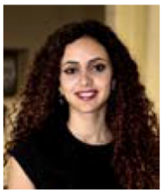
STRENGTHENING AND SHM SYSTEM INSTALLATION ON RC SLAB USING NON-LINEAR FE ANALYSIS

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Re-life of aged load-bearing structures may be more efficient and economical than complete re-building. Nowadays, the strengthening of an RC structure can be designed using non-linear FE analysis, taking a realistic structural behaviour into account. Within the current article, this process is demonstrated in connection with the strengthening of a monolithic RC slab. The slab must be strengthened due to a change of function which increases live load. To ensure satisfactory operation in SLS for increased load, the flexural strengthening of the slab by CFRP strips and bonded steel plates was considered. Static verification of the slab for original and increased loads is performed by linear FE analysis. The load-bearing capacities of the original and the strengthened slabs are also determined by non-linear FE calculation. A comparative analysis of the results highlights the effectiveness of different strengthening methods in increasing the stiffness and SLS capacity of the slab. To perform real-time monitoring of the slab in question, an SHM system is also suggested with the installation of three different types of sensors. This system increases the safety of operation and reduces future maintenance and strengthening costs.

Keywords: monolithic concrete slab, damage diagnostics, 3D nonlinear FE analysis, CFRP strengthening, steel plate strengthening, monitoring, SHM installation.

1. INTRODUCTION

Reinforced concrete slabs are essential structural elements in buildings and bridges. Nevertheless, they might encounter damages under different loadings such as fatigue, aging, overloading, earthquakes, and further natural disasters during their service life (Ahmed, 2016)such as permanent, sustained and transient during their lifetime. Reinforced concrete slabs are one of the most fundamental structural elements in buildings and bridges, which might be exposed to unfavourable conditions such as, impaired quality control, lack of maintenance, adverse environmental effects, and inadequate initial design. Therefore, the resistant capacity of the affected elements would dramatically be reduced which most likely leads to the partial or whole collapse of the structure. Non-destructive testing (NDT. Excessive cracking is one of the most common damages in reinforced concrete slabs, in addition to deflections at midspan (Nejadi, 2022). Therefore, damage detection is vital to guarantee the integrity and safety of civil structures (Fujino, 2002). The conventional procedure is based on human visual inspection, which cannot locate any hidden or minor damages (Kot et al., 2021) which involves inspection, monitoring, and maintenance to support economics, quality of life and sustainability in civil engineering. Currently, research has been con-ducted in order to develop non-destructive techniques for SHM to extend the lifespan of monitored structures. This paper will review and summarize the recent advancements in non-destructive testing techniques, namely, sweep frequency approach, ground penetrating radar, infrared technique, fiber optics sensors, camera-based methods, laser scanner techniques, acoustic emission and ultrasonic techniques. Although some of the techniques are widely and successfully utilized in civil en-gineering, there are still challenges that researchers are addressing. One of the common challenges within the techniques is interpretation, analysis and automation of obtained data, which requires highly skilled and specialized experts. Therefore, researchers are investigating and applying artificial intelligence, namely machine learning algorithms to address the challenges. In addition, researchers have combined multiple techniques in order to improve accuracy and acquire additional parameters to enhance the measurement processes. This study mainly focuses on the scope and recent advancements of the Non-destructive Testing (NDT. Therefore, other non-destructive techniques are necessary to complement visual inspection and can be used to inspect for defects without further damaging the tested component and provide methods for detecting hidden damages (Triantafillou, 1998)such as permanent, sustained and transient during their lifetime. Reinforced concrete slabs are one of the most fundamental structural elements in buildings and bridges, which might be exposed to unfavourable conditions such as, impaired quality control, lack of maintenance, adverse environmental effects, and inadequate initial design. Therefore, the resistant capacity of the affected elements would dramatically be reduced which most likely leads to the partial or whole collapse of the structure. Non-destructive testing (NDT.

In this manner, static linear and non-linear finite element

(FE) models have been extensively applied for structural analysis (Ding et al., 2022)the instantaneous characteristics of the mono-components are firstly extracted from the structural dynamic response and applied to the calculation of likelihood function. Then, the posterior probability density function associated with Bayesian theorem is derived under the assumption of Gaussian prior distribution by using instantaneous characteristics. Afterwards, to calculate the posterior probability density function and improve the sampling efficiency, the delayed rejection adaptive Metropolis-Hastings (DRAM, where FE modelling can perform a global damage detection, and can even predict the possible damages that can appear, for example after a function change that increases the imposed loads on the structure (Durmazgezer et al., 2018). FE analysis is also essential as it verifies the structural elements by examining their load-bearing capacity to withstand the applied dead and live loads; for example, an FE model can predict residual deflection (Zhao & Ye, 2022). For instant, Scamardoa non-standard finite element (FE et al. (Scamardo et al., 2022)a non-standard finite element (FE used FE models to approximate the directions of in-plane tensile cracks, and Foti (Foti, 2014) Italy developed a 3D FE model of Normand tower town of Craco (Matera, Italy) to evaluate the structural behaviour of the tower.

With the decreasing life of existing structures, research and industry are continuously striving to find innovative and cost-effective methods that can increase the life span of these structures to satisfy the needs of owners and users (Khalifa et al., 1999). Strengthening could be achieved by section enlargement, external post-tensioning, externally bonded steel elements, advanced fibre-reinforced polymer (FRP) composites, textile reinforced concrete (TRC), and near-surface mounted (NSM) system, or a combination of these techniques (Heiza et al., 2014).

External bonding of steel plates to damaged RC structures is an important method. It has proved to be efficient and a well-known strengthening technique similar to composite materials like carbon fibre reinforced polymers which can be an alternative to steel plates (David et al., 1998). Steel plates have been used for many years due to their simplicity in managing and applying and their effectiveness for strengthening (Brosens & Van Gemert, 2014).. Externally bonded steel plates to rehabilitate reinforced concrete members were first reported in 1964 in Durban, South Africa (McKenna & Erki, 1994). In 1972, externally bonded steel plates were employed on the tension face of a concrete T-beam bridge in France to correct a permanent deflection of 80 mm due to creep (Merter & Ucar, 2013). The bonding of the steel plates or steel flat bars to the surface of concrete members is ensured by epoxy adhesives, and, in some cases, extra fastening is provided using dowels or bolts glued to the holes drilled in the concrete members (Garg et al., 2020). In the case of RC slabs strengthening, steel plates are used to enhance the bending resistance of structural members. They can be applied to flexural zones of the slab to improve its positive flexural capacity (Pryl et al., 2015).

In recent years, the strengthening and retrofitting of structural members using externally bonded carbon fibre reinforced polymer (CFRP) materials has achieved much attention (Lima et al., 2016). Initial developments of the FRP-strengthening technique took place in Germany and Switzerland (Triantafillou, 1998); besides, it revolutionized the strengthening field and appears to be a superb solution for the retrofitting and strengthening of reinforced concrete elements because of its unique properties, including high strength-to-weight ratio, high fatigue endurance, environmental degradation, and corrosion resistance and user-friendliness (Lima et al., 2016). CFRP has proved to increase the flexural stiffness of the studied samples and increase the axial, flexural, or shear capacities (Azizi et al., 2019), ductility, remaining fatigue life, and durability against harsh environments (Lima et al., 2016). Moreover, Piątek and Siwowski noticed a 22–33% increase in RC beams‘ ultimate load-bearing capacity strengthened by CFRP (Piątek & Siwowski, 2022). Several vital contributions involving the design of RC elements with externally bonded FRP laminates are present in the literature (Pryl et al., 2015, Khalifa et al., 1999, Szabó & Balázs, 2007).

The installation of wireless sensors as Smart Health Monitoring (SHM) tools for buildings is getting boom day by day. The applications of these sensory systems can be visualized by their implementation as an SHM system in the world's famous San Pedro Apostol Church in Peru (Macwilliam & Nunes, 2019). The effects of specific parameters like temperature, strain, acceleration, and wind, were monitored and analysed in an ancient building of the ottoman empire (Urla, Izmir) by installing different sensors constituting the SHM system (Kilic, 2015). Large-scale SHM application can be observed on China's 600 m tall canon tower, where the existing condition and associated damages were examined based on SHM data (Sivasuriyan et al., 2021). SHM system helps carry out real-time monitoring of multi-story buildings subjected to dynamic loading close to coastal areas where structures can be subjected to hydrodynamic forces produced due to tsunami (López et al., 2017). The application of 3D linear and non-linear FE techniques for stress and deformation analysis of buildings was utilized for hospital buildings where piezoelectric sensors were used for proper SHM of buildings (Roghaei & Zabihollah, 2014)the building may suddenly goes to failure, requiring a reliable yet efficient health monitoring system. An array of piezoelectric sensors is mounted at desired location to measure the deformation and stress at critical points. The voltage generated by piezoelectric sensors is sent to computer via a data acquisition system. Measuring and monitoring the trend of changing sensors voltages indicate the probability of existing damages and the rate of propagation. The performance-based seismic is reported based on the nonlinear static analysis (pushover. Similarly, the same sensors have already been used for damage evaluation of a building based on impedance measurement (Park & Inman, 2007) and measure the impedance of structures by monitoring the current and voltage applied to the piezoelectric transducers. Changes in impedance indicate changes in the structure, which in turn can indicate that damage has occurred. An experimental study is presented to demonstrate how this technique can be used to detect structural damage in real time. Signal processing methods that address damage classifications and data compression issues associated with the use of the impedance methods are also summarized. Finally, a modified frequency-domain autoregressive model with exogenous inputs (ARX. In case of multi-story buildings, the primary consideration is given to the vibration response mode of buildings that affects the period of building and produces severe stress concentrations at different floor levels, which ultimately produces severe cracks in the slabs of different floors. These vibration response modes provide installing various sensors at the prescribed locations (Hasan et al., 2002).

FEM modelling and simulation are currently used for long-term monitoring of buildings where the dynamic behaviour of the building is observed by executing the operational ambient survey (Pierdicca et al., 2017)it has been possible to follow the dynamic evolution of the structure, using Finite Element Method (FEM. Nowadays, online damage monitoring tools are also available to observe cracks and deflections of structures without the application of measurable input. These tools use the Operational Model Analysis technique for monitoring (Pierdicca et al., 2016). Rainieri and Fabbrocino used model-based damage detection algorithms to identify civil structures' modal parameters based on the data observed from the SHM system (C. Rainieri & Fabbrocino, 2011a), (Rainieri & Fabbrocino, 2011c), (Rainieri & Fabbrocino, 2011b), (Rainieri & Fabbrocino, 2015), (Wierzbicki et al., 2020). The significance of SHM monitoring of buildings can be easily visualized because 102 papers were presented in an international workshop held at Stanford University in September 1997 (Fawad et el., 2019). The technical presentations highlighted the advancements in sensing technology, modelling, and diagnostic methods, system integration, and applications.

Considering the importance of monitoring systems, three different sensors were used in this research to monitor the deflection, strain, temperature, and cracks on different floors in a concrete building, especially on the 2nd-floor slab. The applied sensors include Inclinometers for deflection monitoring, optical Fibre Bragg Grating (FBG) sensors for strain and temperature monitoring, and crack monitoring smart film for monitoring of cracks. The applications of these sensors can be found in the existing buildings highlighted in many research works (Avci et al., 2021), (Zhang et al., 2014), (Zhang & Xiong, 2018)as an important component of buildings and structures, require inspection for the purposes of crack detection which is an important part of structural health monitoring. Now existing crack detection methods usually use a single technology and can only detect internal or external cracks. In this paper, the authors propose a new sensing system combining BOFDA (Brillouin optical frequency-domain analysis, (Juraszek, 2019).

2. DESCRIPTION OF THE ANALYSED BUILDING

The analysed structure resembles a 7-story monolithic reinforced concrete office building located in Hungary. The building height is approximately 20.75 m above the ground and its overall floor plan dimensions are 25.13×51.96 m. The building sits on an 800 mm thick slab foundation. Floors 2-7 denote 30 cm thick reinforced concrete slabs and 3.10 m height and 20 cm thick shear walls with openings. The ground floor is 3.5 m high and includes a green roof of 30 cm thickness. The authors examined the monolithic RC slab on the 2nd floor *(Figure 1)* Within the above structure due to reclassification from category B to category E, which increased the imposed load from 3 kN/m^2 to 7 kN/m^2 at the customer's request.

3. LINEAR FE ANALYSIS

There are some methods for the analysis of structures. These methods may be linearly elastic or non-linear as well as static or dynamic (Merter & Ucar, 2013). In the context of condition control and damage assessment of civil engineering structures, static load tests are widespread (Mahowald et al., 2010); they can also be used for research studies to explore the progressive collapse behaviour of structural members and sub-structural structures under severe deformations (Garg et al., 2020b).

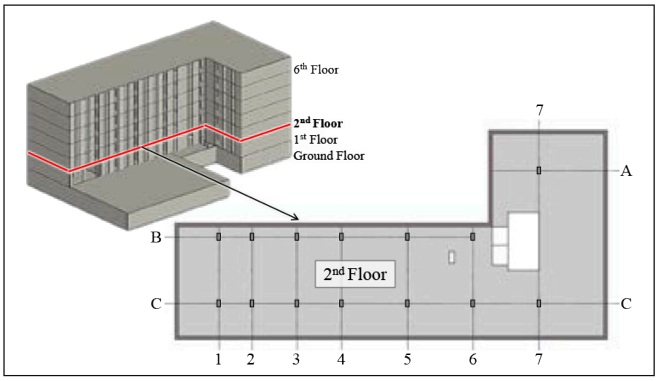


Fig. 1: General view of the building and the location of the examined slab

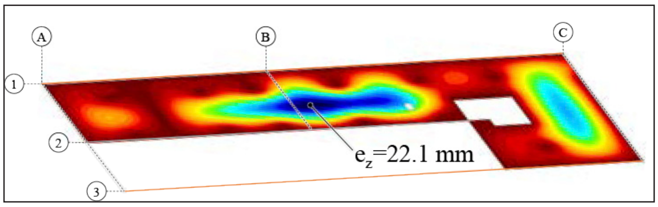


Fig. 2: Deflection of the slab in case of increased live load

In the first part of the research, the authors carried out a 3D linear static FE analysis for the whole building using AxisVM software to obtain the examined slab's internal forces, vertical deflections, and load-bearing capacity before and after function change. In the numerical model, the slab is supported by load-bearing concrete walls and columns. The structural elements were modelled as shells or line elements. Table 1 shows some details about applied materials and loads. The 3D model was also exposed to wind loads according to Eurocode.

Table 1: Material grades and characteristic loads used in the linear numerical model

Concrete grade	C25/30
Steel grade	S500B
Concrete density (self-weight)	25 kN/m^3
Permanent load	2,00 kN/m^2
Live load *before* function change	3,00 kN/m^2
Live load *after* function change	7,00 kN/m^2
Live load on stairs	3,00 kN/m^2
Live load on green roof	5,00 kN/m^2
Snow load	1,00 kN/m^2

Based on the calculated internal forces, the applied reinforcement of the slab was satisfactory for bending, torsion, and shear also after increasing the live load. Deflections were determined on the cracked slab, considering the actual reinforcement, as well as the shrinkage and creep of concrete. The maximum calculated deflection was 18.4 mm and 22.1 mm in case of original and increased live loads, respectively. Both values were less than the limit value (span/250 = 32 mm) from EC2. Calculated deflections are illustrated in *(Figure 2)* in case of the increased live load level. The maximum crack width was 0.35 mm for the original live load, but after increasing the load it increased to 0.43 mm, exceeding the 0.4 mm limit value from EC2. Based on this, we concluded that the slab was not satisfactory for SLS after increasing the live load, which necessitated the strengthening, for which we have analysed two different solutions.

4. NON-LINEAR FE ANALYSIS

Linear elastic analysis can only give a vague idea about the distribution of internal forces, whereas 3D non-linear FE analysis helps learn the plastic moment redistribution (Pryl et al., 2015) and the collapse load of the slab. Since geometric nonlinearity and behaviour of materials beyond the linear elastic limit are considered, it can provide an effective way to determine and evaluate the realistic behaviour of RC structures by civil engineers (Merter & Ucar, 2013). The non-linear behaviour deals mainly with changes in the bending stiffness caused by cracks (Mahowald et al., 2010). The non-linear analysis is either force-controlled or deformation-controlled.

In our research, force-controlled analysis was used, where the load was applied step by step until the maximum load was attained or the collapse of the slab was reached (Dasgupta et al., 2020). In this context, ATENA 3D non-linear analysis software was used to model the examined slab, first in its original state, then after strengthening.

One can benefit from the symmetricity feature in ATENA 3D, so given this, only a portion of the slab was modelled (S2) instead of the whole second-floor slab *(Figure 3)*. Analysed slab portion is 300 mm thick and has 4.00×14.4 m floor plan dimensions. The columns were implemented as surface supports for the slab, acting on 30 mm thick steel plates; this gives more realistic results than applying the supports directly to the concrete. Shear walls were also modelled as fixed surface support acting upward on 30 mm thick steel plates with dimensions identical to the cross-section of the corresponding wall.

According to the actual slab reinforcement, top and bottom rebar mesh of Ø10/200 mm were used in both directions in the slab. In addition to a top reinforcement of Ø25/200 mm was also applied around the column heads in both directions. The top and bottom basic meshes of steel grade S500B were modelled as 'smeared' reinforcement; this method assigns the reinforcement in an average sense to the element by specifying an appropriate reinforcement ratio P [%] which can be calculated: .

ATENA 3D offers a variety of material models that can be used. Based on the experiences of Roszevák and Haris (Roszevák & Haris, 2021), the concrete was modelled as *3D Non-linear Cementitious2* material using f_{cu} = 30 N/mm^2 concrete cube strength, E = 31 kN/mm^2 elastic modulus, f_t = 2.6 N/mm^2 tensile strength, and G_f = 5.5×10^{-5} MN/m specific fracture energy (Roszevák & Haris, 2021). For the modelling of supporting steel plates, the *3D elastic isotropic* material was selected, with an elastic modulus of E_s = 210 kN/mm^2 and a Poisson's ratio of μ = 0.3. The modelling was done using a 0.2 m average mesh size; thus, 19480 finite elements were created altogether. *Table 2* demonstrates the boundary conditions employed for the examined slab segment (S2).

In the strengthened slab, the applied CFRP strips and high yield strength steel plates were introduced as *3D elastic isotropic* material with their specific properties.

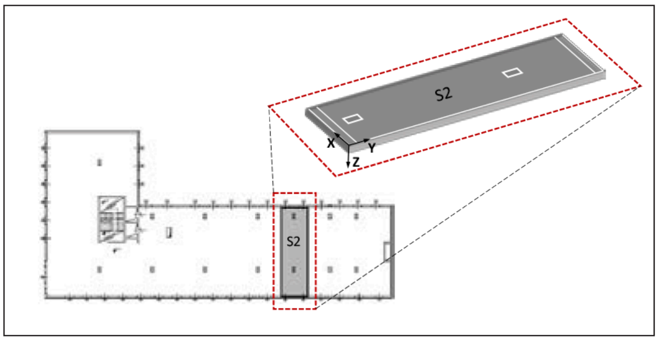


Fig. 3: Location and bottom view of the analysed slab segment (S2)

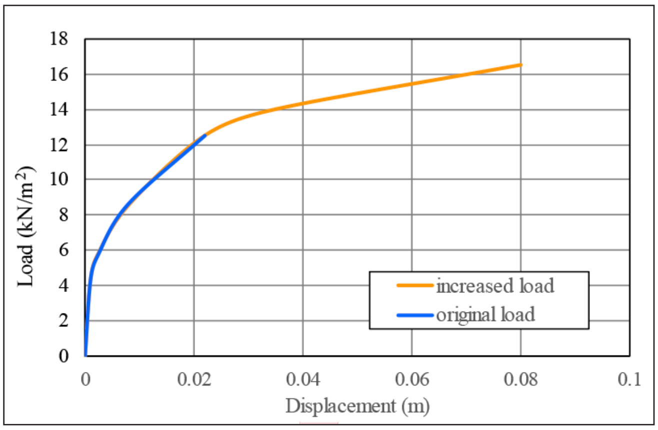


Fig. 4: Load-displacement curves of the slab for original (grey) and increased load (red)

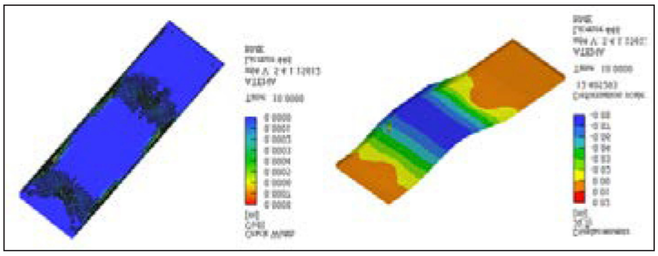


Fig. 5: Calculated crack width and deflection of un-strengthened slab S2 after increasing the load

Due to the applied advanced material models and finite element types, the results of the non-linear analysis may be more accurate than the linear analysis mentioned before. To make the two methods comparable, the original slab was also modelled for the original and increased live load in ATENA and confirmed that the load-bearing capacity of the slab is satisfactory in both cases. However, ATENA calculation also indicated some problems in SLS for the case of increased load *(Table 3)*.

Table 2: Support conditions for the analysed slab segment (S2)

Macro-element/Surface	Support direction		
	x	y	z
1/6	Fixed	Free	Free
1/4	Fixed	Free	Free
2/6	Fixed	Fixed	Fixed
3/6	Fixed	Fixed	Fixed
4/6	Fixed	Fixed	Fixed
5/6	Fixed	Fixed	Fixed

Table 3: *Differences between AxisVM and ATENA results in SLS*

Criteria	AxisVM	ATENA
Original load		
Max. deflection [mm]	18.4	22.0
Max. crack width [mm]	0.35	0.31
Increased load		
Max. deflection [mm]	22.1	80.0
Max. crack width [mm]	0.43	0.80

As described before, the total load acting on the slab increased from 12.36 kN/m^2 to 16.36 kN/m^2 due to the change in the function of the second floor; this caused a serious increase in deflections and cracks appeared. Nevertheless, the slab maintained its load-bearing capacity since the load-displacement curves *(Figure 4)* show no failure at the maximum load levels. The maximum deflection and crack width values calculated by ATENA for the case of increased load (*Figure 5*) are significantly higher than the AxisVM values. This is mainly because ATENA is able to consider the yielding of steel bars and plastic behaviour of the RC material, while AxisVM is always taking elastic material behaviour into account, regardless of the level of internal forces. Calculated maximum deflection and crack width values imply that limit values according to EC2 were exceeded, again indicating the need for strengthening the slab to maintain its safety and reduce the possibility of crack and deflection occurrence.

5. ANALYSIS OF THE STRENGTHENED SLAB

5.1. Strengthening by CFRP strips

The need to strengthen the concrete slab in question was revealed previously; therefore, the authors considered two different strengthening solutions. The first one was the application of bonded, pre-impregnated Carbon Fibre Reinforced Polymer (CFRP) strips. The second one was bonded, high-strength steel plates, both applied parallel to the main load-bearing (*y*) direction as additional external flexural reinforcement. Two separate numerical models were prepared in ATENA software for these two solutions, taking advantage of non-linear calculation into account.

HM-3.0T carbon fibre laminate strips fixed by HM-120CP adhesive were used in the first model. The applied strips are 3 mm thick and 100 mm wide, and they were modelled as a group of 26 strips of length 4.00 m and 50 mm spacing. CFRP strips were applied to the bottom surface of the slab at the position where the maximum deflections occurred *(Figure 6)*. CFRP material was modelled as *3D elastic isoelastic* material with a mean tensile strength of 3100 N/mm^2, a modulus of elasticity of 65 kN/mm^2, an ultimate strain of 1.7%, and a density of 1.6 g/cm^3. A perfect connection was assumed between the concrete and the CFRP strips, but the maximal contact stresses were manually verified and found to be less than the stress-causing delamination.

The construction stages were modelled to obtain more realistic internal force distribution for the strengthened slab. In the first load step only the self-weight of the slab was applied to the un-strengthened structure. After that, the CFRP strip macro-elements *(figure 6)* were activated, and the rest of the loads were applied in several steps. During the analysis,

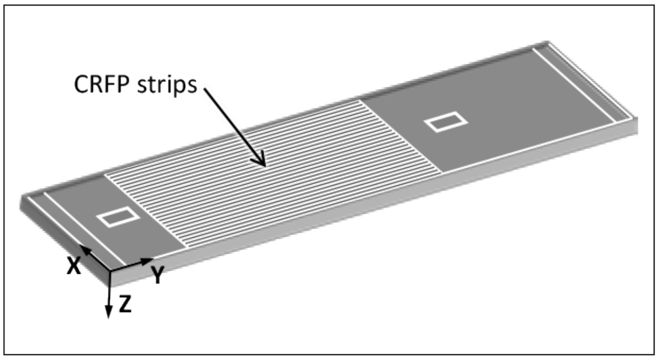


Fig. 6: View of the applied CFRP strip macro-elements

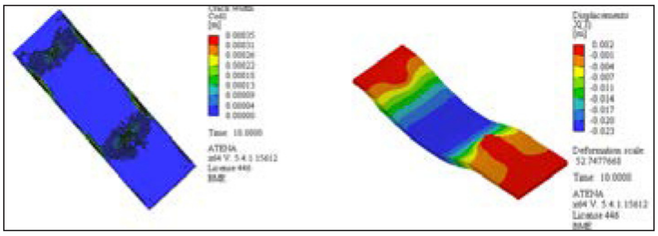


Fig. 7: Calculated crack width and deflection of slab S2 after applying strengthening by CFRP strips

it was assumed that any existing cracks inside the slab are injected before strengthening to restore the original stiffness of the slab. Considering the CFRP strengthening the maximum deflection and crack width values were significantly decreased. *(Figure 7)* shows that deflection decreased to 23 mm and crack width to 0.35 mm, which are below the corresponding limit values.

(Figure 8) illustrates the load-displacement curves for the un-strengthened and CFRP strengthened slab. Thanks to the CFRP material's linear elastic behaviour, the strengthened slab maintained most of its initial stiffness at higher load levels resulting in smaller deflection and crack width. Curves in *(Figure 8)* are plotted only until the maximum load was applied to the slab but increasing the load until failure revealed that the strengthened slab possessed significantly higher load-bearing capacity too. Although the first cracks appeared at the same load level in both cases, internal steel bars started to yield much later, and the post-cracking stiffness was much higher in case of the strengthened slab.

5.2. Strengthening by steel plates

Another numerical model was created to analyse the effect of bonded steel plates as additional external flexural reinforcement at the bottom of the slab. For this purpose, S690QL high yield strength steel plates were used, with a minimum yield strength of 690 N/mm^2, a tensile strength of 770 N/mm^2, and 14% minimum elongation before fracture. Steel plates were used as a group of 8 strips of 500 mm width and 6 mm thickness *(Figure 9)*. In ATENA, *3D elastic isotropic* material was assigned to steel material with an elastic modulus of E_s = 210 kN/mm^2 and a Poisson's ratio of μ = 0.3. Between the concrete and the steel plates, a 3D interface contact surface was used to represent the epoxy resin layer. The connection was assumed to be perfect, but again it was manually verified that no delamination of the steel plate would occur. Construction stages were modelled the same way as described for CFRP strengthening.

Compared to the un-strengthened slab, the maximum deflection was significantly reduced by applying external steel plate reinforcement to 24 mm. In contrast, the maximum

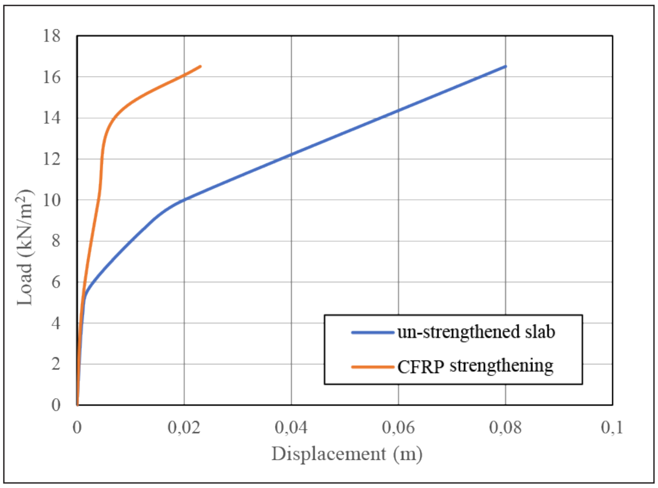


Fig. 8: Load-displacement curves of the un-strengthened and CFRP strengthened slab for increased load

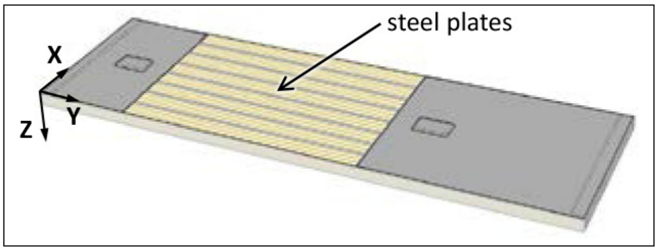


Fig. 9: Load-displacement curves of the un-strengthened and CFRP strengthened slab for increased load

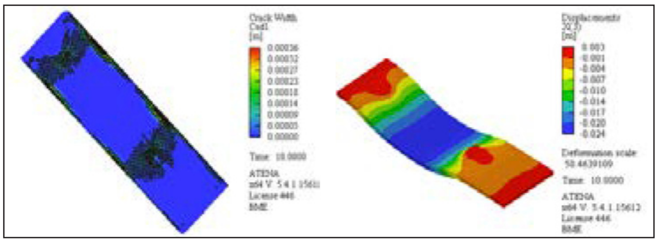


Fig. 10: Calculated crack width and deflection of slab S2 after applying strengthening by steel plates

crack width reached 0.36 mm, meaning that SLS requirements are fulfilled. Calculated cracks and deflections are presented in *(Figure 10).*

(Figure 11) illustrates the load-displacement curves for the un-strengthened and steel plate strengthened slab. In case of the strengthened slab, the initial stiffness was mainly maintained after cracking of concrete. The first jump in the load-displacement curve of the strengthened slab was related to the yielding of internal steel reinforcement, which was later followed by the yielding of external steel plates. Regardless of these, the stiffness of the strengthened slab remained much higher than the original slab, resulting in smaller deflections and crack width. Again, the curves in *(Figure 11)* were plotted only until the maximum load was applied to the slab. But increasing the load until failure revealed that the strengthened slab possesses significantly higher load-bearing capacity.

5.3. Comparison of the proposed strengthening methods

The previous analyses showed similar improvements in deflection and crack width for the slab in case of both strengthening methods. *Table 4* illustrates that CFRP strengthening gave slightly better results in decreasing the deflection and crack width, but the difference was small. CFRP material is more expensive than steel, but on the other hand, the construction is faster and easier in CFRP, resulting in lower labour and machinery costs. Yooprasertchai found that FRP materials are competitive compared to steel materials (Yooprasertchai et al., 2022). Ibach Bridge in the county of Lucerne, Switzerland, was also strengthened by CFRP sheets instead of steel because it suffered accidental damage due to a prestressing tendon in the outer web with some of its wires completely severed by an oxygen lance, and, as a result, the load capacity of the bridge decreased. It was estimated that 175 kg of steel plates would have been required to strengthen the bridge to its original design load. However, only 6.2 kg of CFRP sheets were used instead (McKenna & Erki, 1994).

(Figure 12) compare the load-displacement curves of CFRP strengthened and steel plate strengthened slabs. Because of the linear elastic behaviour of CFRP material, the stiffness of the CFRP strengthened slab remains higher as the load was increasing. The capacity of the slab in SLS was similar in case of both strengthening methods. However, after reaching the maximal load and un-loading the slab again, we had less residual deformation left using CFRP strengthening because of the linear elastic behaviour of CFRP material.

Based on the above considerations, the application of CFRP strengthening was proposed, as it was feasible in terms of price and was better in terms of overall structural performance.

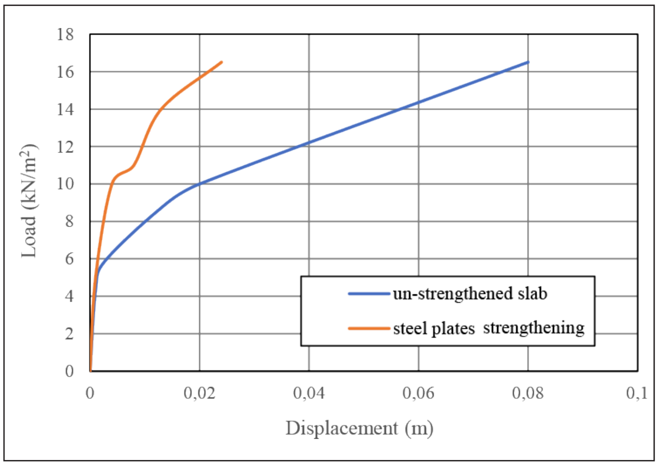


Fig. 11: Load-displacement curves of the un-strengthened and steel plate strengthened slab for increased load

Table 4: Calculated deflections and crack widths in case of slab strengthened by steel plates and CFRP strips in case of increased load

Criteria	Original slab	Slab strengthened by CFRP strips	Slab strengthened by steel plates
Max. deflection [mm]	80.0	23.0	24.0
Max. crack width [mm]	0.80	0.35	0.36

6. PROPOSAL OF THE SHM SYSTEM

The changing trend of the construction industry and the boom of the building monitoring system is urging the owners of this building to install the proper monitoring system for the whole building. As the building is quite young, the owners are not planning dismantling or complete reconstruction in the near future and want to monitor the performance of the building for at least the next 15 years. Therefore, the installation of the

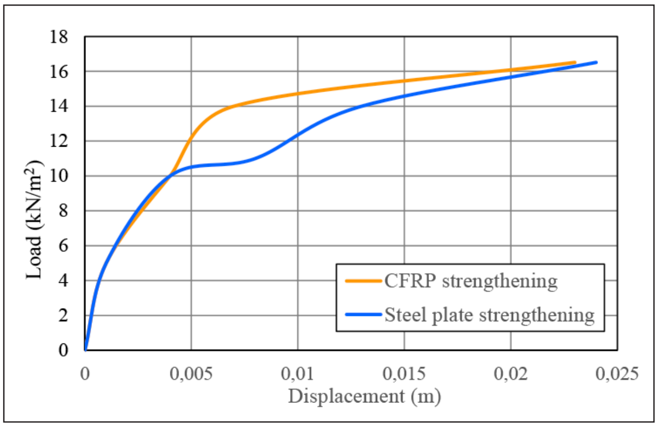


Fig. 12: Comparison of load-displacement curves of the CFRP strengthened, and steel plate strengthened slabs for increased load

SHM system is not only fulfilling the needs of the owners but also provides a cost-effective and user-friendly solution for monitoring the building during its entire life cycle.

Before going for the full-scale monitoring of the whole building, owners are interested in carrying out a study by just installing an SHM system to the problematic areas of the building, which are highlighted on the second floor in this study. Considering the scenario, the authors decided to design the SHM system for the second floor of the building initially, and upon the satisfaction of the owners', results could be projected to the whole building with minor modifications.

In order to install the SHM system, critical zones of structure in terms of deflection and crack width are identified. These identified locations are considered the installation points of sensors. Damage detection data is usually used to establish a new SHM system, but in this research, numerical analysis data will be used to model this system. For this task, linear analysis done by AxisVM software provides the locations of critical points where the parameters mentioned above have the maximum values. These locations are marked for the installations of different sensors, identified as the sensory system for SHM of the building that is capable enough to highlight the main causes of the damages and warn the concerned authorities of any preventive measures well before time.

6.1. Location identification of deflection sensors

Deflection is calculated as per the real loading and support conditions, considering the actual reinforcement inside the slab, as along with the shrinkage and creep of concrete.

Measurement of span displacements usually requires a fixed point of reference. Although displacements without contact (laser, radar) can be measured, but certainly not at many points at the same time. In addition, some places (especially near the beam-column joints) will show such small displacements that will not be measurable. Alternatively, the authors suggest an indirect measurement with inclinometers. The angles of rotation of a bending span can be measured relatively easily and accurately. In this way, it can be assessed whether the span has lost its stiffness. This may be due to repeated cracks, progressive corrosion, or deterioration of the concrete's mechanical properties.

For the whole floor level, two sets of sensors are recommended, one set at the centre locations of grid B and C, where six sensors will cover the maximum deflections of each of the grids 1-6, and a second set along the grid A, where two sensors will calculate the deflection along the centreline of the grid. *(Figure 13)* shows the locations of these sensors. In the cross-section, sensors are located along the slab's length. Sensors of each set are connected by a single cable to the central data acquisition system. In this way, the measured angle of rotations will be giving the deflected shape to each section along with the whole floor level. So, a total of eight sensors are recommended for the entire floor.

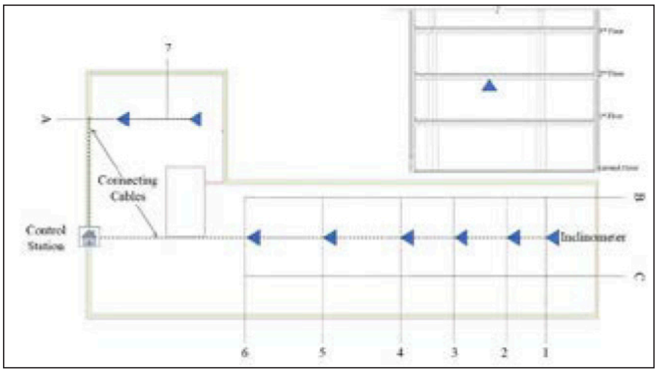
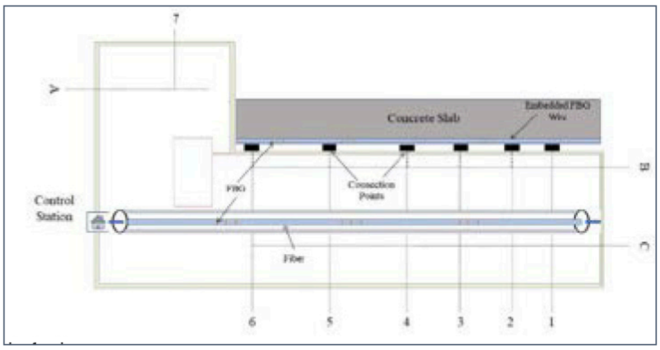


Fig. 13: Planned location of inclinometers along the second floor

6.2. Location identification of strain and temperature sensors

In such a condition where excessive loads might cause problems for the floor slab, strain (axial and bending), and temperature sensors are certainly of significant importance. In the case of this building static and transient strains, and temperature gradient at the height of each floor level are critical factors that need attention. The traditional method of strain measurement involves a dense network of strain gauges which not only increases the size of the test set-up but also produces a very complex electric wiring network. Moreover, strain gauge installation is not favourable in case of the existing conditions of the subject slab, so authors suggest the installation of optical Fibre Bragg Grating (FBG) optical wire. Their installation is very easy as they can be inserted directly into structural elements. Considering their ability to measure strain and temperature using a single wire, their noise tolerance, and their efficiency, these sensors are highly recommended in this case. The location of this wire is suggested along with the points of maximum stress concentrations as the maximum stresses are observed at each of the connections of the beams between the columns, so, this is why these locations require monitoring of the strain values. So, the optical fiber wire will run along the length between the columns, connecting it to the central control unit of the slab. Locations of FBG optical wires are shown in *(Figure 14)*. These optical fiber wires will be connected directly to the reinforcement inside of the concrete.

Fig. 14: Planned locations of FBG fiber optical wires



6.3. Crack width monitoring sensors

From the results of linear and 3D non-linear analyses, it can be observed that some damages like concrete cracks may occur to the slab (regardless the level of maximum applied load). Not to mention other possible future problems like corrosion of the reinforcement. The most important thing here is to monitor the progression of existing cracks and the formation of new ones. This can be done in various ways, but the most advanced approach is to use the smart film method. This smart film contains a group of insulated enamel copper wires. It forms the shape of an optical fiber that sends the electrical signal among different terminals of the film that are connected to a secondary processor at different intervals along the film where the signals are simultaneously detected. Initiation of a crack will break the wires carrying these signals and instantaneously results in the failure of signal detection at the other end. The width of the crack in this system is calculated by the diameter of copper wires used for crack detection using the relation shown in the following equation:

$$R_m = \rho \frac{l - \Delta l}{A} + \rho \frac{\Delta l}{A'}$$

Where *A* is the area of wire before the crack opening, *A'* is the area after the crack opening, is the resistivity of the bar which is taken as 1. 7x10^{-8}Ω.m, l is the length of the section before fracture and D*l* is the length of the fractured section. Using the diameter of the wire before and after the crack opening, crack width can be calculated. The benefit of using this film is its efficiency for the short-term and long-term crack monitoring of buildings without disrupting its regular usage. Further, specific dynamic properties of the material can also be monitored in real-time with this film.

An important task is to identify the location of the installation of smart film. From the results of linear analysis, it is evident that the stress concentrations are producing severe cracks at the beam-column joints. So, the film will be glued externally at the bottom of the slab surrounding these joints throughout the floor. The film will emerge from the central control station and will pass through each of the secondary processors, and after collecting the data from each of the secondary processors, it will end at the other end of the control station. The film will be glued, protected by the plastic coverings, and covered by the false ceiling. The control station will then yield the results about the length, width, and propagation of the cracks. The installation plan of the smart film can be seen in *(Figure 15)*.

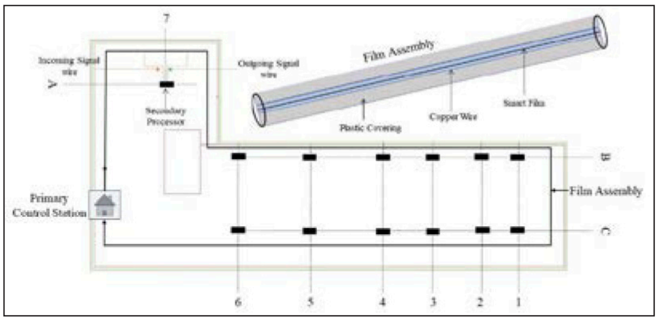


Fig. 15: Planned locations of smart crack monitoring films

7. CONCLUSIONS

The main topic of this article was to analyse the condition of a monolithic RC slab in connection to function change that increases live load. The slab in question was, on one hand, calculated using AxisVM linear FE analysis software, and, on the other hand, 3D non-linear FE analysis was also performed using ATENA 3D software. Both calculations showed sufficient load-bearing capacity also in case of increased loads. However, linear and non-linear analysis also highlighted some problems in the serviceability limit state, the maximum deflection and crack width were beyond the corresponding limit values given by EC2 standard.

Considering the above problems, two different strengthening methods (application of bonded CFRP strips and bonded steel plates as additional external reinforcement) were suggested and analysed using 3D non-linear FE analysis. Based on the analysis results, both strengthening solutions gave similar and acceptable results, as not only load-bearing capacity was increased, but maximum deflection and crack width were also decreased to a satisfactory level. Because of the linear elastic behaviour of CFRP material, the stiffness of CFRP strengthened slab was slightly higher in general, and the residual deformations were smaller in case of repeated loads. Taking the better corrosion resistance also into account, the application of CFRP strengthening provides better overall structural performance. Therefore, it was suggested as suitable strengthening solution.

Considering the expected future damages to the analysed concrete slab, and SHM system was also suggested in this study. The suggested system aims to provide real-time monitoring of the slab using three different types of sensors. The locations of these devices have been extracted from the results of linear FE analysis. To properly monitor the deflection of the whole slab, eight inclinometers are suggested to be installed. Further, monitoring of strain and temperature is planned by using the FBG optical wire that will run through the structural grids of the slab. Besides this, monitoring of existing cracks and the formation of new ones is planned to be done using the smart crack monitoring film that uses the primary and secondary processors to send and receive signals through the concrete, which helps to quantify the size and shape of cracks. Installation of the suggested sensor will give rise to the proper SHM system of the slab. That will provide the safe operation for the next 15 years and save financial resources for the owners.

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