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SEISMIC ASSESSMENT AND INNOVATIVE STRENGTHENING OF CORRODED CANTILEVER STRUCTURES IN MULTI-STOREY BUILDINGS

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Abstract

The research aims to investigate the existing design standards KANEPE (2017) [1] and EN 1998-3 (2005) [2] regarding the seismic assessment of existing reinforced concrete structures through a real case study. Also, the current and applicable design standards together with the available strategies and methodologies for seismic retrofit of existing structures are presented in detail. In addition, the techniques and methods proposed to date for retrofitting reinforced concrete cantilever members such as balconies are analyzed. Through the real example, are presented: the operation of the reference building over the years, its existing conditions and the available architectural and structural engineering studies on the basis of which the building was constructed. In addition, the assessment methodology is described using the seismic assessment results of the existing building by identifying the critical structural elements with an advanced degree of damage aiming to explore innovative retrofitting methods, focusing on cantilever structural elements. The novel aspect of the proposed research project distinguishes it from similar studies by using an actual case study of an existing reinforced concrete structure that was constructed in Rush in the 1980s. As a result of aging, the structure's cantilever failed in a brittle manner. With regard to reinforced concrete cantilevers aging issues make them vulnerable to potential failure and thus the study of their behavior and suggestion of strengthening techniques are one of the most current engineering issues that structural engineers are requesting solutions for. Through this study, a comprehensive methodology of structural assessment and recommendation of retrofitting options is carried out.

Keywords: Seismic assessment, cantilevers retrofitting, Concrete deterioration

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1 INTRODUCTION

The work aims to investigate and examine the current design standards and the seismic assessment methodology of existing reinforced concrete buildings according to KANEPE (2017) [1] and EN 1998-3 (2005) [2]. Initially, the assessment is based on Performance criteria and is presented in a diagram in terms of Base Force versus Displacement, where four phases are distinguished: a) "Immediate Occupancy (IO) " where most of structure's functions can continue immediately as the construction is safe to use. In this category, minor tasks may be interrupted and require repair to resume; b) " Life-safety (LS)" in which moderate damages can be distinguished but with the structure remaining stable. Human life is protected, but the building may not be economically viable to repair; c) "Limited Safety (LS)" where the structure suffers from significant damage, occupants can evacuate but may not re-enter, and d) "Collapse Prevention (CP)" in which the building will suffer significant damage, but the collapse of the structure is prevented. Non-structural elements will possibly collapse, and their repair is basically not possible. The next general rule refers to the range of applicability of four methods of analysis: a) Static Elastic (Equivalent), b) Dynamic Elastic (Spectral), c) Non-linear Static (with Imposed Horizontal Displacements), d) Non-linear Dynamic (time history analysis). The third rule examines the type of failure modes, either ductile or brittle. More specifically as ductile elements are examined beams, columns, walls in bending and as brittle elements are examined, beams, columns, walls, joints in shear.

Last category during the assessment process is the documentation, which is based on the collection of information for the assessment which is distinguished in: Limited Knowledge (structural geometry from the initial study, with sampling confirmation or with full mapping in the absence of an initial study, with assumptions about the materials or a sample for confirmation and reinforcement details from repetition of initial study), Common knowledge (structural geometry and reinforcements from initial study and sample confirmation of about 20% or with full survey and uncover of certain members at a rate of more than 50% in the absence of initial study, materials based on original of study and one or two samples for confirmation) and Full knowledge (structural geometry and reinforcement details from initial study and sample confirmation at a rate of more than 20% or with full mapping and disclosure of reinforcements at a rate of more than 80% in the absence of an initial study, materials based on data sheets of previous tests and one or three samples for confirmation).

Teng et al. (2000) [3] highlighted that RC cantilever beams and slabs are commonly found in buildings. with RC canopies and balconies to be widespread. These cantilever slabs, being statically determinate structures, can fail abruptly when their moment capacity is exceeded at the support. Recent years have witnessed several failures of cantilever slab structures, resulting in injuries and fatalities, which has raised significant public concerns about their safety. The majority of these failures were attributed to the deterioration or incorrect placement of steel reinforcement. Consequently, the development of a straightforward and efficient method to rehabilitate deficient RC cantilever slabs holds great importance [4][7].

Raza et al. (2019) [8] proposed steel jacketing, where the RC section is enlarged by bolting it with a steel section. The gap between the concrete and steel is filled with grout. While effective for seismic performance enhancement, this method is costly, labour-intensive, and involves anti-rust work. Similar to RC jacketing, it alters the stiffness of the structure. Ro et al. (2024) [9] introduced a novel retrofitting technique for RC beams using modularized steel plates. The method encases the RC beam with a steel plate, creating a void filled with non-shrinkage mortar. This approach combines concrete jacketing with steel plate retrofitting, augmenting the rigidity and strength of the existing RC beam while bolstering its ductility.

In a study by Afefy et al. (2020) [10], three mild steel plates (100 mm width, 2 mm thickness, 2100 mm total length) were mounted on the tension side of a defected cantilever slab. Mechanical anchors were used to prevent debonding at both ends of the steel plates. Threaded rods (10 mm diameter, 150 mm length) were fixed on both sides. The substrate slab was prepared by drilling holes, cleaning, and applying epoxy-165 paste. The steel plates were coated with epoxy and aligned, then pressed against the slab surface using external weights. Additional square steel plates (80 mm side length) were provided at anchorage locations, secured with steel nuts on threaded rods. From a ductility perspective, applying externally bonded steel plates allowed the defective slab to exhibit ductile behavior similar to that of a properly detailed slab.

Ayash et al. (2020) [11] advocate for the use of fiber reinforced polymer (FRP) composite materials due to their superior advantages over traditional steel bonding plates, such as a high strength-to-weight ratio, corrosion resistance, and architectural adaptability. To prevent premature debonding failures, researchers suggest employing various end anchorage systems to ensure proper flexural capacity development in strengthened members. Prior studies by Teng et al. (2000) [3] demonstrated the effectiveness of bonding GFRP strips onto reinforced concrete (RC) cantilever slabs, particularly when anchored into walls via horizontal slots using fiber anchors.

In their study, Ayash et al. (2020) [11] examined the behavior of GFRP-reinforced concrete cantilever slabs, both experimentally and numerically. They found that employing GFRP as a top mesh significantly enhances load-carrying capacity and influences deflection, ductility, and stiffness properties.

Due to concrete's low tensile capacity, reinforced concrete (RC) structures are prone to cracking. However, Engineered Cementitious Composite (ECC) offers a solution due to its high tensile strain capacity, making it an ideal retrofitting material. Researchers have explored ECC strengthening in various RC components like beams, columns, beam-column joints, and slabs. ECC is typically combined with FRP textiles or steel bars to significantly boost structural strength. Introducing ECC into a structure induces multiple tiny cracks on tensile surfaces, such as beam bottoms and joint cores, without compromising load-bearing capacity, often leading to a ductile failure mode. While FRP textiles and steel bars enhance structural integrity, their primary role is to increase strength.

Gao et al. (2018) [12] and Hu et al. (2015) [13] utilized Basalt Fiber Reinforced Polymer (BFRP) reinforced ECC to strengthen fire-damaged RC slabs. They varied parameters like fire exposure time, matrix type, and BFRP grid layers. This technique notably increased load-carrying capacity, with ECC outperforming polymer-modified mortar in both load capacity and ductility. However, while the BFRP layer increased load capacity, it reduced ductility. Ioannou et al. (2021) [14] investigated the effect of ECC jacketing for the beam-column elements' load-bearing capacity. Thin ECC jackets, comprising a fiber-reinforced ECC, were applied to replace damaged cover without additional confining reinforcement. Experimental results demonstrated ECC jackets' effectiveness in rehabilitating elements, enhancing strength and ductility while addressing deficiencies in transverse reinforcement.

Notably, in the summer of 2022, a cantilever failure occurred in an existing building in Cyprus due to steel reinforcement corrosion. The current investigation considered this failure as a case study and employed a combined approach involving both destructive and non-destructive methods to characterize the mechanical properties of concrete and steel. By utilizing a structural analysis program and conducting nonlinear analysis, the structural integrity of the building was assessed and identified the critical structural elements. Furthermore, the reasons behind the cantilever failure, with a specific focus on exploring modern retrofitting techniques for reinforced concrete cantilever members were examined. These techniques include retrofitting with Fiber Reinforced Polymers (FRP), Steel Strips or Steel Plates, and Engineered Cementitious Composite Jacketing (ECC).

2 STRUCTURAL CONDITION EVALUATION APPROACHES

2.1 Corrosion in Reinforced Concrete

Corrosion, according to the American Society for Testing and Material [15], is the chemical or electrochemical reaction between a material (usually a metal) and its environment that causes alteration of the material and its properties. It is a natural phenomenon commonly observed in existing reinforced concrete structures, particularly when the concrete has high permeability. This exposure leads to the reinforcing steel being vulnerable to the corrosive environment. Rust, resulting from the oxidation of iron or ferrous alloys (e.g., reinforcing steel bars), causes a reduction in the cross-section of the reinforcement and internal tensile stresses, leading to cracks in concrete cover. As the steel reinforcement's cross-sectional area decreases, the load-bearing capacity of the concrete member is compromised.

There are two primary mechanisms causing reinforcement corrosion in reinforced concrete: (a) chloride ions: In cold weather countries, reinforced concrete is exposed to chloride ions through deicing salts used in winter, leading to corrosion caused by chlorine and (b) carbonation of concrete which also contributes to reinforcement corrosion. When steel is embedded in concrete (as in reinforced concrete), it benefits from the alkalinity of concrete, which shields it from external corrosive factors. The corrosion process involves initiation and propagation over several years before reaching the end of structure's life. In the presence of moisture (H_2O) and oxygen (O_2), bare steel is vulnerable to corrosion. When iron (Fe) comes into contact with chloride ions (Cl-) in an oxidizing and moist environment, electrochemical reactions take place spontaneously.

However, when steel is embedded in concrete, such as in reinforced concrete, it is protected from the external environment by the alkalinity of the concrete. Concrete possesses a high pH of approximately 13.5. Over several years, the corrosion process undergoes two primary stages: initiation and propagation. The concrete cover's alkaline environment results in the formation of a passive layer around the reinforcement. Under normal conditions, this layer slows electrochemical reactions between the rebars and the environment, rendering the metal passive. If sufficient chlorine (Cl-) or carbon dioxide (CO_2) ions from the atmosphere penetrate the concrete cover and reach the reinforcement layer, then the protective passive layer weakens, and the embedded steel becomes depassivated, triggering the corrosion process. Rust forms through the oxidation of steel and reduction of oxygen.

At the same time the surrounding concrete experiences tensile stresses due to expansion of rust products. If these stresses exceed the concrete's tensile capacity, cracking occurs. The mechanical impact is observed through large cracks, spalling, or debonding of the concrete. Concrete's alkalinity decreases, lowering the pH of the pore solution to around 9. At pH below 9, corrosion can occur, damaging the protective reinforcement layer. Other Hazard Mechanisms: Cracking and CO_2 exposure can initiate corrosion.

2.2 Destructive and Non-Destructive Methods to Evaluate Corrosion

Researchers have found that studying existing old buildings over time reveals similarities in design and construction practices within specific regions and periods [16]-[17]. These similarities encompass various aspects such as material grades, member sizes, presence of shear walls, foundation types, and reinforcement details. Changes in structural standards typically delineate time periods with shared characteristics. For instance, seismic design alterations in

Cyprus since the publication of seismic standards in 1992 distinguish buildings designed in different eras.

Existing reinforced concrete structures often exhibit inadequate structural details and damages, including deficiencies like lack of stirrups and transverse reinforcement, which commonly necessitate retrofitting [16]. During building surveys, engineers should thoroughly inspect all structural elements to ensure accurate and reliable designs (Dritsos, 2001 [18]). However, comprehensive sampling for mechanical property determination may not be feasible in occupied buildings, necessitating a balance between accuracy and practicality [16].

To assess material quality effectively, a combination of non-destructive and destructive methods is recommended, as they offer a higher level of knowledge and accuracy in estimating the concrete compressive strength [18].

2.3 Existing Non-Destructive Testing Methods of Concrete

Various non-destructive testing (NDT) methods have been developed over the years or are under development for investigating and evaluating reinforced concrete structures, primarily aiming to estimate strength, monitor corrosion, measure crack sizes, assess concrete quality, detect defects, and identify vulnerable areas [17]. Derived from methods for metallic materials, these NDT methods have a sound scientific basis, but concrete's non-homogeneous nature complicates result interpretation due to various influencing parameters such as materials, mixture, construction quality, and environment [18]. Interpretation of results, particularly for estimating resistance, requires expert knowledge, as generalization is not possible [17]. The purposes of NDT methods include assessing compressive strength, uniformity, quality, integrity, detecting imperfections, monitoring structural changes, identifying reinforcement details, determining steel condition, and measuring concrete properties like elasticity and chemical content. NDT equipment can be categorized based on their use into strength estimation, corrosion assessment, defect detection, and laboratory tests.

2.4 Core Testing for the Evaluation of Concrete Strength

Core testing is a semi-destructive method involving the extraction of cylindrical samples, typically recommended to have a diameter ranging from 10 to 15 cm (Vera et al., 2009). Larger diameter cores are preferred as they yield more accurate results due to reduced susceptibility to damage during drilling. Research indicates that 10 cm diameter cores exhibit strength values approximately 20% higher than 5 cm diameter cores [18]. Cores with a 5 cm diameter are primarily utilized for assessing the internal properties of concrete structures, such as resinization effectiveness.

Regarding the length of the core or the height-to-diameter ratio (L/D), no universally recommended value exists. Previous studies suggest L/D ratios varying between 0.95 and 2.0, with larger values approximating conventional cylindrical specimens [17]. For 10 cm diameter cores, a specimen height exceeding 25 cm is necessary to account for end segment removal. However, ratio values close to unity are deemed acceptable, particularly when cores are extracted from slabs.

3 DESCRIPTION OF THE CASE STUDY

The examined structure is an existing reinforced concrete building located in Kato Paphos, within the urban area of the Municipality of Paphos. The Building Permit was issued by the Municipality of Paphos on July 19, 1983, and pertained to the construction of a two-story building. The structural analysis of the building was conducted in September 1982, based on the BS CP 110-1 [19].

In July 2022, the cantilever (balcony) of the top floor of the above-mentioned structure, as depicted in **Figure 1 (a), (b), (c), and (d),** failed under its own weight, resulting in its collapse. According to the structural analysis conducted in 1982, reinforcement of Φ10/27.5 has been installed. The construction details of the steel reinforcement configuration during that period are shown in Figure 6. Following the aforementioned incident, the building was deemed temporarily unsuitable for occupation to undergo the necessary repair. A series of destructive and non-destructive tests were subsequently conducted to determine the mechanical properties of both concrete and steel reinforcement.

Subsequently, using appropriate structural analysis software, a seismic assessment of the existing building was performed, allowing for the identification of critical failure points. Concluding the seismic assessment of the structure, an investigation followed, proposing new and innovative methods for the repair and retrofitting of cantilevers using finite element analysis software.

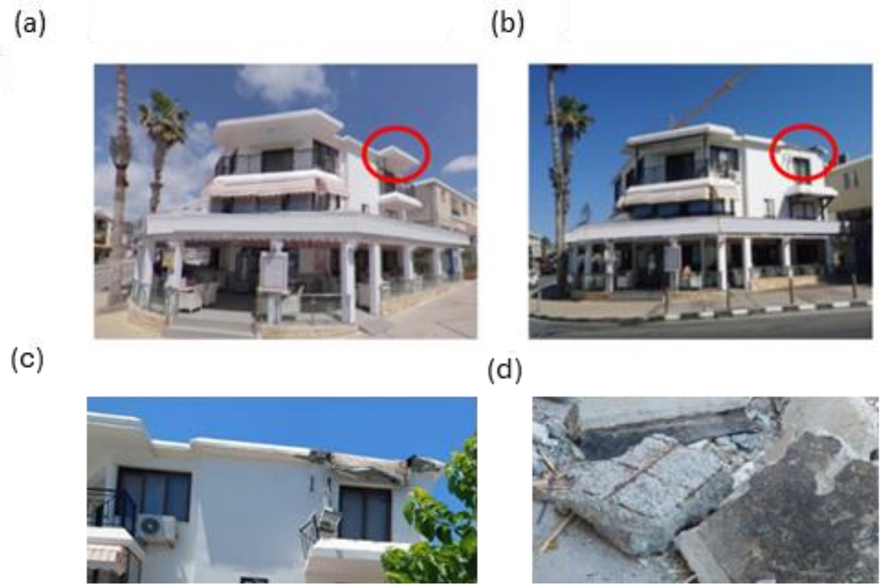

Figure 1: (a) Building facade (the failed cantilever is discernible) (Durable Structural Consulting Engineering Archive); (b) Building facade (the failed cantilever is visible) Durable Structural Consulting Engineering Archive); (c) and (d) Photographic documentation of the failure of the 165 cm cantilever (Durable Structural Consulting Engineering Archive).

3.1 Conclusions of Investigative Documentation on the Structural Load-Bearing System

The investigation efforts were concentrated on the structural irregularities within the load-bearing system, particularly concerning the details of steel reinforcement and the quality of concrete. As shown in Figure 2 (a) and (b), it is imperative to underscore that significant irregularities have been observed, including the pronounced degree of concrete carbonation, corrosion of steel reinforcement, inadequate waterproofing and the weakness of design standards with regards to the minimum spacing requirements for reinforcement, particularly in reinforced concrete cantilevers.

(a) (b)

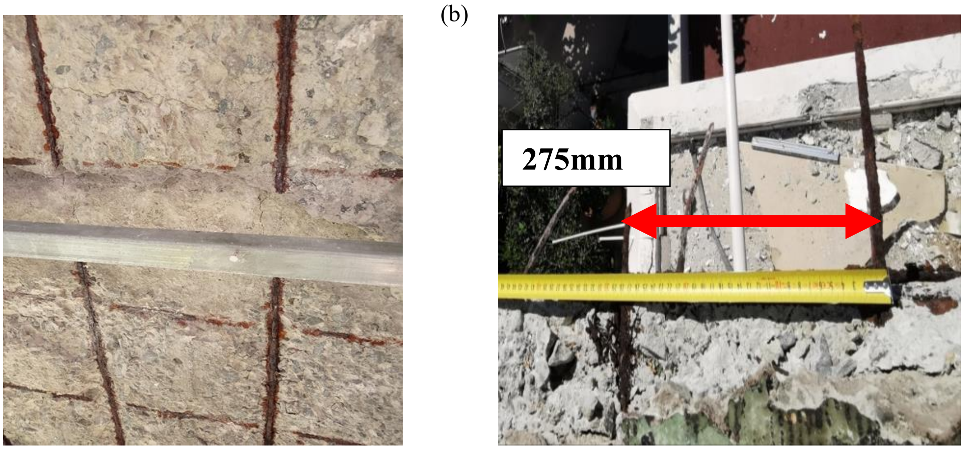

Figure 2: (a) Corrosion of reinforcement (Durable Structural Consulting Engineering); (b)Non-compliance with the minimum reinforcement spacing (Durable Structural Consulting Engineering).

3.2 Destructive and Non-Destructive Methods

3.2.1 Core Sampling

This was performed through undisturbed drilling and extraction of cylindrical concrete specimens using a coring apparatus. The extraction, preparation, and breaking of the cylindrical specimens were conducted according to the standard EN1254-1, (1971) [20]. A total of 5 specimens were extracted, aiming to obtain a representative sample. Subsequently, laboratory testing was carried out in a certified laboratory, involving the determination of compressive strength, carbonation depth, mass and weight of concrete, modulus of elasticity, and Poisson's ratio. The calculation of the in-situ characteristic compressive strength was performed according to the standard EN 13791, (2019) [21]. The sampling procedure and results are demonstrated in Figure (a), (b), (c), Table 1 and Table 2 respectively.

(a) (b) (c)

Figure 3: (a) Concrete core sample; (b) Extraction of core sample and (c) Cantilever's core sample (Durable Engine Structural Engineering LLC Archive).

Sampling location of Core	On-site testing of core N/mm^2 f_c, 1:1 core	*Conversion to compressive strength of cylinder 2:1 f_{ck}, is N/mm^2	**On-site characteristic compressive strength (cylinder) f_{ck}, is N/mm^2	Concrete category according to EN 206-1,2000 [22]
Basement's columns	6,8	5,6	**5,6	***<C8/10
		5,6		
Intermediate floor slab	5,2	4,3	4,3	<C8/10
		4,3		
Roof Slab	17	13,9	3,9	<C8/10
	14,4	11,8		
	4,8	3,9		
		9,9		

Table 1: Concrete core sampling results.

Compressive strength class according to EN 206-1	Ratio of in-situ characteristics strength to characteristics strength of standard specimens	Minimum characteristics in-situ strength N/mm^2	
		$f_{ck,is,cyl}$	$f_{ck,is,cube}$
C8/10	0,85	7	9
C12/15	0,85	10	13
C16/20	0,85	14	17
C20/25	0,85	17	21
C25/30	0,85	21	26
C30/37	0,85	26	31
C35/45	0,85	30	38
C40/50	0,85	34	43
C45/55	0,85	38	47
C50/60	0,85	43	51
C55/67	0,85	47	57
C60/75	0,85	51	64
C70/85	0,85	60	72
C80/95	0,85	68	81
C90/105	0,85	77	89
C100/ 115	0,85	85	98

NOTE 1: The in-situ compressive strength may be less than measured on standard test specimens taken from the same batch of concrete.

NOTE 2: The ratio 0,85 is part of γ_c in EN 1992-1-1: 2004 [23]

Table 2: Minimum characteristic in situ compressive strength [22].

3.2.2 Non-Destructive Testing of Concrete

A total of 5 strikes were conducted on columns and beams of the examined building. The rebounds of the impact hammer on the surface of the concrete were measured (as illustrated in Figure 4), and the average of the measurements was taken to estimate the surface strength of the concrete. The procedure was carried out according to the standard CYS EN 12504-2, (2021) [24]**.** The results according to the above-mentioned procedure are demonstrated in Table 3.

Figure 4: Non-destructive testing of concrete employing Schmidt Hammer.

Location of Concrete	Direction of Hammer	Number of rebounds (R)			Average Value	Equivalent Strength of cube 14-56 days
Column 1 of Basement	>	1=29	5=26	9=28	27,1	18 N/mm^2
		2=27	6=31	10=26		
		3=35	7=23	11=		
		4=24	8=26	12		
Column 2 of Basement	>	1=33	5=23	9=31	27,8	20 N/mm^2
		2=24	6=24	10=27		
		3=26	7=33	11=		
		4=29	8=29	12=		
Column 1 of Intermediate Floor	>	1=33	5=23	9=	33	28 N/mm^2
		2=33	6=	10=		
		3=37	7=	11=		
		4=34	8=	12=		
Column 2 of Intermediate Floor	>	1=47	5=50	9=48	46,6	50 N/mm^2
		2=49	6=48	10=44		
		3=43	7=45	11=		
		4=49	8=38	12=		
Beam of roof slab	^	1=31	5=28	9=27	29,3	15 N/mm^2
		2=28	6=28	10=31		
		3=31	7=29	11=		

		4=30	8=29	12=		
Cantilever of roof slab	^	1=33	5=36	9=34	32,8	20 N/mm^2
		2=31	6=31	10=50		
		3=31	7=36	11=		
		4=33	8=33	12=		

Table 3: Schmidt Hammer results.

3.3 Finite Element Analysis

In modern research finite element analysis is a great tool to assess the structural performance of existing structures. The finite element method is an approximate technique for solving partial differential equations, and therefore, it requires certain criteria to control the quality of the results since the exact analytical solution of the problem is not known [25].

3.3.1 Description of Nonlinear Structural Analysis Method (Pushover)

As per Avramidis et al. (2016) [26], the non-linear static (pushover) analysis method required by EN-1998:3, 2005 [2] relies on the "N2" pushover approach introduced by Fajfar in 2000. Unlike non-linear response history analysis, this method serves to assess the seismic vulnerability of planar frame structures, considering their non-linear behaviour in an approximate and computationally efficient manner. Despite its non-linear nature, the method intentionally employs the superposition principle—typically associated with linear mechanics—for simplicity and practical usability, deviating from the rigor of non-linear response history analysis.

The CSI Analysis Reference Manual [27] states that many approaches are used to improve the precision of structural analysis: (a) Material nonlinearity in Frame elements at discrete, user-defined hinges. The design of the hinge qualities took push over analysis into consideration. According to ASCE 41 and other code-based standards, default hinge characteristics are provided. Discrete fiber P-M3 hinges in ETABS can be allocated to wall elements; (b) Nonlinear static analysis techniques were created specifically to manage the dramatic decline in load bearing capability that frame hinges utilized in push over analyses are known for; (c) Displacement control is possible with nonlinear static analysis techniques, enabling unstable structures to be pushed toward desired displacement objectives.

In the initial phase, the envelope resistance curve of the existing structure was determined in accordance to EN-1998:3, 2005 [22]. The current reinforced concrete building underwent a seismic assessment to verify and certify its unsatisfactory limit state in accordance with Eurocode 8's performance standards and compliance criteria. According to EN-1998:3, 2005 [22], the limit states under examination were:

LS of Significant Damage (SD): The structure is significantly damaged, retaining some residual lateral strength and stiffness, while vertical elements can still sustain vertical loads. Non-structural components are damaged, though partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can withstand aftershocks of moderate intensity but is likely uneconomic to repair.

LS of Damage Limitation (DL): The structure is only lightly damaged, with structural elements prevented from significant yielding, retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage can be economically repaired. Permanent drifts are negligible, and the structure does not require any repair measures.

LS of Near Collapse (NC) Level: The structure is severely damaged, with minimal residual lateral strength and stiffness, while vertical elements may still support vertical loads. Most non-structural components have collapsed, and there are substantial, ongoing drifts. The building is on the verge of collapse and is unlikely to withstand even a moderately strong earthquake.

It is evident from Figures 5(a) and (b), which show the deformed shape along with the developed plastic hinges in the x and y directions, respectively, that the structure's performance level is classified in class c of the ETABS plastic hinges classification, which refers to collapse prevention similar to the NC performance level of EN-1998:3, 2005 [2].

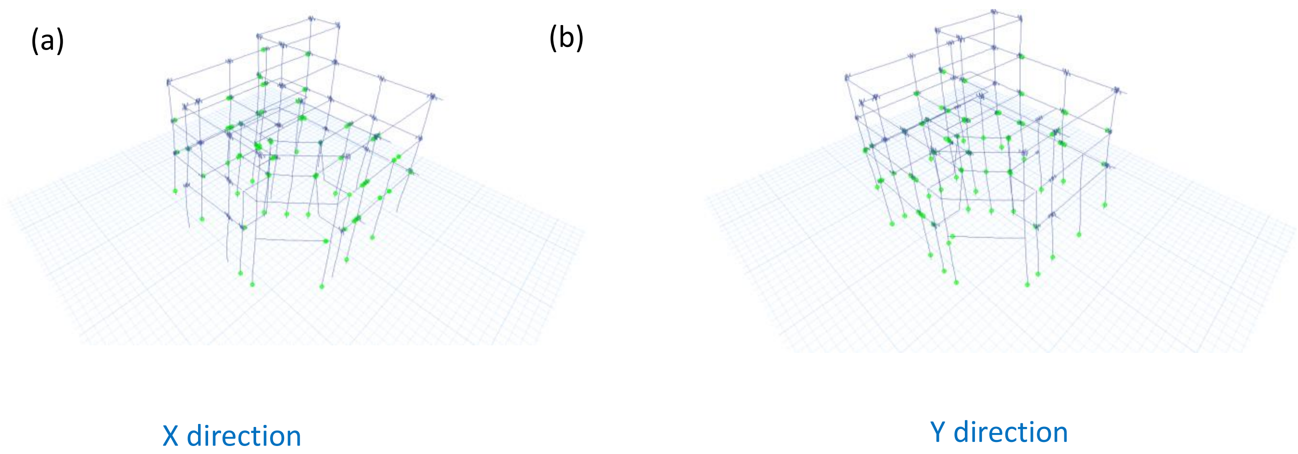

Figure 5: Plastic hinges contribution in (a) X direction and (b) Y direction.

In addition to this, the pushover curves as mentioned earlier, characterize the seismic integrity performance level of the building and facilitate the identification of critical failure points. The process involved modelling the building's geometry, incorporating material mechanical properties detailed in Table 4, and adopting a real case study, illustrated in Figure 6(a). Based on the findings presented in Figure 6 (b) and (c), the structural failure of the building occurred at a notably early stage in both the x and y directions.

	Concrete compressive strength
f_c (MPa)	8/10
Longitudinal reinforcement tensile strength	
f_{y1} (MPa)	410
f_{u1} (MPa)	600
Transverse reinforcement tensile strength	
$f_{y,tr}$ (MPa)	250
$f_{u,tr}$ (MPa)	300

Table 4: Existing reinforced concrete structure materials' mechanical properties.

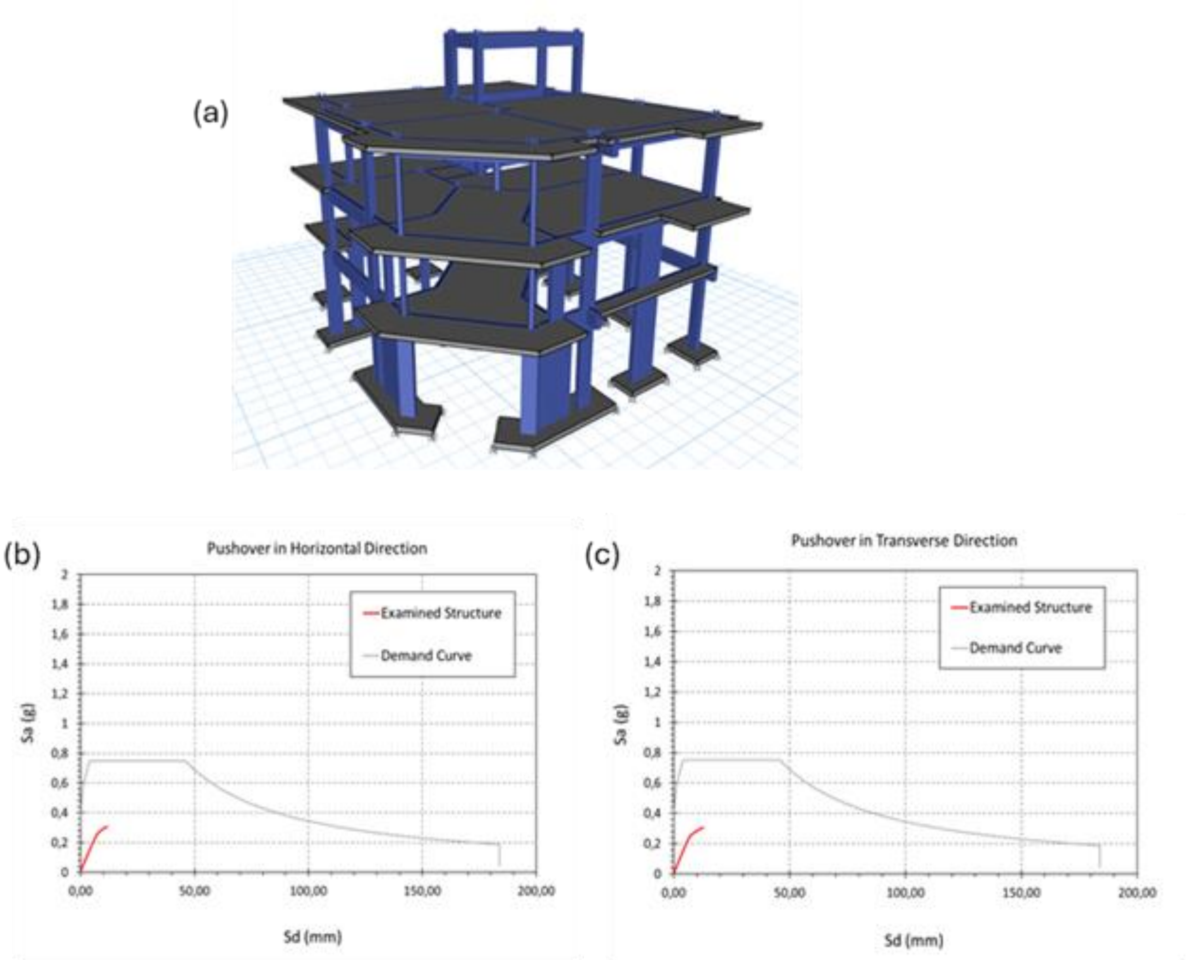

Figure 6: (a) Existing structure structural analysis model; (b) Performance level of existing structure in X direction and (c) Performance level of existing structure in Y direction.

3.3.2 Resolving a Cantilever in Its Original Form—Envelope Resistant Curves – Results of Load-Bearing Capacity

As first step, the cantilever was studied in its original form. It was simulated and examined according to the loading conditions for the purpose of investigating its structural behavior. For verification purposes and in accordance with the Greek Code of Interventions [1] and EN-1998:3, 2005 [2], various types of cantilever retrofitting techniques were evaluated at the critical section to assess the most efficient solution based on increasing the flexural stiffness and ductility of the cantilever.

Following on-site measurements and estimation of the reduction in reinforcement diameter due to corrosion, the cantilever was simulated in its corroded state. Subsequently, three different retrofitting strategies were examined: using steel strips, fiber reinforced polymer composites, and cementitious fiber-reinforced concrete. The structural analysis models were solved using Abaqus software, which is based on the finite element method, and all results are presented in the following figures, diagrams, and tables.

The configuration and development of the physical model and boundary conditions align with the floor plan of the cantilever structure currently under examination. Specifically, the designated area highlighted in blue, as depicted in Figure 7, has been chosen for integration into the Finite Element Analysis (FEA) model. This investigation primarily concentrates on a reinforced concrete strip measuring 100 cm in width. This strip, as depicted in Figure 8 and governed by the outlined boundary conditions, comprises a fixed-supported slab and a cantilever slab.

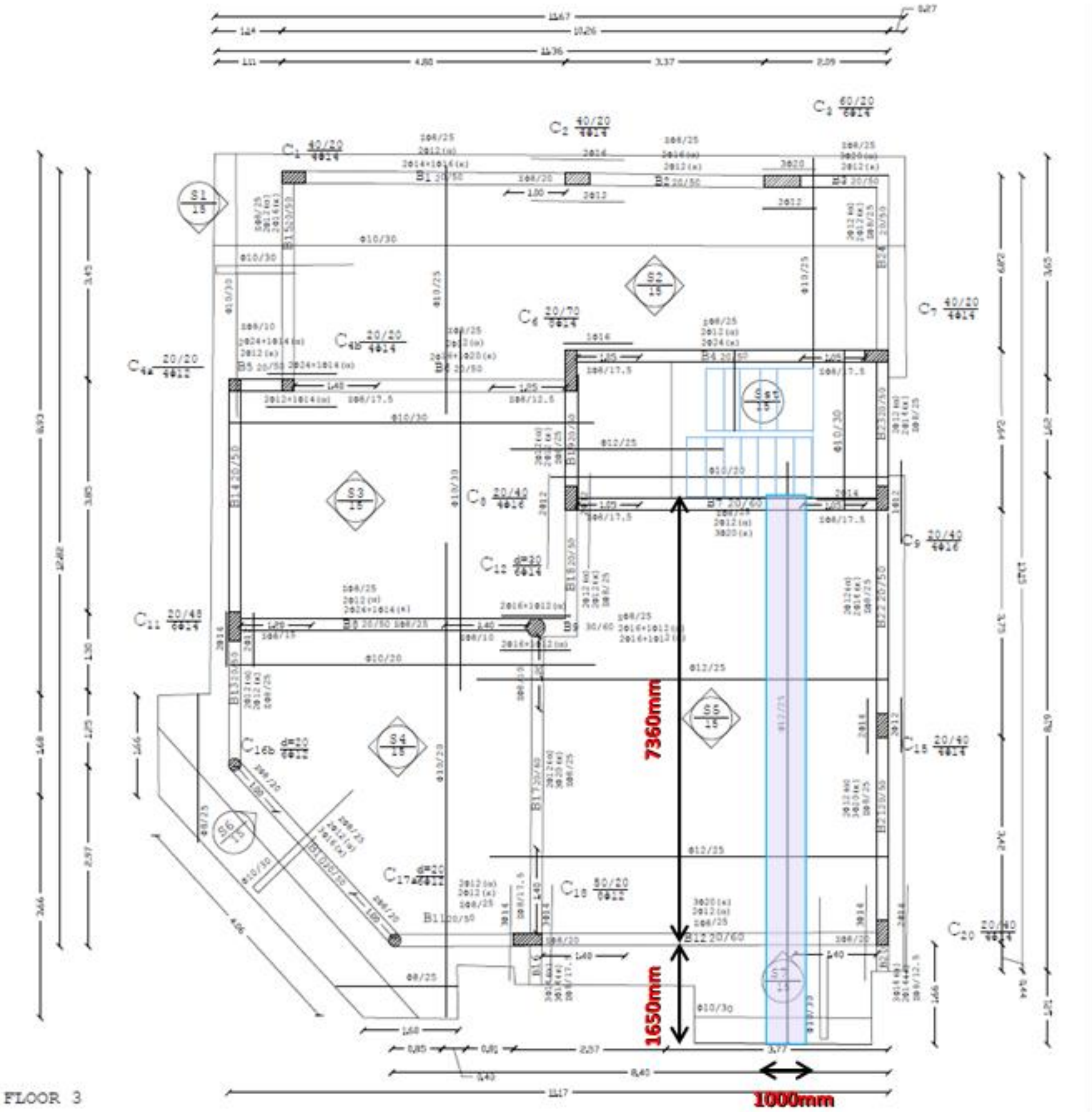

Figure 7: The geometry of the examined reinforced concrete cantilever.

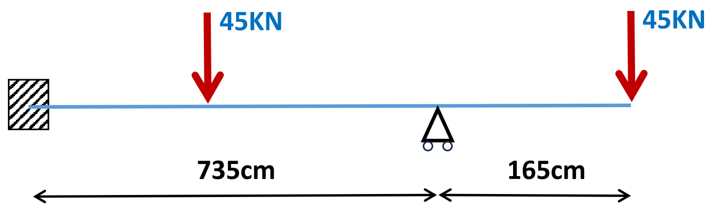

Figure 8: Boundary Conditions.

According to ABAQUS Manual (2023) [28] eight-node hexahedral with reduced integration solid elements were employed to simulate both concrete and the SHCC cover, with typical element sizes set at 20 mm. The material stress and strain behaviors were represented as follows: (a) A Concrete Damage Plasticity Model was utilized for both confined and unconfined concrete; (b) The ascending segment of the uniaxial stress-strain relationship was modeled using a fundamental Hognestad Parabola (1951) [29], achieving the corresponding peak compressive strength at a compressive strain of 0.002; (c) The post-peak descending branch was modeled using the Kent and Park (1981) [30] model, employing values of $\varepsilon_{c,50} = 0.003$ and 0.004 for plain and confined concrete, respectively; (d) The stress-strain curves obtained from experimental uniaxial compression and tension tests were utilized to define the input stress-strain behavior for the SHCC jacketing. The concrete model parameters employed in the analysis are detailed in Table 5. Both plain and SHCC concrete were endowed with elastic properties, with the tangent moduli of concrete and SHCC materials set at $E_{PC0} = 21000$ MPa and $E_{SHCC,0} = 20000$ MPa, respectively, while a Poisson's ratio of 0.2 was assumed. Longitudinal steel reinforcement was represented using beam elements with linear elastic-plastic stress-strain laws, whereas truss elements were employed to model stirrups. Static analysis under force control was conducted in ABAQUS/Standard. Simple supports were introduced along the edges of the cantilever slabs.

Dilation Angle	**Eccentricity**	f_{b0}/f_{c0}	K	**Viscosity Parameter**
36	0.1	1.16	0.67	0

f_{b0}/f_{c0}= biaxial stress ratio; K=shape factor

Table 5: Concrete Damage Plasticity Model Parameters.

With regards to the materials used for FRP and steel strips retrofitting approaches all the mechanical properties given in Abaqus are shown in the Table 6.

Steel Strips	
Property	Value
Young's Modulus (E)	210 GPa
Poisson's Ratio (ν)	0.3
Yield Stress (σ_y)	250 MPa
Yield Strain (ε_y)	0.0015
Plastic Strain (ε_p)	0.002
Technical Data for GFRP Wrap	
Property	Value
Dry Fiber Density (γ_f)	2.56 g/cm^3
Dry Fiber Tensile Strength (J_f)	2500 N/mm^2
Fiber Modulus of Elasticity in Tension (E_c)	72000 N/mm^2
Dry fiber elongation at break	2.08%
Laminate Nominal Thickness	0.168mm
Laminate Nominal Cross Section	168mm^2 per m width
Laminate Tensile Strength	1500 N/mm^2

Table 6: Technical data for Steel Reinforcement and GFRP Wrap [11].

Tie contacts incorporating the geometry of the contact surfaces (surface to surface) without tangential frictional sliding were used to simulate the contact behavior between connected concrete sections. In an effort to capture the post-peak envelope response, the selected type of analysis was nonlinear dynamic explicit time integration. As seen by the on-site measurements, the corrosion modeling was effectively completed by assuming that the bending reinforcement's initially diameter decreased from 10 mm to 6 mm.

Important findings on the most efficient approach and resolution for cantilever retrofitting have been obtained following thorough assessments of each scenario. The failure modes and the resistance curves presented in Figure 10 (a), (b), (c), (d), (e), (f), (g) and Figure 11 respectively indicate that using fiber-reinforced concrete is a more effective solution than using polymer fibers, which allow for significant displacement but reduce deformability. Fiber-reinforced concrete also shows satisfactory displacement and increased strength. Out of all three options that were examined, the retrofitting of steel elements ultimately was selected and put into practice in this specific instance because of its cost-effective and highly effective features. By using the previously indicated technique, deformations are decreased and flexural capacity and stiffness are improved.

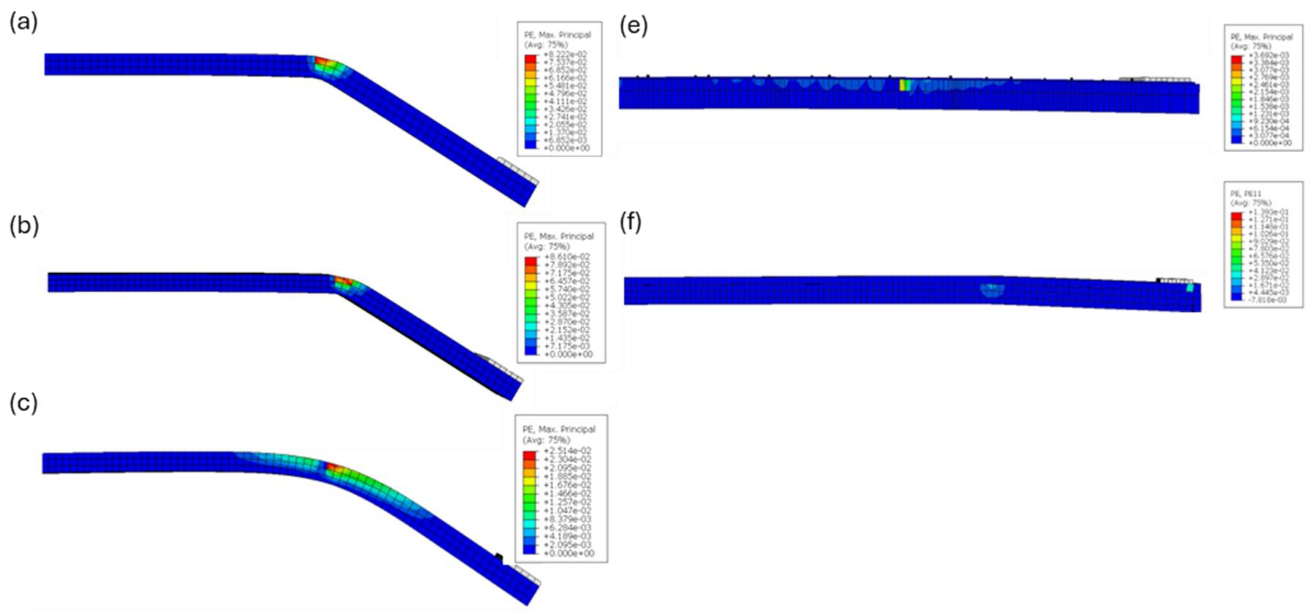

Figure 10: Deformed shape of cantilever considering: (a) to the initial positioning of reinforcement; (b) after reducing the diameter of the main flexural reinforcement, primarily from Φ10 to Φ6, for simulating the carbonation of the existing steel; (c) of retrofitting strategy with GFRP; (d) through the retrofitting with steel strip plates; (e)considering the retrofitted model with ECC jacket.

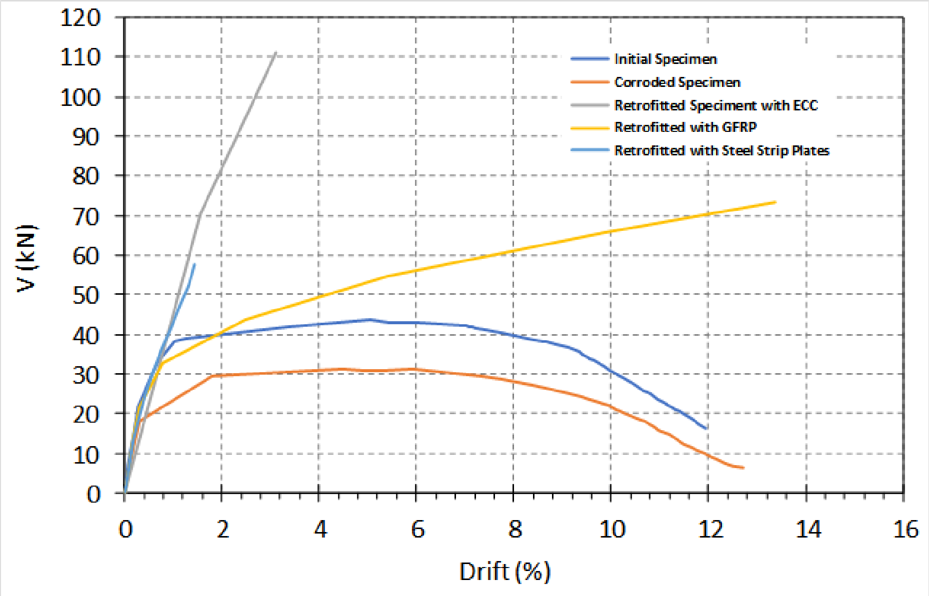

Figure 11: Envelope resistance curve according to the existing structural details of the cantilever, after reducing the diameter of the main bending reinforcement, primarily from Φ10 to Φ6, for simulating the oxidation of the existing steel, after retrofitting with fiber-reinforced polymers, steel strip plates and ECC jacket.

The verification of the cantilever's deflection was based in Eurocode 2 EN 1992-1-1 requirements. More particularly the purpose of establishing serviceability deflection limits for cantilever slabs and other structural components is to avoid excessive deflections that can damage the structure's look, longevity, or operation. Typically, the restrictions are stated in terms of the cantilever's span length (L). The deflection limitations for cantilever slabs as per EN 1992-1-1 [23] may be summed up as follows:

1. Immediate (short-term) Limits of Deflection:
- The immediate deflection limit for cantilevers is usually L/250.
- Under short-term loading circumstances, this indicates that a cantilever slab's deflection shouldn't be greater than L/250 of the cantilever length.

2. Long-term Deflection Limits:
(which account for creep and shrinkage) should also be restricted to L/250.
- The overall deflection, encompassing both short-term and long-term deviations, ought not to above this limit.

It is evident from the model responses that take into account the specimen's yielding drift point at V_y=0.75V_{max} on the ascending branch that only the original, corroded, and retrofitted specimens with steel strip plates are kept close to the L/250 deflection limit. Table 7 indicates that there is a significant difference between the remaining retrofitted specimens and the limit.

Model	$\Theta_{y,FEA}$(%)	Θ_{EC2}(%)
Initial	0.50	0.40
Corroded	0.38	0.40
GFRP Retrofitted	5.50	0.40
Steel Strip Plates Retrofitted	0.95	0.40
ECC Retrofitted	3.10	0.40

Notation: $\Theta_{y,FEA}$= Cantilever drift at Vy and Θ_{EC2}=Drift limit according to EN 1992-1-1 [23]

Table 7: Verification of deflections at yield point of cantilevers Vs limit in accordance to EN 1992-1-1 [23]

4 CONCLUSIONS

The present study demonstrated the history of a reference building, which in the past had experienced failure of a reinforced concrete cantilever from its roof, its current condition, as well as all available structural analysis designs that had been developed during the construction period. The results of seismic assessment (Nonlinear Push Over Analysis) of the building were presented following a series of destructive and non-destructive methods, and subsequently, using the assessment results of the existing construction, critical structural elements with advanced degrees of damage were identified. Innovative reinforcement methods were investigated, and through the use of finite element analysis, optimal repair and reinforcement methods were identified, considering both the safety and durability of the repair process, as well as its compatibility with the overall structure of the building.

In summary, the findings show that the suggested reinforcements-steel plates, fiber-reinforced polymers, and cementitious fiber-reinforced concrete are effective in strengthening the components under investigation, strength and ductility. The concrete's compressive strength levels and the presence or absence of corrosion in the steel reinforcement are essential prerequisites for implementing the techniques mentioned above into practice. When the primary reinforcement has completely corroded and the concrete has carbonated, it is advised to remove and replace the projections. This can be done by rebuilding the reinforcement after conducting a pertinent reinforcement study or by using different replacement techniques that involve using materials like steel and wood while still maintaining an optimal cost-benefit analysis.

Future work is expected to include the development of an experimental program encompassing all the examined cases to validate the FEA models and ensure future parametric studies that will consider the calibrated FEA models.

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