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Seismic Analysis and Strengthening of Existing Building Due to Addition of New Floor

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Abstract

The repair and strengthening of reinforced concrete structures is a process particularly common in earthquake zones either after the occurrence of the seismic event, or before it (pre-earthquake strengthening) in order to secure the structures against subsequent excitations. Additionally, another equally important reason is the construction of one or more additional floors over the existing roof. As the existing structure is usually designed and constructed according to older regulations, it is very important to retrieve the most accurate information and data in order to realistically assess its seismic behaviour and capacity. However, in most of the cases the studies concerning these structures either do not exist anymore or were not applied properly. That is why proper knowledge of the evolution of the seismic regulations and of the structure trends and techniques over these years is required.

Key words

Seismic analysis, load-bearing capacity check, strengthening of reinforced concrete structures, concrete jackets, dowel bars, steel connectors, cost of strengthening.

1. Introduction

An earthquake is considered as an extreme load case similarly to explosion load, [1], The subject of this paper is the load-bearing capacity check of an existing structure due to the addition of a new floor and the application of strengthening with reinforced concrete jackets in order to strengthen the weak members and improve the seismic behaviour of the structure. The additional floor creates a new higher mass resulting in increased seismic shear, overturning moment and stresses from axial and bending loads. Moreover, the differences between the old and new regulations are sometimes chaotic. Differences can be also found in the materials. For the existing structure, concrete class B160 (C12/15) with compressive strength of 12.8 MPa and steel StI (S220) with yielding strength of 220 MPa have been used. As expected, the conformity test reveals weakness in most of the structural elements. That is why it is decided to strengthen the weak members. There exist several techniques and materials used for strengthening. There are variety of techniques which intend to highly increase stiffness such as construction of shear walls or the placement of lattice girder, techniques which intend to improve the ductility of members by increasing their confinement like steel strip cages and FRP panels, [2, 3]. An alternative approach is to use steel braces, [4, 5]. There are also techniques which intend to achieve both improvements such as reinforced concrete jackets. In this paper, the construction of four-sided concrete jackets is applied. The additional amount of steel reinforcement is calculated and the developed shear force is estimated. Finally the necessary steel reinforcement which is applied in the interface is calculated so that the effect of slide between the existing structure and the new layer of concrete is reduced.

2. The Existing Structure

The existing structure (see Figure 1) is a one-storey house built in the 1970s, designed and constructed according to the first seismic regulation of 1959. It is made of reinforced concrete with concrete class B160 with the average compressive strength of 12.8 MPa and the modulus of elasticity 26 GPa. The rebars are StI (S220) with the yield stress 220 MPa and modulus of elasticity 200 GPa. The existing stirrups are in all elements Ø8/200. The structure has no shear walls and there are only square columns in a sufficient symmetric arrangement.

Fig.1. The existing structure

Like the majority of the structures designed and constructed in this period, it cannot be regarded in any case as earthquake-safe structure. This is because during this period the concepts such as inelastic behaviour, ductility and capacity design, were completely unknown to engineers. These concepts began to enter the Greek literature and practice after the recent catastrophic earthquakes of 1978 and 1981. That is why certain inadequacies and problems in such structures are usually observed. The stirrups are usually poorly anchored so that the overall tensile strength cannot be achieved. Their strength is practically zero in many cases. That had as a result of very low degree of confinement and very low shear strength. In-situ preparation and transfer of concrete and of course the absence of quality control are considered as a cause of low strength concrete, as well as heterogeneity in the distribution of the quality of concrete in different parts of the element. Low strength steel and widespread use of smooth reinforcement such as S220 is not acceptable with the existing regulations. Poor anchoring of the longitudinal reinforcement results in poor transfer of the stresses from the beams to the columns. Slightly reinforced or in many cases completely unreinforced joints between beams and columns lead to significant structural damage. Carbonation of concrete and subsequent corrosion of steel reinforcement is caused by small concrete covers. In addition, there was an absence of adequate methods of analysis.

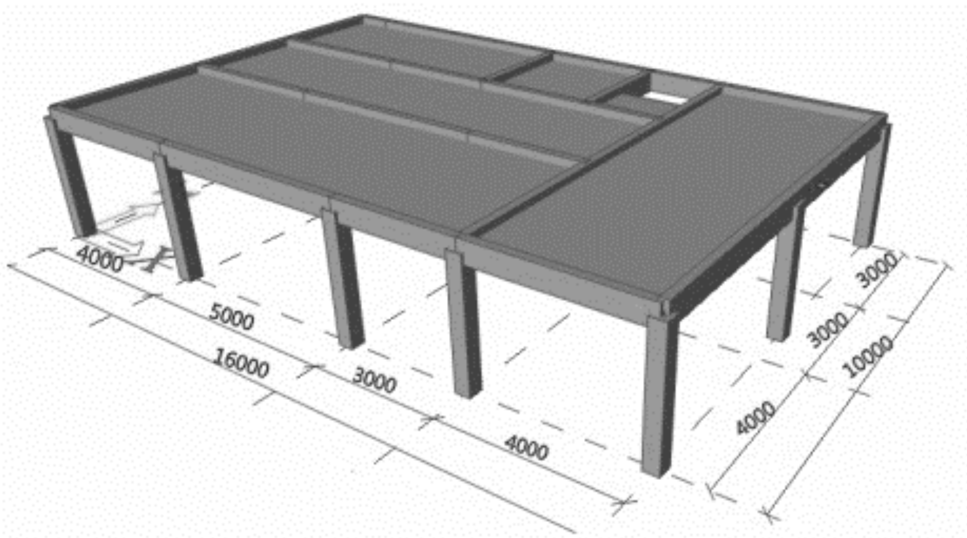

Fig.2. Model of existing structure

The existing floor (see Figure 2) is analyzed for vertical loads and seismic loading by using the modal spectrum analysis. The load combinations 1.35G + 1.5Q (where G is the permanent load and Q is the variable load) and the seismic load combinations G + 0.3Q ± $E_{x,y}$ are defined in the software, where E_x, E_y are the seismic actions according to the design spectrum of Eurocode 8. The method of the analysis, which is applied, is the linear dynamic analysis. Eurocode 8 Part 3 (for the strengthening and repair of structures) gives the ability to use the same methods as for new structures. The design ground acceleration for the seismic analysis is defined as 0.16g (zone I) and the behaviour factor as q = 2 (see Table 1). The reason for taking this value as the behaviour factor is that the regulation, with which the existing structure was designed, did not foresee the elastoplastic behaviour during seismic excitation. On the other hand, a very low value would give very high inertia forces rendering a possible strengthening economically impossible.

Table 1. Characteristics of the seismic excitation

Ground acceleration: A = 0.16g	Foundation coefficient: θ = 1
Subsoil type: B	Viscous dumping: ξ = 5%
Importance factor: γ_I = 1	Dumping correction factor: n = 1
Behaviour factor: q = 2	

The oscillating mass is calculated for load combination G + 0.3Q without taking into consideration the existing walls and the floor.

Table 1. Eigenfrequencies from Scia Engineer software

N	f [Hz]	Ω [1/s]	Ω^2 [1/s^2]	T [s]
1	5.98	37.59	1413.24	0.17
2	6.25	39.28	1543.15	0.16
3	7.57	47.54	2260.25	0.13
4	15.08	94.73	8972.91	0.07

3. The Additional Floor

The added floor is chosen to have the same geometric characteristics as the existing floor, with sections of the same dimensions, same positions of columns and beams in order to avoid non-regularity. However, concrete C20/25 (E_c = 28 GPa) and steel S500s (E_s = 210 GPa) are used. The oscillating mass is calculated for load combination G + 0.3Q. The mass of the masonry of the added floor affects the existing structure, therefore it is also taken into consideration.

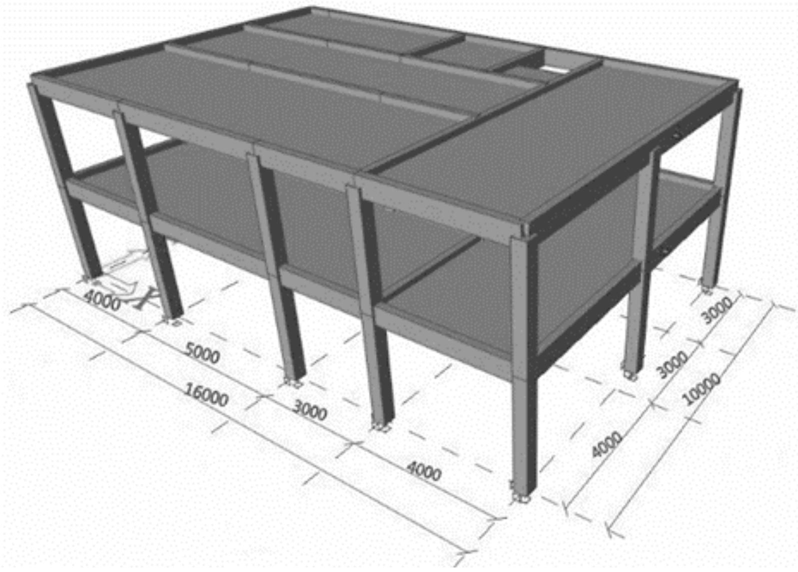

Fig.3. Model of existing structure with the additional floor

Table 2. Eigenfrequencies from Scia Engineer software

N	f [Hz]	Ω [1/s]	Ω^2 [1/s^2]	T [s]
1	3.23	20.27	410.98	0.31
2	3.48	21.87	478.28	0.29
3	4.16	26.14	683.18	0.24
4	9.99	62.75	3937.21	0.10

4. The Load-Bearing Capacity Check

Bending resistance, shear resistance and rotation capacity are checked for each member of the existing structure. The amount of the existing reinforcement is taken into consideration for the purpose of calculation of the bending resistance which is compared with the developed internal forces and structural demands.

The Bending Resistance Check

2Ø14 => $A_{sl} = 308$ mm^2,

$\omega = A_{sl} \cdot f_{yd} / (b_w \cdot d \cdot f_{cd}) = 0.0818,$ (1)

$\mu = 0.08$,

where ω in (1) is the mechanical volumetric ratio of reinforcement. The marginal value of moment μ can be calculated by means of analytical tables from CEB [6].

Minimum Reinforcement

The calculation of the minimum geometrical ratio of the tensile reinforcement (2) is based on Eurocode 2 [7] for the design of members with high demands of ductility.

$\rho_{lmin} = 0.5 \cdot f_{ctm} / f_{yd} = 0.003$ ($f_{ctm} = 1.6$ MPa), (2)

$A_{smin} = \rho_{lmin} \cdot b_w \cdot d = 327$ mm^2.

For determining whether the existing reinforcement is adequate or not, the bending resistance (3) is compared with the developed bending moment:

$M_{Rd} = \mu \cdot b_w \cdot d^2 \cdot f_{cd} = 25.920$ kNm $> M_{Sd} = 20.34$ kNm. (3)

The existing reinforcement is adequate.

The Shear Resistance Check

The action without seismic loading (1.35G + 1.5Q) and the action with seismic loading (G + 0.3Q ± $E_{x,y}$) are taken into consideration for the shear resistance check. There is also discrimination between critical and non-critical area.

Demand of Stirrups

The capacity check showed that there is no significant increase of shear due to the developed seismic bending moments that is why the sections need stirrups and not diagonal reinforcement. The maximum shear is for the combination 1.35G + 1.5Q.

Within the Critical Area

The equations (4 and 8) express the ability of concrete to undertake shear stresses through its compression zone [7] at the shoulder of the beam (V_{Sd}^{a}).

$V_{Sd}^{a} = 48$ kN,

$V_{Rd1} = [\tau_{Rd} \cdot k \cdot (1.20 + 40\,\rho_l)] \cdot b_w \cdot d = 23.045$ kN. (4)

The unit of τ_{Rd} is MPa, its value is retrieved from Eurocode 2 [7] and it is dependent on the characteristic strength of concrete. The necessary coefficients are given by

$k = 1.60 - d = 1.15$ m ($k \geq 1.00$),

$\rho_l = A_{sl} / (b_w \cdot d) = 0.0059$ ($\rho_l \leq 0.02$) .

The ability of concrete is reduced by 70% (5) in the critical area because we expect that plastic hinges will be developed and the height of the compression zone will be reduced respectively, thus the ability of concrete to undertake shear stresses will be lower. The equation 7 determines the distance between stirrups in order to cover the demand

$V_{cd} = 0.3 \cdot V_{Rd1} = 6.91$ kN, (5)

$V_{wd} \geq V_{Sd}^{a} - V_{cd} = 41.09$ kN, (6)

$(A_s / s) \cdot 0.9d \cdot f_{ywd} \cdot (1 + \cot\alpha) \cdot \sin\alpha \geq 41.09$ kN, (7) where s is the distance between stirrups, f_{ywd} is the characteristic strength of the shear reinforcement and α is the angle between stirrups and the axis of the beam. The value of the maximum spacing of the stirrups s_{max} is 188 mm which means that the existing amount of stirrups (Ø8/200) is not adequate within the critical area.

Outside the Critical Area

The value of shear (V_{Sd}^{c}) at the edge of the critical area of the beam (2h) is used for the check. As the check takes place out of the critical area, the whole shear resistance of the mechanism of concrete is taken into consideration (9). The demand (10) is very small, thus the existing amount of stirrups (Ø8/200) is adequate outside the critical area

V_{Ed}^{c} = 27 kN,

$$V_{Rd1} = [\tau_{Rd} \cdot k \cdot (1.20 + 40\,\rho_l)] \cdot b_w \cdot d = 23.045 \text{ kN}, \tag{8}$$

$$V_{cd} = V_{Rd1} = 23.045 \text{ kN}, \tag{9}$$

$$V_{wd} \geq V_{Sd}^{c} - V_{cd} = 4 \text{ kN}. \tag{10}$$

Eurocode 8 Part 3 requires apart from the control in terms of strength, the control of the ability of members in terms of deformation. The ability can be expressed either through the moment-curvature curve (M-φ) or by means of the moment-rotation curve (M-θ) (see Figure 4). Interesting information for each member regarding its plastic rotation capacity and residual bending resistance are retrieved so that it can be decided whether the failure will be brittle or ductile.

Yield Curvature

The equations for the calculation of the parameters A (11 and 15) and B (12 and 16) are empirical formulas described in the Greek regulation for repair and strengthening of structures [8]. They are based on the values of the geometrical ratio of steel reinforcement ρ and the value of axial force. The usage rests in calculation of the height of the compression zone at the moment of yielding (13 and 17). Curvature at the moment of yielding is calculated by means of equations (14 and 18). Purpose of the calculation is to check whether the yielding will occur due to the excess of the deformation capacity of concrete or due to the excess of the deformation capacity of steel.

Yielding of Steel

$$A = \rho + \rho' + \rho_v + \frac{\mathrm{N}}{b \cdot d \cdot f_y} = 0.0066, \tag{11}$$

$$B = \rho + \rho' \cdot \delta' + 0.5\rho_v + \frac{\mathrm{N}}{b \cdot d \cdot f_y} = 0.0036, \tag{12}$$

$$\xi_y = \left(a^2 A^2 + 2aB^2\right)^{0,5} - aA = 0.188, \tag{13}$$

$$\varphi_y = \frac{f_y}{E_s\left(1-\xi_y\right)d} = 0.0029. \tag{14}$$

Yielding of Concrete

$$A = \rho + \rho' + \rho_v - \frac{\mathrm{N}}{1.8a \cdot b \cdot d \cdot f_y} = 0.0066, \tag{15}$$

$$B = \rho + \rho' \cdot \delta' + 0.5\rho_v \cdot \left(1+\delta'\right) = 0.0036, \tag{16}$$

$$\xi_y = \left(a^2 A^2 + 2aB^2\right)^{0,5} - aA = 0.188, \tag{17}$$

$$\varphi_y = \frac{\varepsilon_c}{\xi_y \cdot d} = 0.0101. \tag{18}$$

From the comparison of the curvatures of steel (14) and concrete (18), we can see that the yielding occurs because of yielding of steel as it has the minimum value.

Bending Moment when the Yielding Occurs

$$\frac{M_y}{b \cdot d^3} = \varphi_y \left\{ \begin{aligned} & E_c \frac{\xi_y^{\,2}}{2} \cdot \left(0.5 \cdot (1+\delta') - \frac{\xi_y}{3}\right) + \\ & + \left[(1-\xi_y)\rho + (\xi_y - \delta')\rho' + \frac{\rho_v}{6}(1-\delta')\right] \cdot (1-\delta')\frac{E_s}{2} \end{aligned} \right\} \Rightarrow M_y = 29.476 \text{ kNm} \tag{19}$$

The equation (19) is used for calculation of the value of bending moment when the yielding occurs. Like all the following equations (20 – 23), it is described both in the Greek code [8] and in the Part 3 of Eurocode 8 about the repair and strengthening of structures [9].

Cyclic Shear Resistance

$$\begin{aligned} V_R = {} & \frac{h-x}{2L_s} min\left(N; 0.55A_c \cdot f_c\right) + \left(1 - 0.05\, min(5; \mu_\Delta{}^{pl})\right) \cdot \\ & \cdot \left[0.16\, max(0.5; 100\rho_{tot}) \cdot (1 - 0.16\, min(5; a_s)) \cdot \sqrt{f_c} \cdot A_c + V_w\right] = 12.054 \text{ kN} \end{aligned} \tag{20}$$

where $\mu_\Delta{}^{pl}$ is a factor which takes into consideration the effect of the cyclic loading.

Value of Shear when the Yielding Occurs

$$V_{Mu} = \frac{M_y}{L_s} = 14.887 \text{ kN} \geq V_R \Rightarrow \alpha_v = 1, \tag{21}$$

where L_s is the shear length and α_v is a factor which takes into consideration the effect of shear and slide. If the value of shear (21) is larger than the cyclic shear resistance (20) it is equal to 1 and it increases the value of the chord rotation at yield (22), otherwise it is equal to 0.

Chord Rotation at Yield

$$\theta_y = \varphi_y \frac{L_s + \alpha_v \cdot Z}{3} + 0.0014 \cdot \left(1 + 1.5\frac{h}{L_s}\right) + \frac{d_b \cdot f_y \cdot \varphi_y}{8\sqrt{f_c}} = 0.0045. \tag{22}$$

Total Chord Rotation Capacity

$$\theta_u = 0.016(0.3)^{\nu} \cdot \left[\frac{max(0.01;\omega')}{max(0.01;\omega)} \cdot f_c\right] \cdot 0.225(a_s)^{0.35} \cdot 25^{\left(a \cdot \rho_s \frac{f_{yw}}{f_c}\right)} \cdot \left(1.25^{(100 \cdot \rho_d)}\right) = 0.04367 \tag{23}$$

where

$a_s = \dfrac{L_s}{h}$ is the shear ratio,

$\nu = \dfrac{\mathrm{N}}{b \cdot h \cdot f_c}$ is the normalized axial load.

The coefficient α_s is the confinement effectiveness factor and its calculation is described in Eurocode 8, Part 3 [9]. In the occasion that stirrups do not close with 135^{o} hoops, α_s is taken equal to 0.

Index of Ductility

$\mu_\theta = \dfrac{\theta_u}{\theta_y} = 9.62 > 2 \Rightarrow$ The failure is regarded ductile

$\theta_{pl} = \theta_u - \theta_y = 0.039,$

$\theta_{res} = 3 \cdot \theta_{pl} = 0.117,$

$M_y = 29.476$ kNm,

$M_{res} = 0.25 \cdot M_y = 7.369$ kNm,

where M_{res} is the residual capacity of the element and its value is estimated as the 25% of the capacity at the yielding point. There will be the final failure under the action of the gravity loads after its excess. The residual rotation capacity θ_{res} for beams is three times higher than the plastic rotation capacity θ_{pl}. It is less in case of columns, the residual rotation capacity θ_{res} is approximately two times higher than the plastic rotation capacity θ_{pl}.

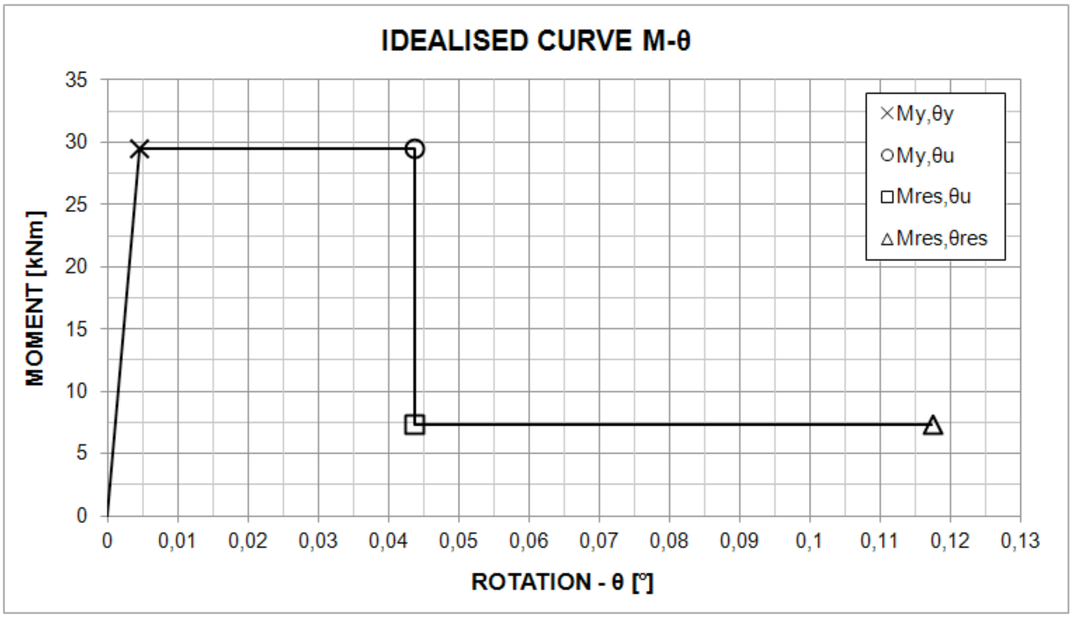

Fig.4. Idealised curve M-θ

5. Strengthening of Columns with Reinforced Concrete Jackets

For the internal forces caused by the additional floor, increased by dividing them by the monolithic coefficient k_r = 0.9, the examined columns suffered from inadequacy at their bending resistance and shear resistance. That is why columns are strengthened by a quadratic concrete jacket. The materials for the construction of the concrete jacket are: concrete C20/25 and steel S500s. The thickness of each additional layer will be t = 100 mm and the surface will be protected against slide with the use of dowel bars and flexible connectors which efficiency was proved, [10].

The existing reinforcement (4Ø20) is taken into consideration. Then, the additional demand is A_s = 40.95 cm^2 – 12.55 cm^2 = 28.40 cm^2. The rebars contained in the concrete jacket are 8Ø22 with A_s = 30.40 cm^2. The axial load which acts in the column is N_{Sd} = -327 kN.

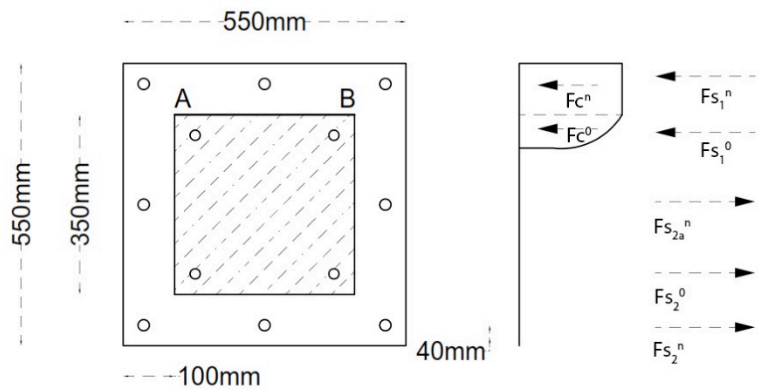

Fig.5. Shear force at the interface

The assumption in order to calculate the shear load acting at the interface (Figure 5) is that the whole compressive force is taken by the concrete jacket ($F_{cm} = F_c^n + F_{s1}^n$) and the tensile reinforcement yields.

From the equilibrium of the internal forces, the following equation is obtained

$$F_{s2}^{\ n} + F_{s2}^{\ 0} + F_{s2a}^{\ n} = F_c^{\ 0} + F_c^{\ n} + F_{s1}^{\ 0} + F_{s1}^{\ n}. \tag{24}$$

From the hypothesis that the compressive force is taken by the concrete jacket the equation (24) becomes

$$F_{s2}^{\ n} + F_{s2}^{\ 0} + F_{s2a}^{\ n} = F_c^{\ 0} + F_{s1}^{\ 0} = F_{cm} = 483.616 \text{ kN}$$

The compressive force of the concrete jacket acts in the interface and creates the shear load along the height U_o = 1.8 m, while the interface AB is located in the compression zone. For this length U_o the necessary dowel bars (see Figure 6 and Table 4) is calculated for resisting the shear load and preventing the slip between the existing layer of concrete and the new layer of concrete. In addition, the beneficial action of cohesion and friction are not taken into consideration for safety reasons.

Table 4. Characteristics of the dowel bars

DOWEL BARS		
d_b	Dowel section Ø (mm)	14
l_b	anchoring length (mm)	140
t	Thickness of existing concrete (mm)	350
s_{cr}	critical length (mm)	200
s_{eff}	Length between dowels (mm)	200
c_p	Concrete's cover in the direction of shear (mm)	100
c_{max}	Maximum concrete's cover in the perpendicular direction of shear (mm)	175
c_{min}	Minimum concrete's cover in the perpendicular direction of shear (mm)	175

The maximum distance between dowels is given by the condition: $s_{max} \leq \min(6h_{min}, 800 \text{ mm})$, therefore s_{max} = 600 mm. The critical length to avoid overlapping of concrete cones is $s_{cr} = \gamma_{Rd} \cdot (l_b + d_b)$ = 200 mm. The resistance of a dowel bar is the minimum value of the resistance against the three modes of failure shown in Figure 7. The equations (25 – 27) introduced below are described in the regulation of the technical chamber of Greece [8] about the strengthening of structures against seismic loading. The first mode of failure, equation (25), occurs in the interface due to the exceeding of the characteristic strength of steel. The second mode of failure, equation (26), appears with a conical secession in the direction of shear with the development of a plastic hinge at the dowel bar. The third mode of failure, equation (27), appears when there is low quality of concrete and the dowel bar is installed very close to the edges without keeping the necessary distances.

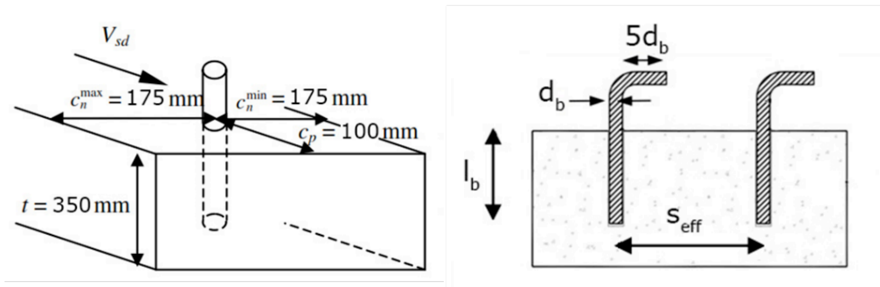

Fig.6. Detailed sketch of dowel bars

The first mode of failure:

$$V_{ud}^{1} = \frac{A_s \cdot f_{yd}}{\sqrt{3}} = 38.62 \text{ kN}. \tag{25}$$

The second mode of failure:

$$V_{ud}^{2} = \gamma_m \cdot d_b^{2} \cdot \sqrt{f_{cd} \cdot f_{yd}} = 11.52 \text{ kN}. \tag{26}$$

Where $\gamma_m = 1$ is coefficient for cyclic loading.

The third mode of failure:

$$V_{ud}^{3} = k_1 \cdot \alpha_1 \cdot \alpha_2 \cdot \sqrt{f_{cd} \cdot d_b} \cdot \left(\frac{l_b}{d_b}\right)^{\frac{1}{5}} \cdot c_p^{\frac{3}{2}} = 12.29 \text{ kN}, \tag{27}$$

where

$$k_1 = 0.28 \frac{\sqrt{\text{N}}}{\text{mm}},$$

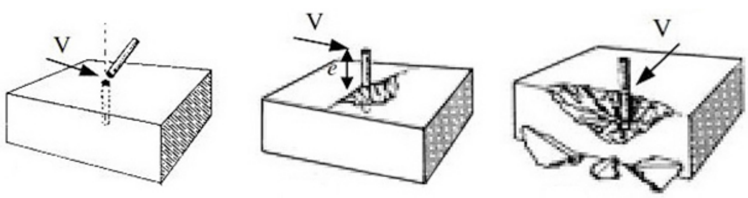

Fig.7. Three modes of failure

$$\alpha_1 = \left(\frac{t}{1.4 \cdot c_p}\right)^{\frac{2}{3}} \le 1.0,$$

$$\alpha_2 = \max\left[0.3 + 0.7 \cdot \frac{c_n^{\min}}{1.5 \cdot c_p} \cdot \frac{\left(c_n^{\max} + c_n^{\min}\right)}{3.5 \cdot c_p}\right] \le 1.0.$$

The coefficients k_1, α_1, α_2 take into consideration the position of the dowel bars and the thickness of the existing concrete (see Figure 6 and Table 4).

From the third failure modes, the resistance takes the value of the second failure mode *(*V_{ud} = 11.52 kN*)* which is the most preferable as it is characterized by elastoplastic behavior. Then, the number of required dowel bars is calculated by dividing the shear load with the resistance of the single dowel as

$$n_D = \frac{F_{cm}}{V_{ud}} \Rightarrow 42 \text{ dowel bars.} \qquad (28)$$

In order to avoid the significant decrease of the resistance of the dowel bars, because of the creation of secession cones in concrete

$$s_{eff} = \frac{U_o}{41} \Rightarrow s_{eff} = 43 \text{ mm} << s_{cr}, \qquad (29)$$

it is advisable to put the dowel bars with 200 mm spacing which is the critical value. Then, the total number of dowel bars is calculated respectively to the critical length as

$$n_D = \frac{U_o}{s_{cr}} \Rightarrow 10 \text{ dowel bars.} \qquad (30)$$

The total resistance of the dowels

$$V_{ud}^{tot} = n_D \cdot V_{ud} = 115.2 \text{ kN} \qquad (31)$$

is not adequate for resisting the developed shear load. Therefore, flexible connectors (see Figure 8) are installed for the rest of the demand given by

$$\Delta V = F_{cm} - V_{ud}^{tot} = 368.416 \text{ kN}. \qquad (32)$$

The characteristics of the flexible connectors are: diameter d_b = 18 mm, steel grade S500s and their height h_s = 52 mm. The resistance of the flexible connector is calculated by taking into consideration the quality of steel of the three connected elements. Then, despite the fact that the resistance of the connector is 98 kN we use the value 60.07 kN which is calculated with the design value of the weakest steel, which is shown below

$$T_a = \frac{2 \cdot 10 \cdot A_s}{h_s} = 98 \text{ kN} > F_s = A_s \cdot f_{yd} = 60.07 \text{ kN}. \qquad (33)$$

By dividing the additional demand with the resistance of the single shear connector, the total number of 6 required flexible connectors are obtained, as shown below

$$n_D = \frac{\Delta V}{F_s} \Rightarrow 6 \text{ flexible connectors.} \qquad (34)$$

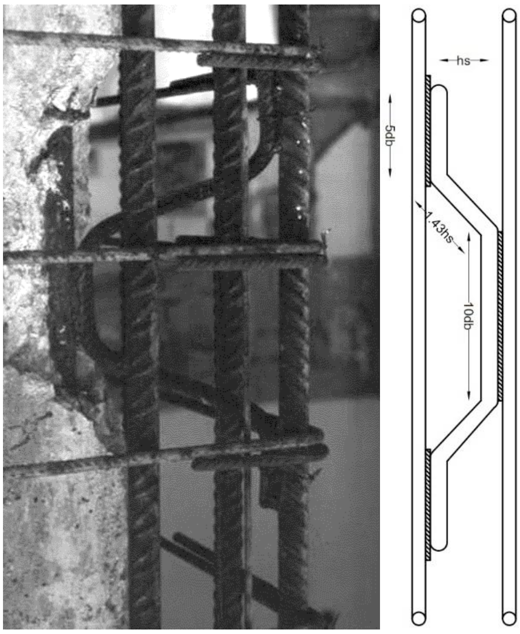

Fig.8. Flexible connector

6. Cost of Strengthening

The cost of application of concrete jackets (see Figure 9) includes a large range of tasks (see Table 5) which require specialized crews. The analysis of the particular tasks in Table 5 reveals that the overall cost increases significantly due to the use of the shotcrete pump in the case when the gunite jacket is applied.

Table 5. Cost of main operations for strengthening with reinforced concrete jackets

Removal of Plain Concrete	m^3	27.00 €
Removal of Reinforced Concrete	m^3	44.00 €
Removal of Plaster Work	m^2	4.50 €
Removal of Floors	m^2	10.00 €
Concrete Jacket with Cast Concrete	m^3	250.00 €
Concrete Jacket with Shotcrete	m^3	300.00 €

In total, the cost of strengthening of all the columns with reinforced concrete jackets for this certain project will be about 10,000 €. The price includes all the already mentioned processes. On the other hand, the cost of the additional floor is about 200,000 €, (1,200 €/m^2), which is the average price for the Greek market. This price includes the whole range of works from the necessary acquirement of the licence till the last application for the completion of the project.

Fig.9. Construction of reinforced concrete jackets

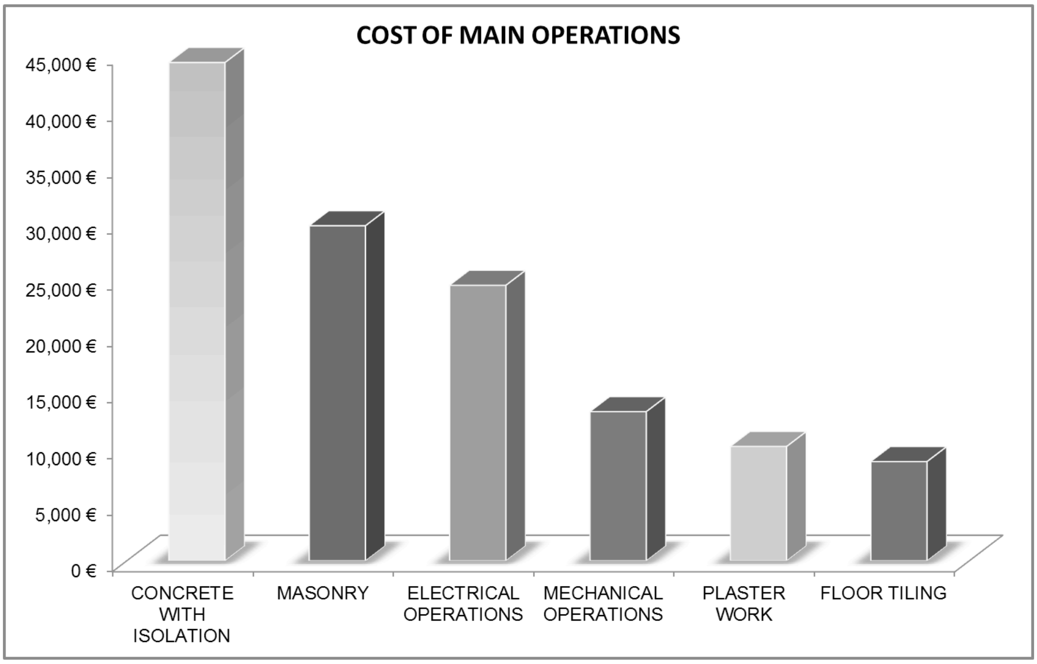

Fig.10. Cost of main operations of additional floor

Figure 10 illustrates the cost of the main operations for the construction of the additional floor. Their value constitutes 66% of the total cost. It is interesting to note the comparison between the cost of concrete of the additional floor which is about the 25% of the total cost and the cost of strengthening. We will notice that the cost of strengthening is less than a quarter compared to the cost of concrete of the additional floor and just 5% of the total cost.

Discussion

It should be mentioned that the origin of the paper is connected with certain social aspects. Difficult situation of contemporary society forces a lot of young people to return homes which can be located in seismic zones. The reason is that the big cities do not provide enough opportunities for employment these days. New additional floors are built in order to increase the capacities of native homes. In spite of the fact that the paper is focused on the structures located in Greece, it is possible to apply the principles of provided technique of seismic retrofitting by means of concrete jacketing in different places around the world.

The basic idea results from the requirement for increasing of both ductility and stiffness. The use of the dowel bars and the flexible connectors plays a crucial role in providing of a proper shear connection between the additional concrete layer and existing column. The cost of strengthening of reinforced concrete columns by means of concrete jacketing is approximately 5% of the total cost. Such a fraction of the total cost is worth being invested instead of run a risk of destruction caused by the earthquake.

Conclusions

The major conclusion of this paper is that the existing structures, which were designed and constructed according to the first seismic regulation in Greece, suffer from many deficiencies when compared to the newer structures which were carried out according to the more recent regulations. The existing structures do not meet the load-bearing capacity design criteria for stronger columns and weaker beams. Another of their weaknesses is the used material, specifically the steel grade S220, which is not acceptable by the new regulations for the design of members with high demands on ductility.

Despite the fact that the initial design of the existing structures foresaw a future addition of another floor, the actual load-bearing capacity check shows inadequacy in shear and bending in the majority of the elements. That is logical as the seismic coefficient, according to which the structures were designed, has been increased by 400%. In general, it can be stated that the addition of the new floor creates new higher masses which result in increased seismic shear, overturning moment, axial and bending stresses. Regarding the capacity check, it is very important to retrieve the most accurate information and data concerning the design and construction of the existing structure in order to realistically assess its seismic behavior and load-bearing capacity.

Finally, the detailed example of calculation was introduced which considered strengthening with reinforced concrete jackets, which is widely used for increasing the strength and ductility. Special attention was paid to connection between the existing column and the concrete jacket. The example was concluded with clear evidence of the real cost of the strengthening in comparison to the entire cost of the added floor.

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References

1. M.Bruncvík, P.Štemberk, “Explosion-resistant reinforced-concrete construction kit for shelter in extreme situations”, Modelling, Measurement and Control, B Mechanics and Thermics, Volume 81, Issue 1, 2012, pp. 34-49.
2. C.Yalcin, O.Kaya, M.Sinangil, “Seismic retrofitting of R/C columns having plain rebars using CFRP sheets for improved strength and ductility”, Construction and Building Materials, Volume 22, Issue 3, 2008, pp. 295-307.
3. L.P.Ye, K.Zhang, S.H.Zhao, P.Feng, “Experimental study on seismic strengthening of RC columns with wrapped CFRP sheets”, Construction and Building Materials, Volume 17, Issues 6–7, 2003, pp. 499-506.
4. L.D.Sarno, G.Manfredi, “Seismic retrofitting with buckling restrained braces: Application to an existing non-ductile RC framed building”, Soil Dynamics and Earthquake Engineering, Volume 30, Issue 11, 2010, pp. 1279-1297.
5. C.Durucan, M.Dicleli, “Analytical study on seismic retrofitting of reinforced concrete buildings using steel braces with shear link”, Engineering Structures, Volume 32, Issue 10, 2010, pp. 2995-3010.
6. CEB, *Ductility of Reinforced Concrete Structures*, 1998.
7. EN 1992-1-1: 2004, Eurocode 2, *Design of Concrete Structures*, – Part 1-1: General rules and rules for buildings
8. OASP, *Regulation for Repair and Strengthening of Structures*, Athens, 2012.

9. EN 1998-1:2004, Eurocode 8, *Design of Structures for Earthquake Resistance*, – Part 3: Assessment and retrofitting of buildings
10. K.G.Vandoros, S.E.Dritsos, “Concrete jacket construction detail effectiveness when strengthening RC columns”, Construction and Building Materials, Volume 22, Issue 3, 2008, pp. 264-276.
11. L. J. Humar, *Dynamics of Structures*, Taylor & Francis Group plc, London, 2005.
12. E. L. Wilson, *Three-Dimensional Static and Dynamic Analysis of Structures*, Computers and Structures, Inc., Berkeley, 2002.
13. Greek technical chamber, *Strengthening of structures for seismic loading*, Athens, 2004.
14. G.G.Penelis, A.J.Kappos, *Earthquake-resistant Concrete Structures*, Taylor & Francis Group plc, London, 1997.
15. ATC-40, *Seismic Evaluation and Retrofit of Concrete Buildings*, Volume 1, Applied Technology Council, California, 1996.
16. EN 1998-1:2004, Eurocode 8, *Design of Structures for Earthquake Resistance*, – Part 1: General rules, seismic actions and rules for buildings