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Reinforcement Project for Earthquake-Damaged Buildings in Albania, case study Durrës, Albania

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ABSTRACT : *Albania has experienced significant seismic activity over the years, and the damaging earthquake of November 26, 2019, highlighted the need for extensive reinforcement efforts in buildings to mitigate the risks of future earthquakes. To ensure the safety and resilience of structures, a comprehensive reinforcement project would be essential to restore and strengthen earthquake-damaged buildings, particularly in high-risk zones.This was one of the strongest and one of the most catastrophic earthquakes in Albania, with a magnitude of 6.4 which struck the north-west region of Albania. The epicenter was 16 km west-southwest of the town of Mamurras in Kurbin Municipality. Areas like Tirana, Durresi, Laçi, Thumana were affected. This earthquake caused 51 casualties and 985 million euros or 1.2 billion dollars losses in construction.[1] Buildings that were built in different time zones and stoyers were affected experiencing structural and non structural damages and even collapsed in some cases. Albania is known for being a seismic region during the years as it was not the first earthquake with a big impact. A case study has been presented to illustrate the seismic damages caused to a building in Durres city, one of the most affected places from the earthquake. The post earthquake observations in this building shows the damages and the immediate need to reinforce the structure. A structure which was built in 1998 and was based on the projections codes like KTP-78 and KTP-N.2.89. These observations demonstrate the damages in columns, beams mainly in the first and second floors and several structural damages and not only. This paper presents on overview on the old codes used on constructing then and the ones used now days, how the reinforcement is done and which are the materials and methods used for the cause.*

Keywords: *Post earthquake damages, reinforcement techniques, structural integrity, building code, carbon fibers, jacketing.*

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I. INTRODUCTION

The case study being presented is a 6 floors hotel with a wooden roof located in Durrës city. This structure was one among the many that was damaged during the earthquake that stroke on the 26 the November 2019. An evaluation of the damages needed to be made.Structural Building Codes Used The building in question was constructed in 1998, and the seismic design codes in effect at the time in Albania were KTP-78 and KTP-N2-89.[2] These codes governed the structural design of the building, including aspects related to seismic resilience. On the KTP-78 code the design of reinforced concrete, steel, wood and masonry structures is performed using the limit state design, which takes into account the free and imposed vibrations, the deformed shape of the structure.

Seismic design code KTP-N2-89 describes better the need for structural uniformity.[3] It is expressed in terms of building's geometry, distribution of structural components, elastic mechanism and construction materials.[1] Never the less the times and needs have changed over the years including the materials use to built then and reinforce now the methods used to determine the damages, the programs used to provide the solution etc.[4]

Fig.1 Building location. [9]

Reinforcement methods proposed after the evaluation The evaluation of the building indicated significant structural weaknesses that could compromise its performance in the event of another earthquake. To address these issues, the following reinforcement measures are proposed:

- Jacketing of Columns: All columns on all floors will be jacketed, with an additional 10 cm thick concrete layer applied to each side of the columns. This will enhance the columns' capacity to withstand seismic forces.
- Addition of reinforced concrete walls: Four new reinforced concrete walls, each 30 cm thick, will be added on all floors. These walls will help to improve the lateral stability of the structure and resist earthquake-induced forces.
- Foundation Reinforcement: The connection between the plinths and foundation beams will be reinforced. This will involve the creation of a 30 cm thick reinforced concrete plate at the 0.00 floor level to improve the interaction between the foundations and enhance their load-bearing capacity.
- Repair of Damaged Columns: All damaged columns will be repaired by removing the deteriorated concrete and replacing it with high-resistance structural mortar. This will restore their strength and prevent further degradation.
- Reinforcement of Beams: The building's high and flat beams will be re-dimensioned to ensure they meet the structural requirements for earthquake resistance.
- Top Floor (Wooden Structure): No interventions are planned for the top floor, which is made of a wooden structure. This floor has not shown any signs of damage, and its impact on the overall stability of the building is considered minimal due to its suspended nature.

These measures aim to significantly improve the building's seismic resilience, ensuring that it is better able to withstand future earthquakes.

Structural Description

During site visits, all structural elements were identified through visual inspection. The structural elements are as follows:

- Columns: Reinforced concrete square columns, 30x30 cm, with constant dimensions along their height.
- Beams: Reinforced concrete beams with dimensions of 30x40 cm and 50x25 cm.
- Slabs:
  - floors with solid reinforced concrete slabs, cast in place, with a thickness of 13 cm.
  - 2 floors with voided reinforced concrete slabs, cast in place, with a 10 cm slab and 40 cm polystyrene infill, resulting in a total thickness of 25 cm.
- Staircase: Monolithic reinforced concrete stairs, cast in place, with a thickness of 15 cm.

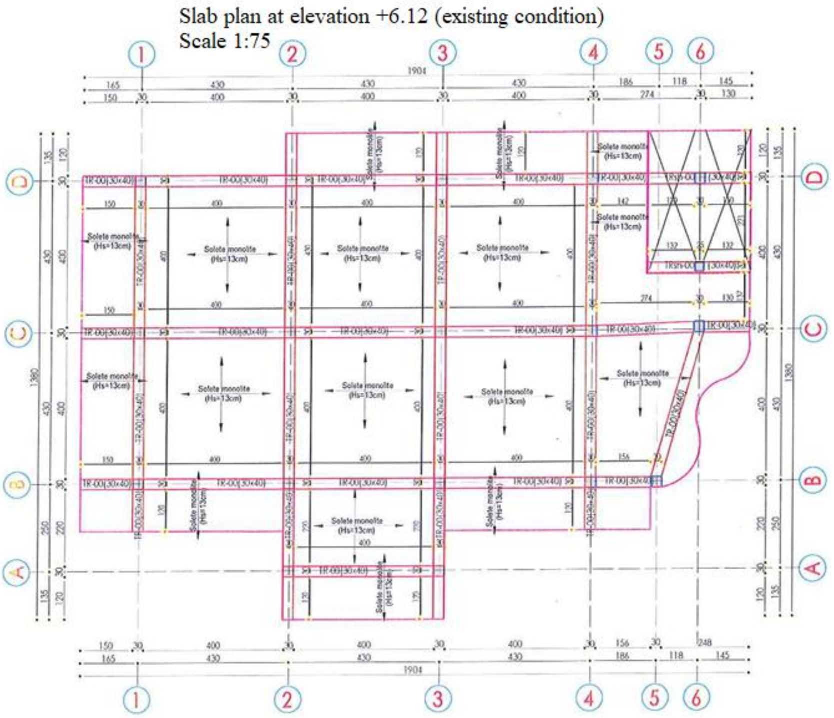


Fig.2 Plan of the existing structures at elevation 6.12 [2]

Based on observations, the existing condition of the building appears problematic. Small cracks have been identified in the masonry partition walls and at the joints between walls. Additionally, damage or cracks have been observed in slabs, beams, and columns, particularly in critical areas. The cracks are small in size and not continuous.

Fig. 3 Photos showing some of the inspections carried out after the Durrës earthquake of November 26. [2]

II. METHODOLOGY

Subsequently, we began analyzing the building and completed a full 3D model of the structure, In accordance with Eurocodes, calculations were made in the ultimate and serviceability limit states (ULS, SLS).

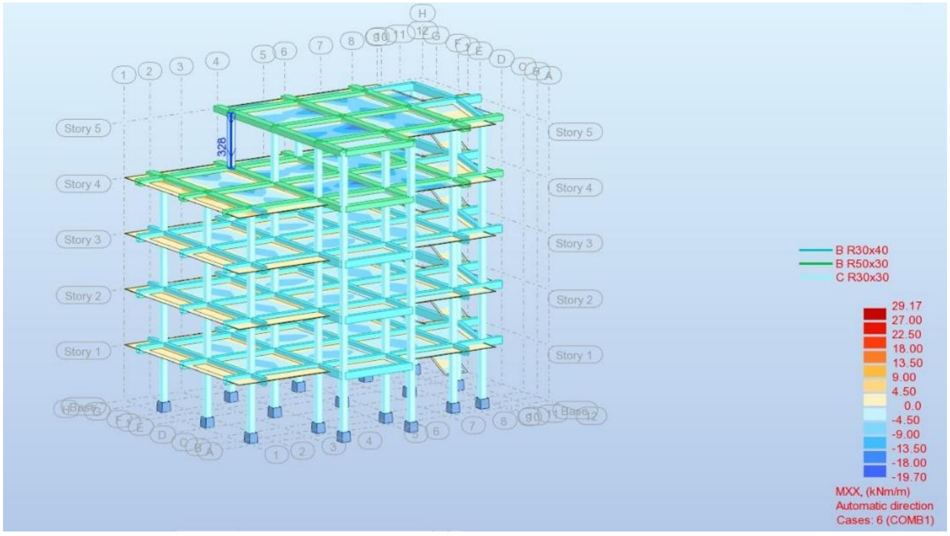


Fig. 4 3D model of the structure [2]

Concrete

Property	Value
Class	C12/15
Project specification	M-200
Self-weight	ρ = 2500 kg/m³
Cubic compressive strength	fck.cube = 150 daN/cm²
Prismatic compressive strength	fck = 120 daN/cm²
Modulus of elasticity	Ec = 270000 daN/cm²
Mean tensile strength	fctm = 16 daN/cm²
Minimum tensile strength	fctk 0.05 = 11.0 daN/cm²
Ultimate strain	εcu1 = 0.0035
Poisson's ratio	ν = 0.1
Partial safety factor	γC = 1.5
Design compressive strength	fcd = 80 daN/cm²

Ferrous Alloy Reinforcement Bars

Property	Value
Type	S 500
Tensile strength	ftk = 5000 daN/cm²
Yield strength	fyk = 5000 daN/cm²
Modulus of elasticity	Ec = 2100000 daN/cm²
Relative elongation coefficient	A5 = 12%
Partial safety factor	γS = 1/1.15
Design tensile strength	fyd = 4350 daN/cm²

Fig.5 Table presenting the characteristics of the used materials. [2]

- **Design Loads**

For the existing structure, considering its intended use, we accounted for dead loads, live loads, and seismic loads. The wind load was not considered, as seismic loads prevail over wind loads in this case. The design loads are as follows:

- **Dead and Live Loads**

  The following loads were used:
  - Dead load (g): 3 kN/m² (permanent load from layers and gypsum walls) [8]
  - Live load (p): 2 kN/m² (temporarily load) [8]
- **Seismic Loads**

  Seismic coefficients for the design were determined based on:
  - Seismic studies of the construction site near the building
  - The seismic acceleration map of the Republic of Albania

The following parameters were used: [8]

- Maximum ground acceleration (ag): 0.3g3
- Soil category: Category C (based on the geological report)
- Structural behavior coefficient (q): Calculated using:
- Ductility class factor (Kd): 3 (DCM, medium ductility)
- Regularity coefficient in height (Kr): 0.8 (irregular structure)
- Overstrength factor (au/al): 1.15
- Structural behavior coefficient (q0 × Kd × Kr × au/al): 2.76

The building was analyzed for three key states: Ultimate Limit State (ULS), Serviceability Limit State (SLS), and Serviceability Limit Deformation (SLD). The analysis was conducted following the Albanian design standards, considering permanent, temporary, and seismic loads.

- **Key Load Combinations:**

1. Dead Load (G) and Live Load (Q) were combined using partial safety factors:
- **gg** = 1.35 (or 1 if more more conservative)
- **gg** = 1.5 (or 1 if more more conservative)
- **y0i** and **y2i** are combination coefficients for variable loads

2. Seismic Loads:
- The seismic loads was considered with two orthogonal components: IEx (along the x-axis) and IEy (along the y-axis).
- Complete Quadratic Combination (CQC) method was used to combine these components.
- Seismic coefficients based on the geological study of the site and seismic acceleration data:
- ag = 0.3g (maximum ground acceleration)
- Soil category: Category C
- Structural behavior coefficients: Kd = 3 (ductility), Kr = 0.8 (irregular structure, and au/al = 1.15 (overstrength factor)

- **Load Combinations:**
- SLU (Ultimate Limit State): Permanent + seismic loads.
- SLS (Serviceability Limit State): Rare, frequent, and nearly permanent load combinations.
- SLD (Serviceability Limit Deformation): Seismic load deformation analysis.

These load combinations and safety factors ensure the building can withstand both expected loads and seismic forces, while maintaining functionality and safety.

Results of the Seismic Analysis

Displacement of the Floor:

"Requirements for limited damage" are considered acceptable if during a seismic event, with a probability greater than the seismic action that corresponds to the non-collapse requirement, the floor displacement remains limited. For our case, the design earthquake with a return period of 475 years and a 10% exceedance in 50 years is considered for calculations.

For structures with non-structural elements, where weak components are attached to the structure, the drift should satisfy the condition: $vqdr < 0.005h$

$0.5 * 11.7 > 3.5 * 0.005 = 1.75$ $5.85\text{ cm} > 1.75\text{ cm}$

Where:

- dr is the displacement of the floor relative to the possible damage limit state (From the program for the current condition of the object, dr = 13.80 cm, see the table below).
- h = 3.06m (the height of the floor).

The drift achieved in the x-direction is 13.80 cm, which exceeds the 3% value, whereas the Eurocode for drifts is much more stringent and allows no more than 1% for the performance level of significant damage (DC). [8]

Case/Story	UX (cm)	UY (cm)	dr UX (cm)	dr UY (cm)	d UX	d UY	Max UX (cm)	Max UY (cm)	Min UX (cr
1/ 1	-0.0	-0.0	-0.0	-0.0	-0.00	-0.00	0.0	0.0	-
1/ 2	-0.0	-0.0	-0.0	-0.0	-0.00	-0.00	0.0	0.0	-
1/ 3	-0.0	-0.0	-0.0	0.0	-0.00	0.00	0.0	0.0	-
1/ 4	-0.0	-0.0	-0.0	0.0	-0.00	0.00	0.0	0.0	-
1/ 5	-0.0	-0.0	-0.0	-0.0	-0.00	-0.00	0.0	0.0	-
2/ 1	-0.0	-0.0	-0.0	-0.0	-0.00	-0.00	0.0	0.0	-
2/ 2	-0.0	-0.0	-0.0	0.0	-0.00	0.00	0.0	0.0	-
2/ 3	-0.0	0.0	-0.0	0.0	-0.00	0.00	0.0	0.1	-
2/ 4	-0.0	0.0	-0.0	0.0	-0.00	0.00	0.0	0.1	-
2/ 5	-0.1	0.0	-0.0	-0.0	-0.00	-0.00	0.0	0.1	-
4/ 1	3.4	0.8	3.4	0.8	0.01	0.00	3.4	2.1	
4/ 2	7.1	1.6	3.7	0.8	0.01	0.00	7.3	4.5	
4/ 3	10.2	2.2	3.1	0.7	0.01	0.00	10.6	6.4	
4/ 4	12.5	2.7	2.3	0.5	0.01	0.00	12.9	7.7	1
4/ 5	13.8	2.4	1.4	-0.3	0.00	-0.00	14.3	6.2	1
5/ 1	1.2	3.5	1.2	3.5	0.00	0.01	2.9	5.3	
5/ 2	2.5	7.1	1.3	3.6	0.00	0.01	6.1	11.2	
5/ 3	3.6	10.0	1.0	2.9	0.00	0.01	8.6	16.0	
5/ 4	4.3	12.1	0.7	2.1	0.00	0.01	10.3	19.0	
5/ 5	4.7	10.9	0.5	-1.1	0.00	-0.00	11.2	14.3	

Fig.6 Table presenting the drifts of the existing structure. [2]

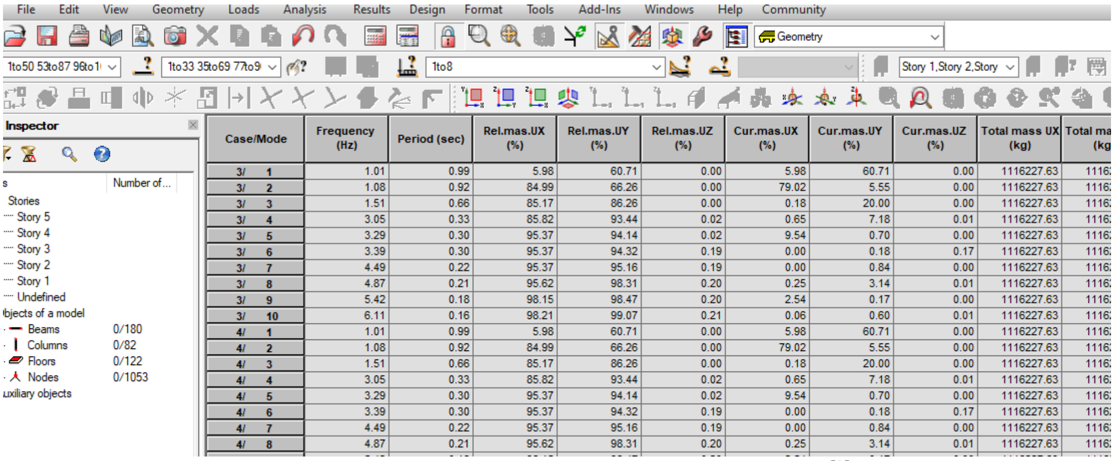
Displacements:2 - Cases: 1 2 4to8

	UX (cm)	UY (cm)	UZ (cm)	RX (Rad)	RY (Rad)	RZ (Rad)
MAX	16.5	21.0	4.9	0.024	0.016	0.013
Node	622	157	1777	1777	150	1759
Case	7 (C) (CQC)	8 (C) (CQC)	5	8 (C) (CQC)	7 (C) (CQC)	8 (C) (CQC)
Mode			CQC			
MIN	-0.4	-0.2	-4.2	-0.001	-0.001	-0.000
Node	96	193	1777	1825	1272	1758
Case	6 (C)	6 (C)	6 (C)	6 (C)	6 (C)	6 (C)
Mode						

Fig.7 Table presenting the maximum and minimum displacements of the existing structure [2]

In the case of the design earthquake, the maximum displacement of the building is 21 cm. This displacement exceeds the values permitted by the Eurocode or the current local regulations (KTP). According to the codes, displacements greater than 1/200 of the building's height are not allowed, meaning the maximum allowed displacement is 10 cm.

Period of the building's oscillations:
The period resulting from the program for the existing structure is 1.76 seconds, which is significantly greater than the value recommended by the Eurocode (0.5 to 0.6 seconds). This is shown in the table below:



Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)	Cur.mas.UX (%)	Cur.mas.UY (%)	Cur.mas.UZ (%)	Total mass UX (kg)	Total ma (kg
3/ 1	1.01	0.99	5.98	60.71	0.00	5.98	60.71	0.00	1116227.63	1116
3/ 2	1.08	0.92	84.99	66.26	0.00	79.02	5.55	0.00	1116227.63	1116
3/ 3	1.51	0.66	85.17	86.26	0.00	0.18	20.00	0.00	1116227.63	1116
3/ 4	3.05	0.33	85.82	93.44	0.02	0.65	7.18	0.01	1116227.63	1116
3/ 5	3.29	0.30	95.37	94.14	0.02	9.54	0.70	0.00	1116227.63	1116
3/ 6	3.39	0.30	95.37	94.32	0.19	0.00	0.18	0.17	1116227.63	1116
3/ 7	4.49	0.22	95.37	95.16	0.19	0.00	0.84	0.00	1116227.63	1116
3/ 8	4.87	0.21	95.62	98.31	0.20	0.25	3.14	0.01	1116227.63	1116
3/ 9	5.42	0.18	98.15	98.47	0.20	2.54	0.17	0.00	1116227.63	1116
3/ 10	6.11	0.16	98.21	99.07	0.21	0.06	0.60	0.01	1116227.63	1116
4/ 1	1.01	0.99	5.98	60.71	0.00	5.98	60.71	0.00	1116227.63	1116
4/ 2	1.08	0.92	84.99	66.26	0.00	79.02	5.55	0.00	1116227.63	1116
4/ 3	1.51	0.66	85.17	86.26	0.00	0.18	20.00	0.00	1116227.63	1116
4/ 4	3.05	0.33	85.82	93.44	0.02	0.65	7.18	0.01	1116227.63	1116
4/ 5	3.29	0.30	95.37	94.14	0.02	9.54	0.70	0.00	1116227.63	1116
4/ 6	3.39	0.30	95.37	94.32	0.19	0.00	0.18	0.17	1116227.63	1116
4/ 7	4.49	0.22	95.37	95.16	0.19	0.00	0.84	0.00	1116227.63	1116
4/ 8	4.87	0.21	95.62	98.31	0.20	0.25	3.14	0.01	1116227.63	1116

Fig.8 Results of the period of the existing structure. [2]

As seen from the analysis above, it results that:

1. The oscillation period of the structure (0.99 s) is much greater than the value recommended by Eurocode 8, where the recommended value is 0.5 to 0.6 seconds.
2. The drifts of the structure exceed the limits recommended by Eurocode 8.
3. The structural elements are not reinforced in accordance with the code requirements for local ductility (there is a lack of densely spaced reinforcement in critical zones, inadequate reinforcement of columns, and improper placement of rebar in columns).
4. Local ductility is not ensured in CP areas.
5. The reinforcement of the columns in the plastic areas can not provide the demand in rotation.

III. CONCLUSION

Based on the results of the calculations and the observations made on the structure, we conclude that the building does not meet the required performance levels and urgently needs to be retrofitted in order of it to fulfilling the requirements of the structural response needed. The form used to the reinforcement will be meshing/jacketing of the columns/beams. Jacketing is a very common method use for strengthening of columns/beams. The most common types of jackets are steel jacket, reinforced concrete jacket, fiber reinforced polymer composite jacket, jacket with high tension materials like carbon fiber ,etc. The main reason we use jacketing is to increase in the shear capacity of columns in order to accomplish a strong column-weak beam design, to improve the column's flexural strength, to increase concrete and shear strength confinement by transverse fiber reinforcement, to increase flexural strength by longitudinal fiber reinforcement provided, that are anchored at critical sections. [6] In traditional reinforced concrete jacketing, the section of the column is enlarged by casting a new reinforced concrete/mortar section over a part or the entire length of the column. The new section is bonded to the original section through anchor rebars or high-strength bolt.[5]

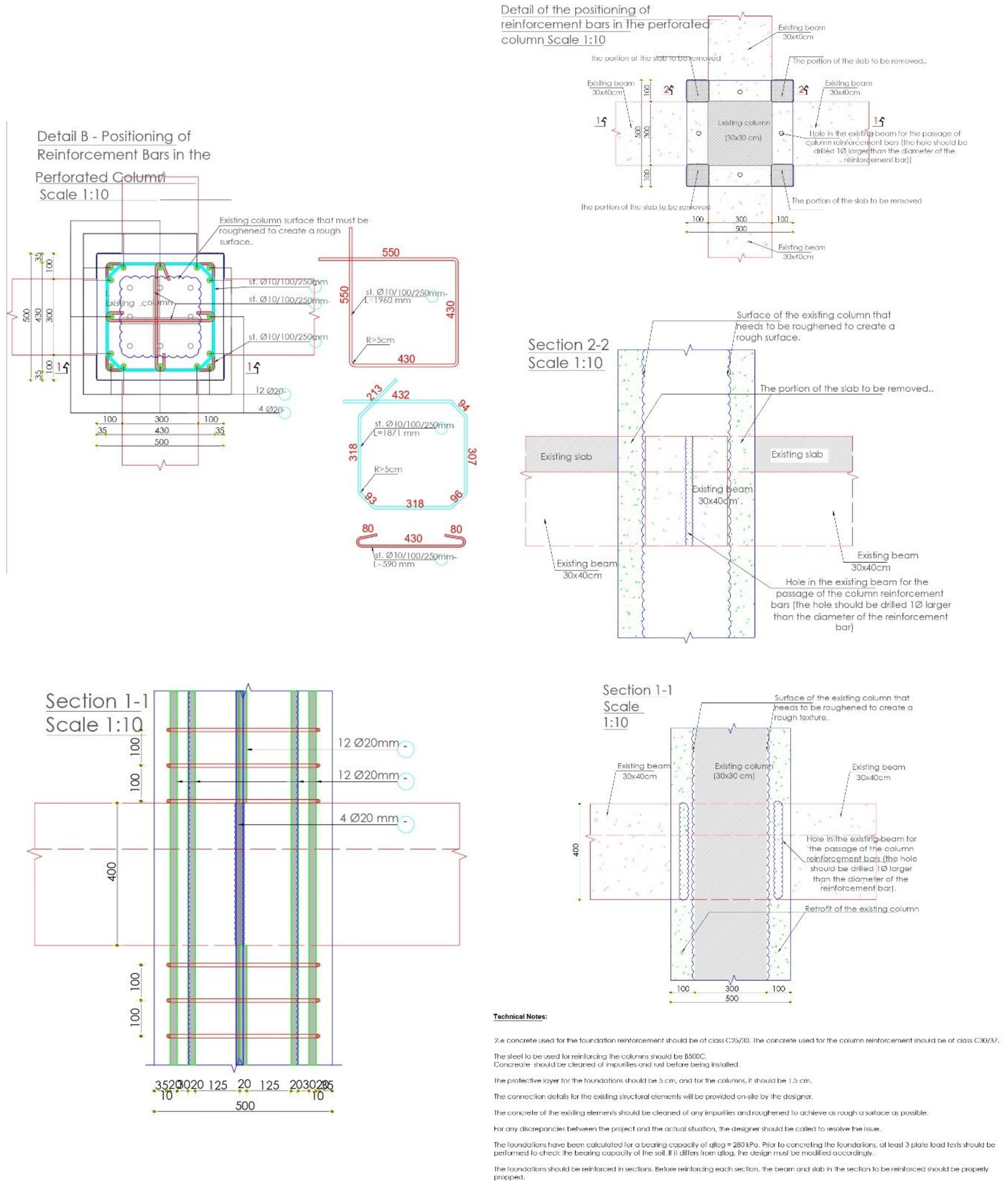


Fig.9 Details – Positioning of Reinforcement Bars in the Perforated Columns [2]

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