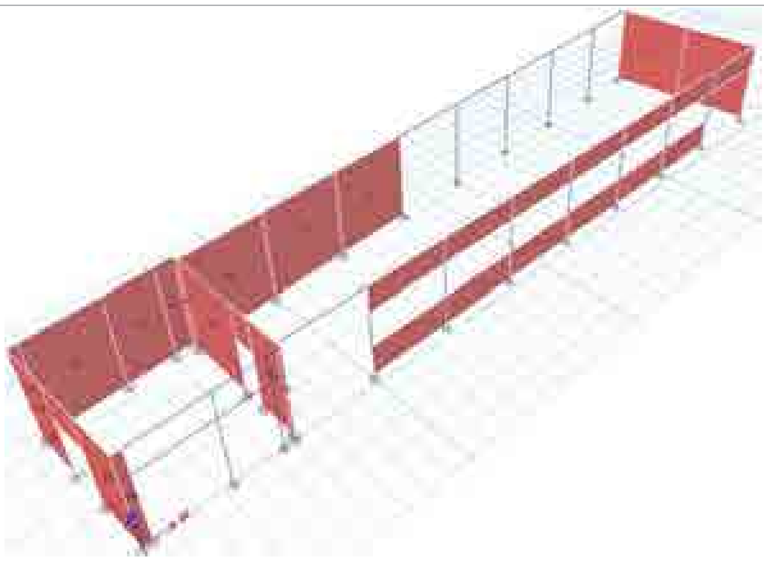


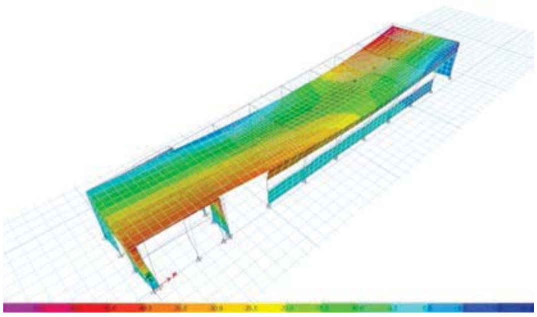
*Figure no. 2 - Calculation model – Spatial modeling – existing situation*

## 3.2 Analysis of natural vibration modes

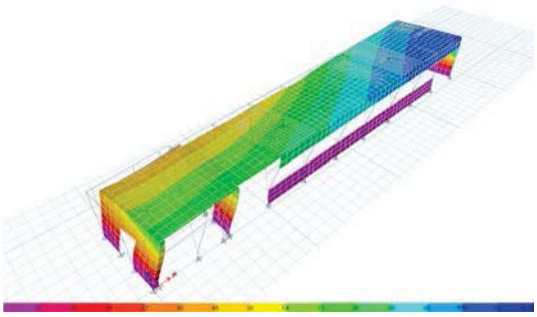
In the calculation, 12 eigenmodes of vibration were considered so that the sum of the modal participation coefficients in the two horizontal directions was greater than 0.9.

*Table no. 26 -*

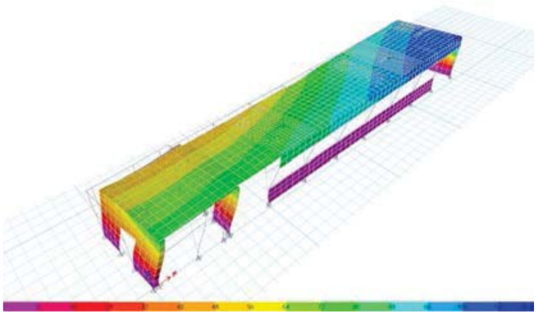
| Case | Mode | Period | Ux | Uy | Sum UX | Sum UY |
|---|---|---|---|---|---|---|
| Modal | 1 | 0.110 | 0.0374 | 0.695 | 0.0374 | 0.695 |
| Modal | 2 | 0.065 | 0.8146 | 0.0632 | 0.852 | 0.7582 |
| Modal | 3 | 0.049 | 0.0771 | 0.0106 | 0.9291 | 0.7688 |
| Modal | 4 | 0.038 | 0.0037 | 0.2003 | 0.9328 | 0.9691 |
| Modal | 5 | 0.029 | 0.0119 | 0.022 | 0.9447 | 0.9911 |
| Modal | 6 | 0.028 | 0.0446 | 0.0015 | 0.9892 | 0.9926 |
| Modal | 7 | 0.021 | 0.0013 | 0.0043 | 0.9905 | 0.9969 |
| Modal | 8 | 0.017 | 0.002 | 0.001 | 0.9925 | 0.998 |
| Modal | 9 | 0.016 | 0.0001 | 0.0000 | 0.9926 | 0.998 |
| Modal | 10 | 0.014 | 0.0012 | 0.0002 | 0.9938 | 0.9982 |
| Modal | 11 | 0.014 | 0.0014 | 0.0008 | 0.9952 | 0.9989 |
| Modal | 12 | 0.013 | 0.0013 | 0.0001 | 0.9965 | 0.999 |



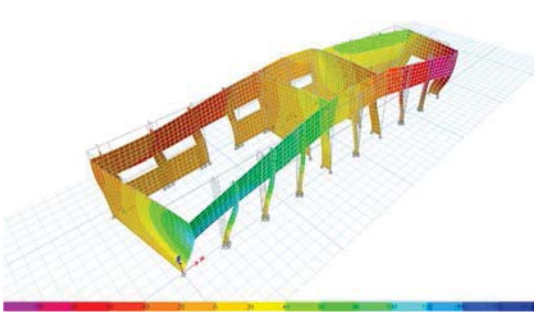
*Vibration mode 1 (3D view)*



*Vibration mode 2 (3D view)*



*Vibration mode 3 (3D view)*



*Vibration mode 4 (3D view)*

## 3.3 Analysis of forces in the slats

### 3.3.1 Calculation scheme - slats numbering

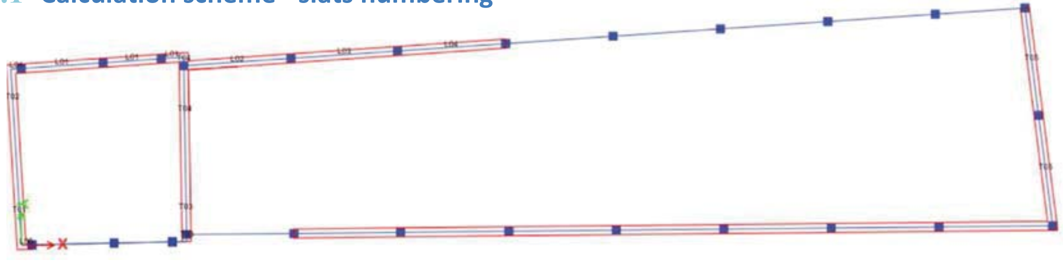


*Figure no. 6 - Ground floor beams – X; Y direction*

# 4. DEGREE OF STRUCTURE INSURANCE – QUANTITATIVE EVALUATION

## 4.1 Seismic assessment for the effects of in-plane wall action

The capable efforts in the slats were calculated with the relations from the normative *P100-3/2019 Annex D*.

### 4.1.1 Determining the degree of insurance for the entire building, on the slats in each direction

$$R_{3i} = \frac{V_{cap,i}}{F_{b,i}}$$

$V_{cap,i}$ is the shear force capable of wall "i" (the smaller of $V_{fd}$ and $V_{ff}$).

*Table no. 12 - Degree of insurance on the slats in the longitudinal and transverse direction*

| **Degree of insurance on the slats in the longitudinal direction** | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| L01 | 0.25 | 4.83 | -98.17 | 12.63 | 128.88 | 37.23 | 96.25 | F | 37.23 | 41.02 | > 1 |
| L02 | 0.25 | 3.00 | -76.44 | 12.02 | 60.97 | 10.00 | 56.76 | F | 10.00 | 41.02 | > 1 |
| L03 | 0.25 | 2.98 | -83.97 | 16.24 | 65.79 | 10.99 | 57.97 | F | 10.99 | 0.00 | 0.68 |
| L04 | 0.25 | 3.03 | -93.99 | 19.34 | 73.90 | 12.30 | 61.92 | F | 12.30 | 0.00 | 0.64 |
| L05 | 0.25 | 0.29 | -8.60 | 2.52 | 0.66 | 1.13 | 4.38 | D | 0.66 | 0.00 | 0.26 |

| **Degree of insurance on the slats in the transverse direction** | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slat | t [m] | $l_w$ [m] | $N_d$ [kN] | $V_d$ [kN] | $V_{f1}$ [kN] | $V_{f21}$ [kN] | $V_{f22}$ [kN] | D/F | $V_{fmin}$ [kN] | $V_{Rdi}$ [kN] | $R_{3i}$ |
| T01 | 0.25 | 1.77 | -54.10 | 16.73 | 24.88 | 7.08 | 26.93 | F | 7.08 | 41.02 | > 1.0 |
| T02 | 0.25 | 1.73 | -55.21 | 2.27 | 24.68 | 7.23 | 26.73 | F | 7.23 | 0.00 | > 1.0 |
| T03 | 0.25 | 1.53 | -43.34 | 29.76 | 17.36 | 5.67 | 22.64 | F | 5.67 | 0.00 | 0.19 |
| T04 | 0.25 | 2.82 | -100.00 | 41.39 | 71.77 | 13.09 | 56.30 | F | 13.09 | 0.00 | 0.32 |
| T05 | 0.25 | 6.12 | -145.56 | 62.00 | 238.66 | 55.75 | 128.26 | F | 55.75 | 41.02 | 0.90 |

*Table no. 13 - Degree of insurance by structure*

| Direction | Investment |
|---|---|
| Ground floor - Longitudinal | **1.00** |
| Ground floor - Transversal | **0.66** |

| **Technical expert:** | ING. IOAN ROTARESCU |
|---|---|
| **Signature:** | |