Gazi Mühendislik Bilimleri Dergisi
2021, 7(3): 286-297
Araştırma Makalesi/Research Article
https://dergipark.org.tr/gmbd

Yeni Betonarme Katmanla Güçlendirilen Bir Betonarme Yapının Nümerik Olarak İncelenmesi

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MAKALE BİLGİSİ

Alınma: 30.07.2021
Kabul: 07.10.2021

Anahtar Kelimeler:
Güçlendirme,
Betonarme yapı,
Nümerik inceleme

ÖZ

Bu çalışmada, öncelikle literatürde yer alan güçlendirme yöntemleri genel olarak incelenmiş ve servis ömrü boyunca kolon ve kirişlerinde sismik kaynaklı yapısal hasarlar oluşmuş bir betonarme binanın hasar tespiti yapılmıştır. Betonarme yapının tüm kolonlarına ve hasarlı kirişlerine yeni bir betonarme tabaka eklenerek ve buna ilave olarak yapıya ek betonarme perdeler yerleştirilerek uygulanan güçlendirme işlemi nümerik olarak değerlendirilmiştir. Hasar tespiti aşamasında, betonarme kolonların neredeyse tamamının ve kesme-eğilme çatlaklarının bulunduğu betonarme kirişlerin taşıma kapasitelerinin Türkiye Bina Deprem Yönetmeliği-2018 (TBDY-2018)'e göre yetersiz olduğu tespit edilmiştir. Uygulanan güçlendirme işleminden sonra, kolonların etkileşim diyagramları ve kirişlerin moment-eğrilik ilişkileri güçlendirme öncesi durum ile karşılaştırılmıştır. Karşılaştırma sonrasında kolonların eksenel yük taşıma kapasitesinde %300, moment taşıma kapasitesinde %666, kirişlerin moment taşıma kapasitesinde ise %485 artış olduğu tespit edilmiştir. Bunların yanı sıra, güçlendirme işleminden sonra yapının ilk üç moduna ait periyotlarda oluşan düşüşe bağlı olarak yapıya etki eden yatay ve düşey deprem ivmelerinin değerleri sırasıyla %294 ve %400 oranında artış göstermiştir. Nümerik çalışmalar SAP2000 sonlu elemanlar programı ile gerçekleştirilmiştir. Nümerik çalışma sırasında, betonarme kolon ve kirişlere yeni bir betonarme tabaka eklenmesi literatürde kullanımı oldukça kısıtlı olan SAP2000-Section Designer kullanılarak gerçekleştirilmiştir. Elde edilen sonuçlara göre hasarlı betonarme bir binada bulunan kolon ve kirişlere yeni bir betonarme tabaka eklenerek ve yapıya betonarme perdeler yerleştirilerek yapının sismik davranışının iyileştirileceği nümerik olarak ortaya koyulmuştur.

https://dx.doi.org/10.30855/gmbd.2021.03.11

Numerical Investigation of a Reinforced Concrete Structure Strengthened by New Reinforced Concrete Layer

ARTICLE INFO

Received: 30.07.2021
Accepted: 07.10.2021

Keywords:
Strengthening,
Reinforced concrete structures,
Numerical investigation

ABSTRACT

In this study, strengthening methods of the structures mentioned in the literature were examined in general sense and an existing reinforced concrete building that has been damaged from its columns and beams in previous earthquakes during service life was investigated numerically by determining the damage status. Then, strengthening was applied by adding a new reinforced concrete layer to all columns and damaged beams followed by placing additional reinforced concrete shear walls to the structural layout. Finally, finite element analysis of the rehabilitated building was carried out and the numerical results were compared with the initial state. At the stage of damage assessment, it has been determined that the load-bearing capacities of almost all reinforced concrete beams with flexural-shear cracks and reinforced concrete columns were insufficient according to Turkey Building Earthquake Code-2018 (TEC-2018). Once the strengthening was applied, the new interaction diagrams of the columns and the moment-curvature

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To cite this article: Ö. Mercimek, "Numerical Investigation of a Reinforced Concrete Structure Strengthened by New Reinforced Concrete Layer", *Gazi Journal of Engineering Sciences,* vol.7, no.3, pp. 286-297, 2021. doi:10.30855/gmbd.2021.03.11

relations of the beams were compared with the un-strengthened state. It was determined that there was an increase of 300% in the axial load-carrying capacity and 666% in the moment-carrying capacity of the columns, and 485% in the moment carrying capacity of the beams. In addition, the values of horizontal and vertical earthquake accelerations affecting the structure increased by 294% and 400%, respectively, due to the decrease in the periods of the first three modes of the building. Numerical studies were carried out with SAP2000 finite element program. In the numerical study, adding a new reinforced concrete layer to the reinforced concrete columns and beams was implemented using the SAP2000-Section Designer which has quite limited usage in the literature. According to the results obtained, it has been numerically revealed that the seismic performance of the damaged structure can be improved by adding new reinforced concrete layers to the existing columns and beams and placing reinforced concrete shear walls to the building.

https://dx.doi.org/10.30855/gmbd.2021.03.11

1. INTRODUCTION *(GİRİŞ)*

The main purpose of strengthening methods is to ensure sustainability and cost efficiency. In this context, researchers have tended to use structural materials with advanced technological properties over time. Firstly, reinforced concrete structural elements were jacketed with an additional cross-section created by using concrete and steel reinforcement with higher strength than the material strength in the existing structure, thereby, the load-bearing capacity of the structure was tried to be increased [1-6]. Although desired results were obtained in terms of bearing capacity, the challenges encountered in the process of jacketing (difficulties faced with in formwork, reinforcement and concrete application, inability to use the structure while strengthening, deterioration of the architecture of the structure, etc.) diminished the interest on this technique. Instead, strengthening the structural components with steel members against shear and bending forces has come to the forefront [7-11]. In this strengthening technique, however, risks caused by lack of qualified workers, adverse environmental effects such as fire or corrosion, or cost and architectural concerns were encountered. For the last 20 years, fiber reinforced polymer (FRP) strengthening method has been widely used and many structures have been strengthened with FRP instead of these strengthening techniques previously mentioned. Besides, many scientific studies have been carried out on this strengthening technique and FRP has become one of the most preferred materials in the studies on repair and strengthening of damaged reinforced concrete structures [12-26].

In most of the studies, an organic epoxy material was used to form a cohesive zone between the FRP and the surface on which FRP was applied. However, a number of deficiencies that need to be addressed in this strengthening technique such as (a) poor resistance against fire [27], (b) non-applicability on wet or damp surfaces [28], (c) lack of ability to replace the coating layer [29] (d) classification as a hazardous material during disposal, (e) high cost of FRP techniques [29], (f) poor vapor permeability and the use of organic resins that damage the concrete [27]. Considering these disadvantages, the spread of the textile reinforced mortar layer (TRM) with scientific research has been accepted as a remarkable progress in the field of structural strengthening [28, 29] . TRM, which is also known as TRC, FRCM in English and TGSK in Turkish, is a composite building material formed by a cement-based inorganic mortar together with textiles obtained from different materials (such as steel, carbon, basalt, glass). Textiles used as reinforcement of composite material typically consist of perpendicular (bidirectional) fibers. Considering that the mortar is produced and applied by conventional methods, TRM has many advantages over fiber-reinforced polymers (a widely used, epoxy-based composite material). The main advantages of TRM strengthening method can be listed as (a) low cost, (b) resistance to high temperatures, (c) applicable to concrete, reinforced concrete and masonry construction surfaces, (d) applicable to wet surfaces, (e) low heat permeability, and (f) high load-bearing capacity.

Despite the fact that the jacketing method is one of the oldest strengthening methods in the literature and has some disadvantages, it is still a preferred due to its remarkable affect on the load-bearing capacity of the structural members, the availability of the qualified workers and the ease of material supply, as well [30, 31]. Researchers have used advanced, technological and innovative programs and tools in the design and analysis of engineering structures [32-39] When the literature is examined, no study has been carried out on the structures by adding a new reinforced concrete layer to the load-bearing members of the reinforced concrete structures with the SAP2000 finite element program. Within the scope of this study, the analysis of a reinforced concrete building whose surveying was done, crack distribution in the load-bearing members was detected and concrete properties were determined with the samples taken, was performed with the SAP2000 finite element program. Considering the numerical results, a new reinforced concrete layer was added to the existing columns and beams, and reinforced concrete walls were placed on the load-bearing system which has no reinforced concrete walls in order to improve the behavior of the structure.

2. CASE STUDY *(VAKA ÇALIŞMASI)*

The storey formwork plan of the building (Table 1) is given in Figure 1, and the vertical section A-A in this plan is shown in Figure 2. In the storey formwork plan, the axis spacing was measured as 4.30 m. The information and patterns of the cracks in the building are presented in Table 2. It has been determined that the cracks in the storeys were capillary sized. However, the cracks in the columns and beams were measured to be in the range of 1-2 mm. Flexural and shear cracks were observed in some of the beams while shear cracks were observed in the joints in the columns. The dimensions of the columns, beams and storeys in the building and the details of the reinforcement steel are given in Table 3.

Table 1. Details of RC structure
(Betonarme yapının detayları)

Concrete Strength	15.1 N/mm^2
Modulus of Elasticity of Concrete	$E_{theoretical} / 2 = 13500$ N/mm^2
Reinforcement Steel Grade	S220
Structural Behavior Factor	4
Seismic Design Category	Given in Figure 4
Soil Classification	ZB
Building Type	School
Building Use Category	1
Foundation Type	Raft Foundation
Structural System	Moment Resisting Frame
Infill Walls	Clay Brick
Storey Covering	Parquet Strip (beechen)

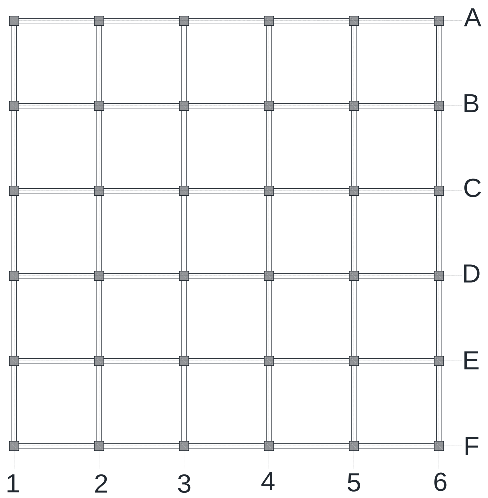


Figure 1. Formwork plan *(Kat kalıp planı)*

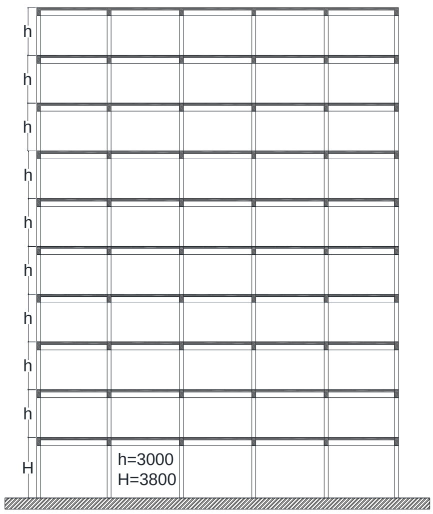


Figure 2. Reinforced concrete structural system (mm)
(Betonarme yapısal sistem (mm))

3. NUMERICAL INVESTIGATION *(NÜMERİK İNCELEME)*

3.1. Modeling of Existing Structure *(Mevcut Yapının Modellenmesi)*

The model of the building was created as shown in Figure 3 as close to actual by using the required data. All rigidities in storeys with cracks have been reduced to 30%. It is assumed that the beams carry neither moment in the flexural regions nor shear force in the region of shear cracks, while in the columns, shear cracks were created supposing that no force was carried in order to stay on the safe side.

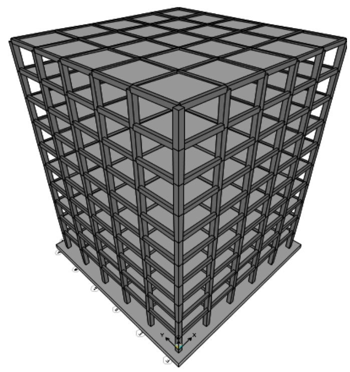


Figure 3. Structural model *(Yapısal model)*

The required earthquake data obtained for the location of the building are shown in Figure 4. The foundation of the building was formed in the ZB soil classification. The configuration of earthquake loads was carried out in the light of these information. For beams, 6 kN/m dead load was assigned. For storeys, dead load and live load, according to TS498, were assigned as 3.5 kN/m^2 and 5.0 kN/m^2, respectively.

Table 2. Crack pattern *(Çatlak durumu)*

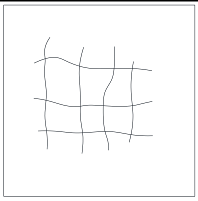
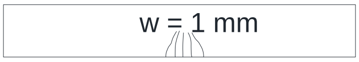
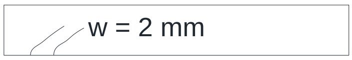
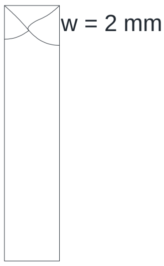
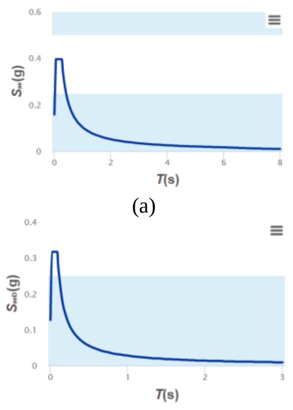
Structural members	Storey	Axes	Crack pattern
Slab	1	C-D 4-5	
	2	B-C 1-2	
	3	B-D 4-5	
Beam	1	B 1-2	
	1	B 5-6	
	1	D 5-6	
	2	D 5-6	
	3	B 5-6	
	4	B 1-2	
	4	B 5-6	
	6	D 5-6	
Beam	2	2 C-D	
	2	3 B-C	
	2	3 E-F	
	4	C 2-3	
	4	2 B-D	
	6	2 B-D	
	6	4 C-D	
	6	D 3-4	
Columns	1	1-B	
	1	2-B	
	1	5-D	
	1	6-D	
	2	5-D	
	2	6-D	
	3	5-D	
	3	6-D	
	1	6-F	
	2	6-F	

Table 3. Details of reinforcement *(Donatı detayları)*

Structural members	**Dimensions (mm)**	**Rebars**
Slab	h=100	Ø10/300 plain, Ø10/300 bent-up
Beam	300 x 500	2Ø10 (top), 5Ø14 (bottom), Ø8/250 (stirrup)
Columns	500 x 500	8Ø18, Ø8/200
Foundation	h=700	14Ø20 (top), 14Ø20 (bottom)

3.2. Analysis of the Existing Structure and Results *(Mevcut Yapının Analizi ve Sonuçlar)*

In the light of the information given in the previous sections, the analysis of the structure was carried out according to the Turkish Building Earthquake Code and results were obtained according to the 9 load combinations (1,4G+1,6Q, G+Q+Ex, G+Q-Ex, G+Q+Ey, G+Q-Ey, 0,9G+Ex, 0,9G-Ex, 0,9G+Ey ve 0,9G-Ey) given in TS500. In the combinations, it was observed that the worst-case loading was G+Q+Ey and the relative story drifts obtained for this combination are given in Table 4. Information about natural periods of the existing building is given in Table 5. When all the results related to storey drifts are examined, it was seen that there was no torsional irregularity in both X and Y directions. The required controls were performed for the soft storey irregularity using the Table 6 for the earthquake in the X direction and the Table 7 for the earthquake in the Y direction according to the earthquake code.



$S_S = 0.440$ $S_1 = 0.129$ $S_{DS} = 0.396$ $S_{D1} = 0.103$

$PGA = 0.189$ $PGV = 11.257$

(c)

Figure 4. (a) Horizontal and (b) Vertical elastic design spectra, (c) Seismic design parameters obtained from 2018 Seismic Hazard Map *((a) yatay ve (b) düşey elastic spektrum, (c) 2018 Sismik Tehlike Haritası'ndan ele edilen sismik tasarım parameterleri parametreleri)*

Table 4. Relative storey drifts of the existing building
(Mevcut binanın göreli kat ötelenmesi)

Storey	**U1 (mm)**	**U2 (mm)**	**U3 (mm)**
1	2.55	17.26	-6.73
2	2.92	17.13	1.20
3	2.30	15.54	-0.05
4	1.65	13.08	-0.09
5	1.51	11.95	-0.12
6	1.39	10.79	-0.13
7	1.24	9.30	-0.13
8	1.10	7.44	-0.12
9	0.91	5.46	-0.09
10	0.72	3.42	-0.05

Table 5. Natural periods of the existing building
(Mevcut binanın doğal periyotları)

Mode	Period (sec)	Frequency (1/sec)	CircFreq (rad/sec)	Eigenvalue
Mode 1	2.14 (translation-y)	0.48	2.94	8,63
Mode 2	2.10 (translation-x)	0.47	2.99	8.94
Mode 3	1.88 (torsion)	0.53	3.33	11.12
Mode 4	0.68	1.46	9.18	84.21
Mode 5	0.67	1.47	9.27	85.90
Mode 6	0.61	1.64	10.32	106.47
Mode 7	0.39	2.60	16.31	26613
Mode 8	0.38	2.61	16.40	269.02
Mode 9	0.35	2.86	18.00	323.89
Mode 10	0.32	3.09	19.34	374.15
Mode 11	0.27	3.71	23.30	543.05
Mode 12	0.26	3.76	23.61	557,18

Table 6. Soft storey control in X direction *(X yönünde yumuşak kat kontrolü)*

Storey	**Loading**	**U1 mm**	**Control mm/mm**	**Result**
1	EX	13.07	0.51	< 2
2	EX	25.58	0.68	< 2
3	EX	37.36	0.77	< 2
4	EX	48.25	0.82	< 2
5	EX	58.34	0.86	< 2
6	EX	67.44	0.89	< 2
7	EX	75.31	0.92	< 2
8	EX	81.68	0.95	< 2
9	EX	86.42	0.96	< 2
10	EX	89.50	-	< 2

Table 7. Soft storey control in Y direction *(Y yönünde yumuşak kat kontrolü)*

Storey	**Loading**	**U1 mm**	**Control mm/mm**	**Result**
1	EY	17.24	0.50	< 2
2	EY	34.35	0.69	< 2
3	EY	49.94	0.79	< 2
4	EY	63.05	0.84	< 2
5	EY	75.02	0.87	< 2
6	EY	85.83	0.90	< 2
7	EY	95.12	0.93	< 2
8	EY	102.50	0.95	< 2
9	EY	107.89	0.97	< 2
10	EY	111.25		< 2

As a result of the analyzes carried according to TS500 and 2018 Turkey Earthquake Code, it was ssen that the load-bearing capacity of many columns has been exceeded and the strengthening was essential, as shown in Figure 5 in 3D and Figure 6 in 2D.

PRINT ISSN: 2149-4916 E-ISSN: 2149-9373

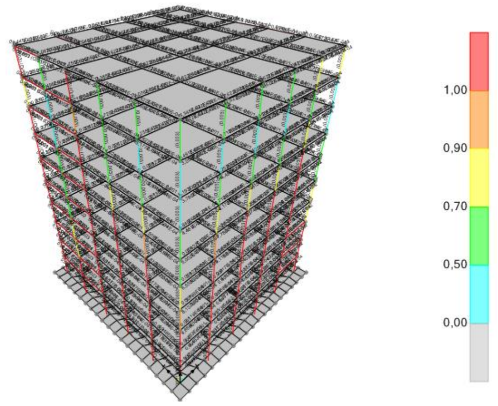
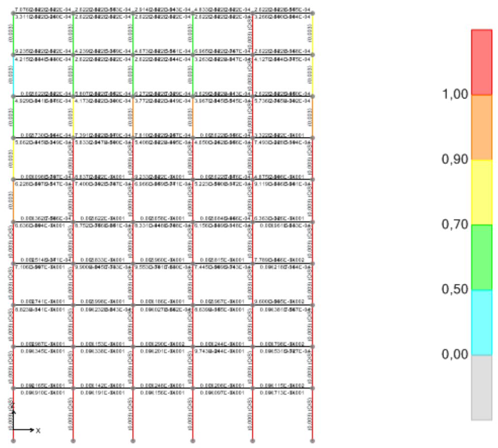
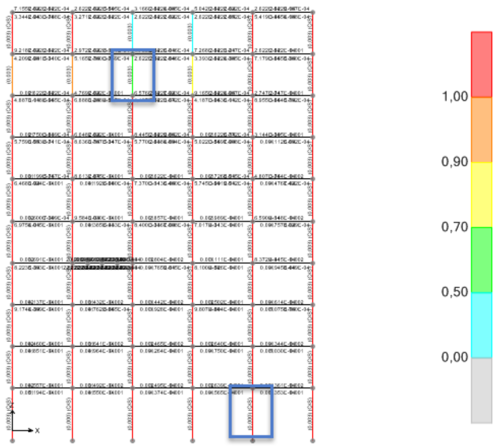


Figure 5. Exceeding the load bearing capacities of structural members (3D) *(Yapısal elemanların yük taşıma kapasitelerinin aşılması (3D))*



(a)



(b)

Figure 6. Exceeding the load bearing capacities of structural members (2D) *(Yapısal elemanların yük taşıma kapasitelerinin aşılması (2D))*

Interaction diagrams, which shows the bearing capacity, are the same for all columns since the cross-sections and reinforcement steel are the same. However, the axial force and moment values on the columns are different. The interaction diagrams are given in Figure 7 by examining the two columns whose bearing capacity is exceeded and within the bearing capacity limits which are shown in Figure 6.

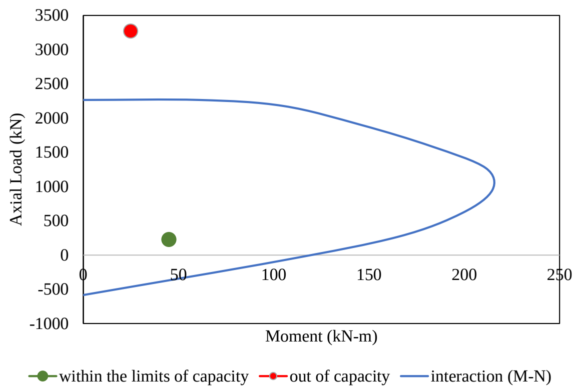


Figure 7. Interaction diagram of selected columns *(Seçilen kolonların karşılıklı etkileşim diyagramı)*

4. STRENGTHENING OF THE BUILDING *(BİNANIN GÜÇLENDİRİLMESİ)*

As a consequence of the examination of the existing building, it was seen that the strengthening of the structural-system is required. Concrete with 30 MPa compressive strength and steel with 420 MPa yield strength were used for the jacketing to be implemented in the strengthening. As depicted in the plan in Figure 8, a reinforced concrete wall with a width of 30 cm in accordance with the beam width was placed on the structure. During the placement stage of the reinforced concrete walls in the plan, it was aimed that the large forces in the structure would mostly exert on the outer columns and that keeping the overlap of the mass-rigidity center of the building. In the succeeding stage, the building was analyzed with SAP2000 software. However, the load-bearing capacity of some columns was exceeded. Hereby, it was decided to apply the jacketing to all columns and beams with sections shown in Figure 9. The column jacketing process was applied to all columns, whereas the beam jacketing process was applied only to the beams with cracks. A new reinforced concrete layer with a thickness of 125 mm was added around the column. Therefore, the column cross-section became 750 mm x 750 mm. While the beams were being strengthened, a new reinforced concrete layer was added with a thickness of 50 mm on the side surfaces of the beam and 100 mm on the bottom surface. In this layer, a total of 10Ø16 reinforcement steel was used on the side surfaces and 10Ø16 reinforcement on the lower surface. The general view of the strengthened structure is shown in Figure 10. The modulus of elasticity for C30 concrete was calculated theoretically and the structural behavior factor has been updated to 6.

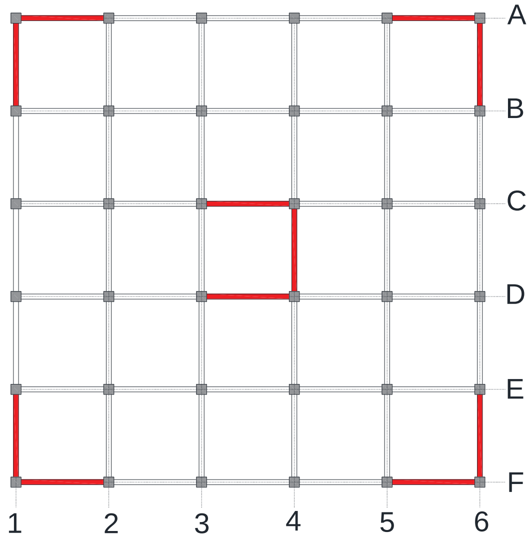


Figure 8. Lay-out of reinforced concrete walls
(Betonarme perde duvarların yerleşimi)

4.1. FEM Analysis of the Strengthened Building and Results (Güçlendirilen Binanın SEM analizi ve Sonuçlar)

In the light of the information given about the strengthening process, the analysis of the structure was performed and 9 load combinations given in TS500 (1.4G+1.6Q, G+Q+Ex, G+Q-Ex, G+Q+Ey, G+Q-Ey, 0.9G+Ex, 0.9G-Ex, 0.9G+Ey and 0.9G-Ey) were obtained. It was observed that the worst-case loading was G+Q+Ey and the relative story drifts obtained for this combination are given in Table 8. Information about natural periods of the strengthened building is given in Table 9. When all the results related to storey drifts are examined, it was seen that there was no torsional irregularity in both X and Y directions.

Table 8. Relative storey drifts of the strengthened building *(Güçlendirilen binanın göreli kat ötelenmesi)*

Storey	U1 (mm)	U2 (mm)	U3 (mm)
1	0.003	0.757	0.253
2	0.012	1.165	0.135
3	0.023	1.513	0.093
4	0.033	1.774	0.060
5	0.041	1.953	0.033
6	0.049	2.063	0.012
7	0.054	2.113	-0.003
8	0.060	2.114	-0.014
9	0.064	2.081	-0.017
10	0.066	2.020	-0.008

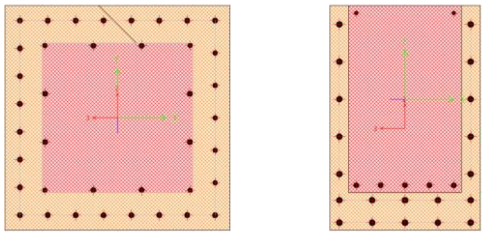


Figure 9. Strengthening details of (a) column, (b) beam *((a) kolon, (b) kiriş güçlendirme detayı)*

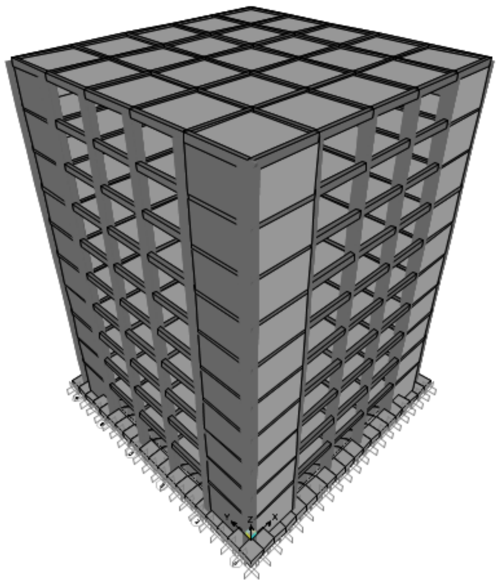


Figure 10. General view of strengthened building
(güçlendirilen binanın genel görünüşü)

Table 9. Natural periods of the strengthened building
(Güçlendirilen binanın doğal periyotları)

Mode	Period (sec)	Frequency (1/sec)	CircFreq (rad/sec)	Eigenvalue
Mode 1	0.597 (translation-y)	1.67	10.51	110.52
Mode 2	0.597 (translation-x)	1.67	10.52	110.72
Mode 3	0.395 (torsion)	2.52	15.88	252.46
Mode 4	0.136	7.35	46.19	2133.90
Mode 5	0.130	7.67	48.24	2327.40
Mode 6	0.097	10.23	64.28	4132.40
Mode 7	0.096	10.32	64.87	4209.20
Mode 8	0.096	10.32	64.88	4210.60
Mode 9	0.094	10.54	66.23	4386.60
Mode 10	0.094	10.57	66.41	4411.00
Mode 11	0.092	10.86	68.24	4657.50
Mode 12	0.092	10.86	68.26	4659.70

Table 10. Soft storey control in X direction
(X yönünde yumuşak kat kontrolü)

Storey	Loading	U1 mm	Control mm/mm	Result
1	EX	13.07	0.51	< 2
2	EX	25.58	0.68	< 2
3	EX	37.36	0.77	< 2
4	EX	48.25	0.82	< 2
5	EX	58.34	0.86	< 2
6	EX	67.44	0.89	< 2
7	EX	75.31	0.92	< 2
8	EX	81.68	0.95	< 2
9	EX	86.42	0.96	< 2
10	EX	89.50		< 2

The required controls for the earthquakes in X and Y directions were checked out in Table 10 and Table 11, respectively in terms of soft storey irregularity.

As a result of the FEM analyzes performed according to TS500 and TEC-2018, all of the load-bearing members were able to withstand the load combinations, as shown in Figure 11 in 3D and in Figure 12 in 2D.

Table 11. Soft storey control in Y direction *(Y yönünde yumuşak kat kontrolü)*

Storey	Loading	U1 mm	Control mm/mm	Result
1	EY	0.01	0.59	< 2
2	EY	0.02	0.74	< 2
3	EY	0.02	0.82	< 2
4	EY	0.03	0.87	< 2
5	EY	0.03	0.90	< 2
6	EY	0.04	0.93	< 2
7	EY	0.04	0.96	< 2
8	EY	0.04	0.98	< 2
9	EY	0.04	1.01	< 2
10	EY	0.04		< 2

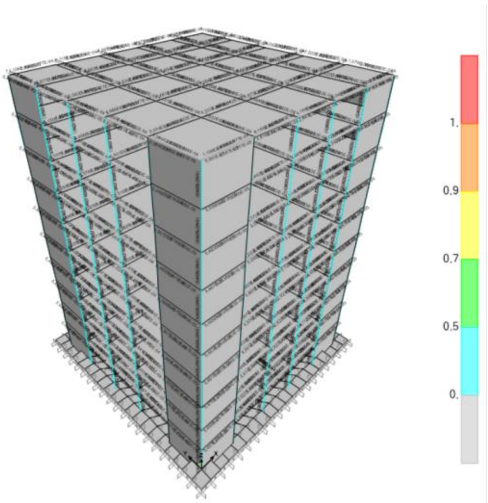


Figure 11. Control of load-bearing capacities of structural members (3D) *(Yapısal elemanların yük taşıma kapasitelerinin kontrolü (3D))*

Since the cross-sections and reinforcement steel of the columns are identical, the interaction diagrams are same for all columns. Nevertheless, the axial force and moment values on the columns are different. The interaction diagram of the strengthened columns is given in Figure 13 for the 5-D axis. Similar to the 5-D axis, the moment and axial-force values of all columns obtained from the numerical results are within the acceptable limits in the interaction diagram.

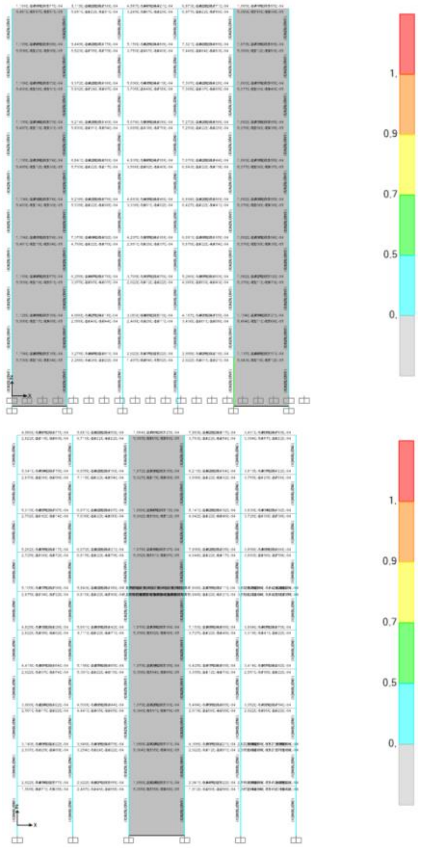


Figure 12. Control of load-bearing capacities of structural members (2D) *(Yapısal elemanların yük taşıma kapasitelerinin kontrolü (2D))*

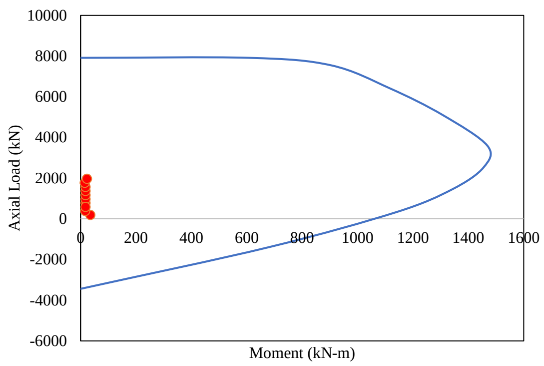


Figure 13. Interaction diagram of the strengthened columns *(Güçlendirilmiş kolonların karşılıklı etkileşim diyagramı)*

5. COMPARISON OF NUMERICAL RESULTS
(NÜMERİK SONUÇLARIN KARŞILAŞTIRILMASI)

In this part of the study, general behavior of the strengthened building is compared with the un-strengthened situation and the information about the load-bearing capacity increase of the columns and beams is presented.

The most important factor affecting the behavior of the building during an earthquake are the predominant periods and mode shapes of the building. The periods of the existing and the strengthened building are compared in Figure 14. Since the structural system of the existing building had a symmetrical lay-out, the behavior of the first two modes was in the form of translation. The behavior of the third mode was in the form of rotation. It was considered that this rotational behavior would prevent the large shear stresses that will occur as a result of torsion in the structure during the earthquake, and as a consequence attention has been paid to ensure that the center of rigidity does not move away from the center of gravity during the placement of the reinforced concrete shear walls added to the building. Therefore, there was no change in behavior in the predominant periods of the building, that is, the first two periods were observed as translation and the third period as rotation. As a result of the strengthening, the period of the building decreased from 2.14 to 0.60 seconds for mode-1, 2.10 to 0.60 seconds for mode-2 and 1.88 to 0.40 seconds for mode-3, respectively. Besides, the proportional increase in the rigidity of the building was approximately 13 times greater than the proportional increase in the weight of the building.

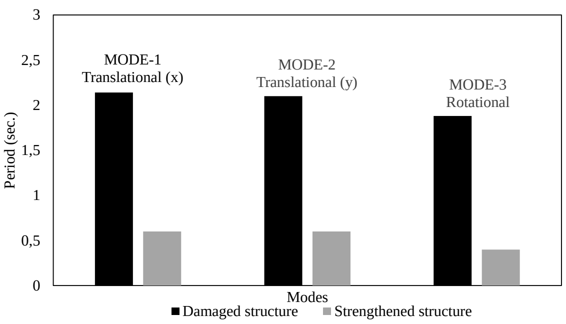


Figure 14. Period of the modes in the building *(Bina periyotları)*

The decrease in the period as a result of the increase in rigidity amplified the earthquake accelerations acting on the building. The information acquired from Figure 4 for the variation of horizontal and vertical earthquake accelerations are given in Figure 15 and Figure 16, respectively. As a result of the strengthening, the horizontal and vertical earthquake accelerations affecting the structure increased by average of 294% and 400%, respectively. In addition, 10 times decrease was observed in the strorey drifts of the strengthened building compare to the existing one.

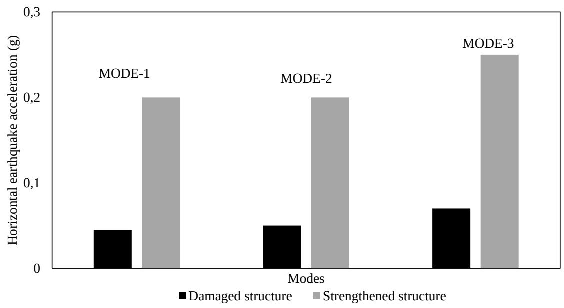


Figure 15. Horizontal earthquake accelerations depending on modes *(Modlara bağlı yatay deprem ivmeleri)*

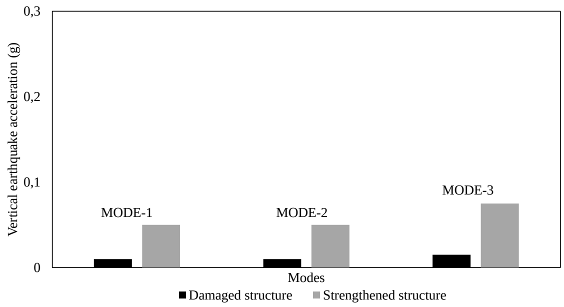


Figure 16. Vertical earthquake accelerations depending on modes *(Modlara bağlı düşey deprem ivmeleri)*

Significant increases occurred in the load-bearing capacities of the reinforced columns and beams. The comparison of the interaction diagrams of the reinforced columns before and after the strengthening is shown in Figure 17. In the case of zero moment, the axial compressive force bearing capacity and axial tensile force bearing capacity of reinforced concrete columns increased by 300% and 490%, respectively, compared to the unstrengthened ones. Whereas, In the case of zero axial force, the moment carrying capacity of the column increased by 666%.

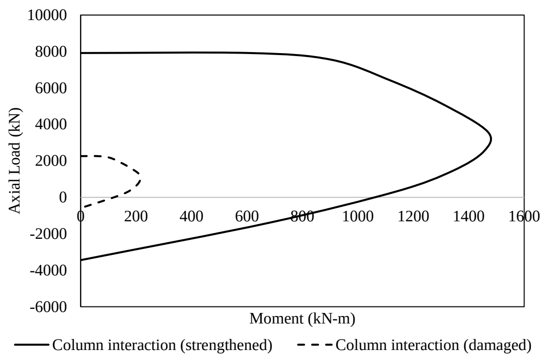


Figure 17. Comparison of column interaction diagrams *(Kolon karşılıklı etkileşim diyagramlarının karşılaştırılması)*

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One of the information specifically examined in the behavior of reinforced concrete beams is the moment-curvature relationship. The variation of the moment-curvature curve in strengthened beams with respect to the initial state is shown in Figure 18. The moment carrying capacity of the strengthened beams increased by 485%. When the strengthened beams reach their maximum bearing capacity, the rotational value decreased by 76% due to the increase in flexural stiffness compare to the unstrengthened beams.

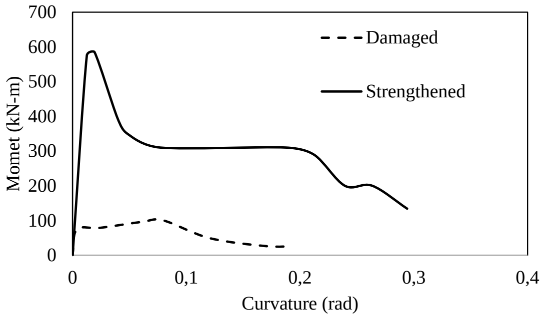

Figure 18. Comparison of moment-curvature diagrams of beams *(Kirişlerin moment-eğrilik diyagramlarının karşılaştırılması)*

6. CONCLUSION *(SONUÇLAR)*

In this study, finite element analysis of an insufficient reinforced concrete building, whose surveying was done, then, the crack distribution in the load-bearing members was detected and the concrete grade was determined with the samples taken, was performed using SAP2000 software. Considering the numerical results of the existing building, a new reinforced concrete layer was added to the columns and beams, and reinforced concrete walls were placed on the load-bearing system in order to improve the behavior of the structure. The FEM analysis of the rehabilitated building was carried out and the results were compared with the initial state. A summary of the main findings and the suggestions which have arisen in this study are listed as follows:

- It has been observed that the insufficient bearing capacities of the columns in the existed building have become sufficient according to TS500 and TEC-2018.

- Due to the increased rigidity after strengthening, the displacements in the building decreased for all combination cases stated in TS500.

- The predominant period of the building was decreased from 2.14 seconds to 0.60 seconds after the strengthening process.

- Depending on the change in the period of the building, the horizontal and vertical earthquake accelerations acting on the building increased by 294% and 400%, respectively. Additional forces due to this increase were safely carried by reinforced concrete shear walls and strengthened columns.

- The moment carrying capacity of the strengthened beams raised up to 5 times, and at this level, the rotation decreased by three fourths due to the increase in flexural stiffness.

- When compared to unstrengthened ones, in case of zero moment, the axial compressive force bearing capacity and the axial tensile force bearing capacity of the reinforced concrete columns increased up to 3 and 5 times, respectively. Whereas, in case of zero axial force, the moment carrying capacity of the rehabilitated columns increased up to 6,6 times.

It is revealed that the existing un-strengthened reinforced concrete buildings can be aligned with the current earthquake regulations and specifications by adding a new reinforced concrete layer to the load-carrying members and placing additional reinforced concrete shear walls on the structure. However, cost which is considered as one of the main factors in building management should also be taken into account. The author recommends comparing the expenses of the reconstruction and the strengthening of the building in future studies.

(CONFLICT OF INTEREST STATEMENT)
(ÇIKAR ÇATIŞMASI BİLDİRİMİ)

The authors declare that there is no conflict of interest.

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