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THE EFFECTS OF A COLUMN DAMAGE TO STRUCTURAL BEHAVIOR OF A LOW-STRENGTH CONCRETE BUILDING

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Keywords
Structural damage, Demolition, Nonlinear behavior, Performance analysis.

Abstract

In recent years, a new urban transformation process has been started to minimize the seismic hazard risk in Turkey. The main aim of this process is to avoid uncontrolled urbanization and faulty structuring by replacing old buildings with new seismic-resistant structures. Unfortunately, it brings new risks. The most important risk is formed with the damages which may occur on the adjacent buildings during the demolition. There are several cases and lawsuits reached to local authorities and courts about this issue. The main cause of these damages is the violation of the rules in regulations and ignorance of engineering principles related to demolition applications. In this study, a low-strength building in Istanbul is investigated which has been damaged by the demolition of an adjacent building. The effects of these damages to the seismic performance level of the building are analyzed with three different numerical models. Primarily, both seismic performance levels of undamaged and damaged states are evaluated using nonlinear analysis method. Furthermore, a new structural member is added to the analysis model using the damaged state's results to strengthen the damaged zone. The local structural behavior of the damaged zone and overall seismic performance of the building are investigated by three different cases aforementioned above to assess the results of this kind of damages. As a result of this study, the necessary strengthening approach is revealed by means of strengthening the local damaged zone or whole structure to provide necessary seismic performance level of the low-strength buildings having this type of damages.

DÜŞÜK DAYANIMLI BETONARME BİR YAPIDA OLUŞAN KOLON HASARI VE BU HASARIN YAPISAL DAVRANIŞA ETKİSİ

Anahtar Kelimeler
Yapısal hasar, Yıkım, Lineer olmayan davranış, Performans analizi.

Öz

Ülkemizde son yıllarda olası deprem risklerini minimize etmek amacıyla yeni bir kentleşme süreci başlatılmıştır. Kentlerimizdeki sağlıksız yapılaşmanın ve kontrolsüz kentleşmenin önüne geçmek amacıyla; depreme dayanıklı, daha sağlıklı yapılar üretmek için başlatılan bu süreç, bazı riskleri de beraberinde getirmiştir. Bu risklerden en önemlisi, yıkım çalışmaları esnasında komşu binaların zarar görmesi olarak ortaya çıkmıştır. Bu konuda belediyelere ve mahkemelere intikal eden birçok vaka bulunmaktadır. Söz konusu durumlara, fen ve sanat kurallarına aykırı olarak gerçekleştirilen yıkım uygulamaları sebep olmaktadır. Bu çalışmada İstanbul'da bulunan düşük-dayanımlı betonarme bir binada, yan parseldeki yıkım uygulamasından dolayı oluşan hasarlar incelenmiştir. Oluşan hasarların, yapının sismik performans seviyesine olan etkisi üç farklı analiz modeli oluşturularak belirlenmiştir. Öncelikle yapının hasarsız ve hasar aldığı durumundaki performans seviyesi nonlineer analiz yöntemi kullanılarak tespit edilmiştir. Ayrıca hasar alan bölgeyi güçlendirmek amacı ile yapının hasarlı durumu esas alınarak, söz konusu bölgeye yeni bir taşıyıcı eleman teşkil edilmiştir. Üç farklı durumda, hasarlı bölgenin lokal davranışı ve yapının genel performans seviyesi incelenerek deprem etkileri karşısında bu tip hasarın oluştuğu durumların ne gibi sonuçları olacağı irdelenmiştir. Yapılan çalışma neticesinde, benzer yapısal hasarların oluştuğu binalarda, gerekli sismik performans seviyesinin

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sağlanması için belirlenecek güçlendirme yaklaşımının, lokal veya tüm yapı için mi olması gerekliliği ortaya konmuştur.

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1. Introduction

Turkey is located on an earthquake-prone zone having a vast number of building stock under seismic risk. Thus, a significant amount of these buildings may either get heavily damaged or collapse in case of a strong ground-motion. Most of the present buildings (older than 20-30 years) especially in Istanbul are built with low-strength concrete and plain reinforcement bars. Thus, the government has announced a regulation (Resmi Gazete, 2012) about the urban transformation of the regions under seismic risk. This regulation aims to establish secure and regular environments by providing legal rights to re-construct the older buildings. The regulation also presents the appropriate methods and applications of demolishing and construction processes. However, in practice, there are several cases and lawsuits reached to the local authorities and courts about the damages occurred on the adjacent buildings during the re-construction applications. Most of these damages can be classified as beam/column cracks or destruction, which results from the hit of a construction equipment or movement of the adjacent building during demolition. The major aim of this study is investigating the effects of these kind of damages to structure's seismic performance which has already been built with low-strength concrete. Another concern of this study is determination of the appropriate strengthening method by means of strengthening the local damaged zone or whole structure to provide necessary seismic performance level. An example building located in Istanbul is considered to evaluate the aforementioned damages and their effects. The details about this building are collected from a lawsuit's technical reports, prepared by a commission of experts and a company (Çoşgun et al., 2014; Kütüğ et al., 2013; Dipu, 2013). The building built in 1987 and a column on the 4th story is totally damaged during an incident (demolition of an adjacent building) occurred in 2013. The effects of this damage to the seismic performance level of the building are analyzed with three different numerical models. Primarily, both seismic performance levels of undamaged and damaged states are evaluated using nonlinear analysis method. Furthermore, a new structural member is added to the analysis model using the damaged state's results to strengthen the damaged zone. The local structural behavior of the damaged zone and overall seismic performance of the building are investigated for three different cases mentioned above to assess the results of this kind of damages.

2. Nonlinear Analysis Methods

In Turkey, the rules for seismic performance assessment of buildings are dictated by "Specification for Structures to be Built in Disaster Areas (TSC, 2007)". The code presents three different methods for the nonlinear analysis of structures as "Repulsion Analysis with Incremental Equivalent Seismic Load Method", "Repulsion Analysis with Incremental Mode Combination Method" and "Calculation with the Non-linear within the Scope of Time Definition Method". In other words, first two methods are pushover methods using equivalent seismic load or mode combinations and the last one is the time-history analysis method. The details of these methods and the procedures to follow are given in the related code. The example building constructed with low-strength concrete which has irregularities in the plan and partial basement floor. Thus, the first method (pushover method using equivalent seismic load) is chosen due to the avoidance of extra complexity that can be emerged by time-history steps and modal combinations. The column, beam, and squat-walls (only surrounding basement floor against the soil) are modeled as frame members having lumped plasticity at the end zones. The fiber hinges are used in columns and walls which are useful for defining coupled axial and biaxial-bending behavior in frame objects. In fiber hinges, the cross-section is discretized into a series of representative axial fibers extending longitudinally along hinge length. Each fiber has a stress-strain relationship depending on the material in its tributary area. Integrating behavior over the cross-section, then multiplying by hinge length, provides axial force-deformation and biaxial moment-rotation relationships. The fiber hinge model is more accurate in that the nonlinear material relationship of each fiber automatically accounts for interaction, changes in along the moment-rotation curve, and plastic axial strain. A trade-off is that fiber application is more computationally intensive. Fiber hinges are ideal for dynamic behavior since they capture nonlinear hysteretic effects (CSI, 2017). In our country, there are several studies in the literature to capture the nonlinear behavior of different kind of buildings regarding the actual regulation. (Dinçer ve Mert, 2014; Uçar ve Düzgün, 2013; Korkmaz ve Düzgün, 2006; Aydınoğlu, 2003). Also, another case study for a different building having similar damages has been investigated by Çoşgun (2017) using nonlinear analysis.

3. Case Study

3.1. General Information

The damaged building has 7 stories (2 basement floors, ground floor, 4 typical floors) with 2.90 m average story height. The 2nd basement floor is a partial basement floor surrounded by squat-walls with 20 cm thickness. The squat-walls have Ø10 longitudinal and Ø8 lateral reinforcement bars with 200 mm spacing in each direction. The building has rectangular beams with 20/50, 20/60 and 20/70 cm dimensions. The bottom and top reinforcement ratio of the beams near the join region is %0.5 and %0.7, respectively. Stirrups of beams are Ø8 reinforcement bars with 200 mm spacing. In general, the building has various types of rectangular columns. The column dimensions decrease at 4th and 6th stories which causes abrupt changes in story stiffnesses. This is not an appropriate situation in the regions with high seismicity. Column dimensions are 25/25, 25/50, 30/50, 30/60, 30/65, 30/70, 30/75 and 30/80 cm with Ø16 longitudinal reinforcement bars and Ø8 stirrups having 250 mm spacing. Two-way reinforced concrete slabs with 10 and 12 cm thickness are used on each floor. The largest dimensions of the building in X and Y directions are 18.00 and 14.90 m., respectively. The building settles on approximately 200 m^2 area with a trapezoidal layout in the plan. The front and left side views of the building are shown in Figure 1. An example floor plan of the building is also given in Figure 2.

Figure 1. The views of the example building

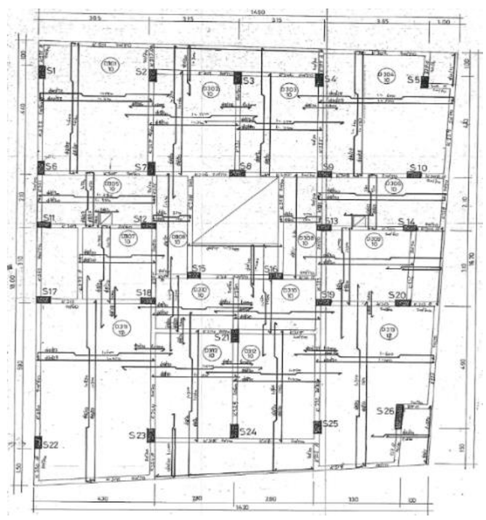

Figure 2. An example floor plan of the building

3.2. Laboratory Studies

The reinforcement details and the actual concrete compressive strength are determined based on in-situ and laboratory tests carried out by "DİPU İnşaat Proje ve Uygulama Ltd. Şti.". The properties of the soil are also presented in the report by DİPU, (2015). Several number of structural members are checked with monitoring devices and one edge of some columns are stripped to visually check the spacing and the diameter of the reinforcement bars. The spacing and the diameter of reinforcement bars are as mentioned in Chapter 3.1. The plain reinforcement bars with S220 (f_{sy}=220 MPa) grade steel quality are used. The actual unconfined concrete compressive strength is calculated using the test results of five specimens which are taken from various locations on the building. The test results and calculations based on the recommendations given in TSC (2007) are given in Table 1.

Table 1. Unconfined Concrete Compressive Strength Test Results

No	$f_{c,\ cube}$ (MPa)	Avg. $f_{c,\ cube}$ (MPa)	St. dev.	$f_{c,\ cube}$ (MPa)	$f_{c,\ cylinder}$ (MPa)	E_c (MPa)
1	16.10	13.50	2.16	11.34	9.64	24091
2	14.30					
3	11.00					
4	10.90					
5	15.20					
$f_{c,\,cube}$ = Avg. $f_{c,\,cube}$ – St. dev. ($f_{c,\,cube}$) $f_{c,\,cylinder}$ = 0.85 x $f_{c,\,cube}$						
E_c = 3250 $(f_{c,\,cylinder})^{1/2}$ + 14000						

It has been noted that the building's unconfined concrete compressive strength is around 9.64 MPa, which can be classified as low-strength concrete. The soil properties and spectrum characteristics in accordance with TSC (2007) are shown in Table 2.

Table 2. Soil Properties

Seismic Zone	2
Allowable Soil Bearing Stress	1.38 kg/cm^2
Soil Class	B
Local Site Class	Z3
Effective Ground Acceleration Coefficient	0.30
Spectrum Characteristic Periods (T_A-T_B)	0.15-0.60 sec.
Modulus of Subgrade Reaction	1656 t/m^3

The uncertainties related to both the structure and the soil are minimized using these test results in the analyses. Thus, nonlinear behavior of the building will be captured more precisely.

3.3. Investigation of the Damage

A column located at the periphery of the 4th story has been damaged with a hit of a construction equipment during the demolition process in the next parcel. The exact location and the damage on the related column are shown in Figures 3 and 4.

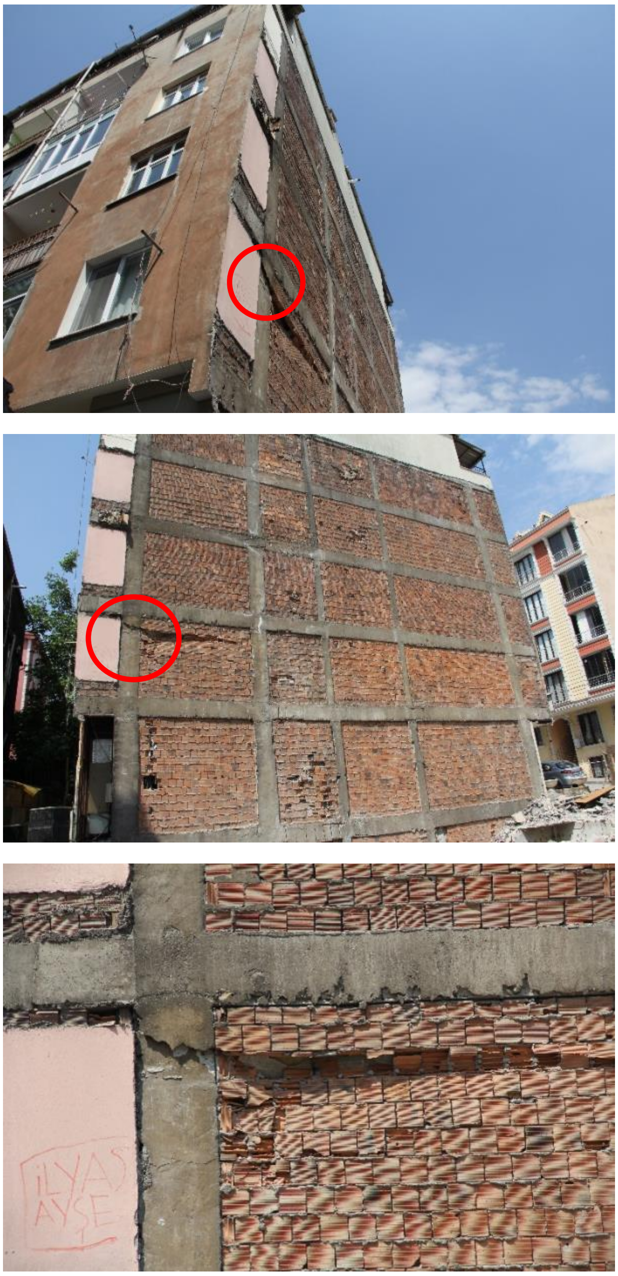

Figure 3. The damaged column (1/2)

The column is heavily damaged from a point near the top joint. It can be seen also in the next figures that the crack width is measured up to 5 cm. Some of the longitudinal reinforcements in the column are buckled and some of them are ruptured. It should be noted that, when the incident occurred, a part of the column was shifted from its vertical axis resulting in an undesired eccentricity. Thus, this situation should be considered while determining an appropriate strengthening method.

Figure 4. The damaged column (2/2)

Two different strengthening approaches may be followed for this region. As the first option, the crack can be filled with high-strength cement-based materials. As the second option, the column can be re-constructed keeping the unbuckled part of longitudinal reinforcements and total destruction of the damaged column. Thus, new bars will be attached to existing bars and concrete can be filled around. Since most of the existing reinforcements are ruptured in the related area, the second option will be preferable.

4. Nonlinear Finite Element Analyses

The effects of this structural damage to the seismic performance level of the building are assessed with three different mathematical models. Primarily, both seismic performance levels of undamaged and damaged states are evaluated using nonlinear finite element analysis method. Furthermore, the damaged model is reconstructed by placing a new column to the damaged zone considering the damaged state's results.

4.1. Analysis Approach and Models

Three-dimensional analyses were conducted using the structural analysis software ETABS (CSI, 2016) for nonlinear analysis of the structure. The effective stiffness for beams is assumed as 0.4EI (flexural rigidity of the cross-section) according to TSC (2007). Live load acting on the structure is defined as 2.0 kN/m^2 and floor coating load is defined as 1.5 kN/m^2. The partition wall loads are applied as line loads on related beams (3.2 kN/m^2 x story height at the periphery and 2.5 kN/m^2 x story height at the interior). Live load participation factor for residential buildings is defined as "0.3" in TSC (2007). The building information level is assumed as "Medium Information Level" defined in TSC (2007). Thus, 90% of the actual material strength values are used in the analyses. Beam, column and squat-wall elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the members (Kazaz et al., 2012). Fiber hinges are used in vertical structural members. Analysis model is shown in Figure 5.

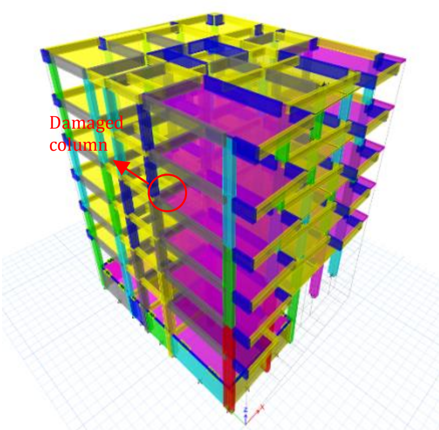

Figure 5. 3D View of the analysis model

Incremental single mode pushover scenarios start with vertical pushover with full loading. Then, lateral pushover steps are continued up to the target displacement values assuming vertical pushover results as initial conditions. Three different cases are created to simulate the undamaged, damaged and strengthened states. Nonlinear analysis is carried in three dimensions (PushG, PushX, PushY) to capture the behavior of the structure. In the first case, the undamaged state of the structure is analyzed. In the second case, the damaged column is removed and analyses are repeated. In the third case, a new column is attached instead of the damaged one. The vertical displacement obtained from the second model at the joint supported by damaged column is applied to the third case by defining a nonlinear link (gap) element connecting the top joint and new column's top end. By this manner, the columnless case's vertical displacement until the start of lateral pushover step is taken into consideration. This technique provides a better approach to consider the vertical displacement occurred during the absence of column support. The attached column has moment releases at both ends. Figure 6 shows the details of three cases.

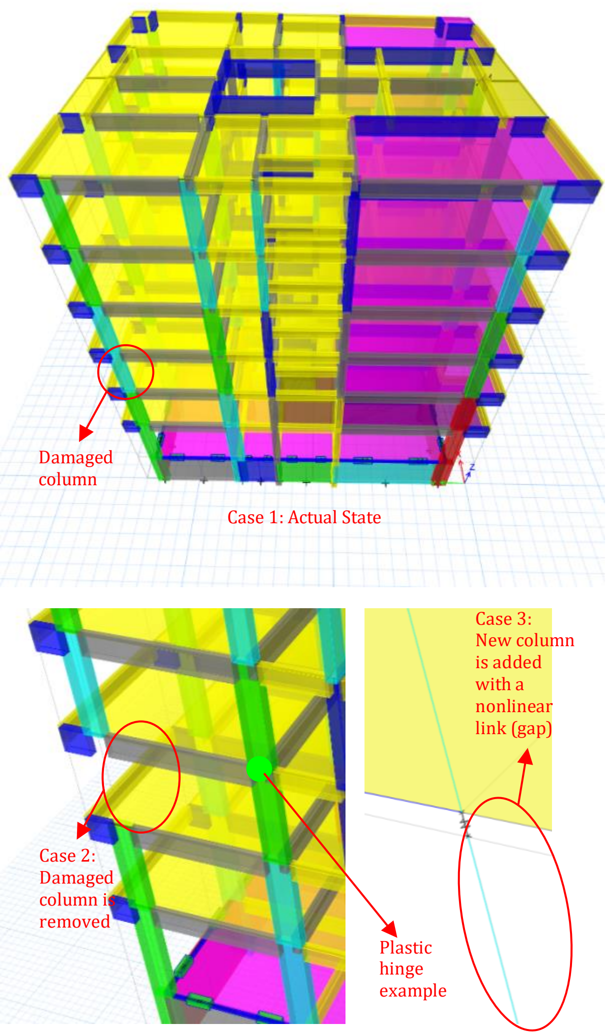

Figure 6. Representation of 3 different models (1st actual, 2nd damaged, 3rd strengthened)

4.2. Analysis Results

The local behavior of the damaged region and overall pushover analysis results of the three different cases are presented in this chapter. Target displacement of this building is calculated as 142 mm in X direction and 154 mm in Y direction. However, the top displacements in X and Y directions could be reached up to100 and 70 mm, respectively. The achieved top displacements are smaller than the target displacement values due to the formation of large number of plastic hinge mechanisms. In other words, lateral movement is halted after the limits mentioned above which can also be seen in the pushover curves (Figure 9). The plastic hinge formation at the end of vertical pushover analysis for each case is given in Figure 7.

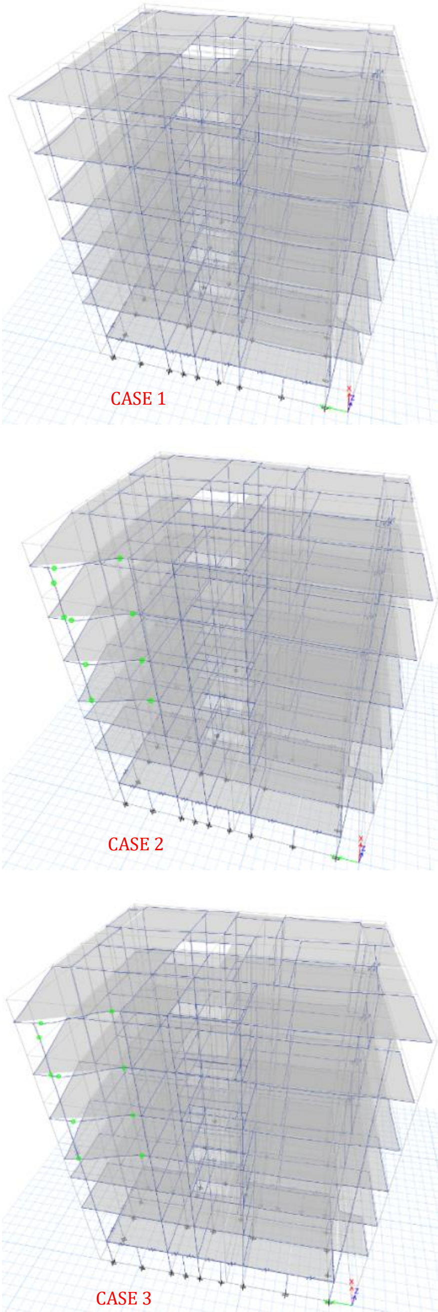

Figure 7. Plastic Hinge formation at the end of vertical pushover (Push-G) analysis

In the first case, no plastic hinge formation observed since the region was not damaged. In Case 2, the vertical deformation of the top joint is calculated as 21 mm. The vertical movement of the damaged region caused the formation of plastic hinges on the members located above the damaged zone. Since the first case's vertical deformation freedom on the damaged region's top joint is assigned to case 3 with a nonlinear gap element, similar plastic hinge formation is observed in the case 3. It should be noted that, in the case where the damaged zone is represented by a deleted column, the damaged region affected most of the beams connecting the columns on the upper floors. The plastic hinge formation on damaged members shows that, yield moment capacity of the members is exceeded and irreversible plastic deformations are occurred. The lateral pushover cases' hinge formations for both directions are given in Figure 8.

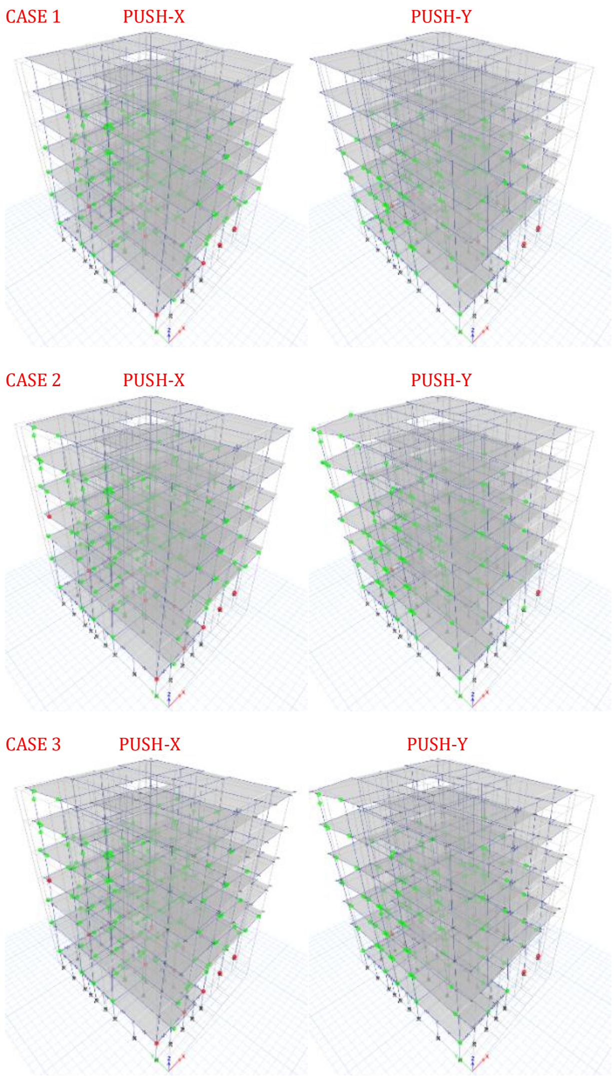

Figure 8. Plastic Hinge formation at the end of lateral (Push-X and Push-Y) pushover analysis

Deformed shapes at the end of lateral pushover analysis of all cases show that some columns located on the base floors exceeded their ultimate strain limit (red marked ones). Some beams and columns are also exceeded their yield strain limit (green marked ones). The upper column located above the damaged zone reached its ultimate strain capacity in X direction's pushover analysis (Case2&3). This is because of previously occurred plastic deformations at the end of vertical pushover analysis. Most of the structural members located above the damaged region have irreversible plastic deformations in cases 2 and 3. Same members do not have such deformations in case 1. The results of an example plastic hinge on a beam connected to the damaged region (marked with a green circle in Figure 6) is shown in Figure 9 in order to compare the aforementioned effects. The pushover curves of both cases are given in Figure 10.

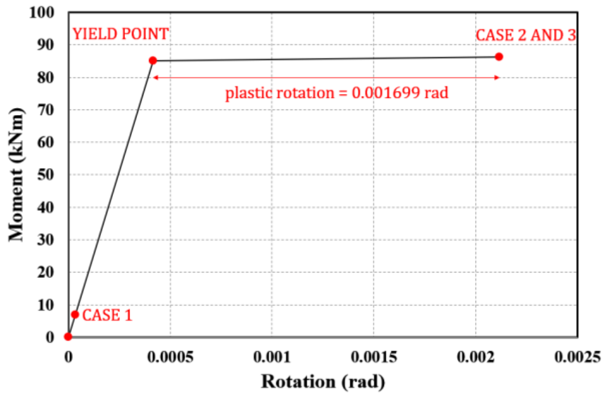

Figure 9. Moment-rotation results of an example plastic hinge at the end of the vertical pushover analysis (Push-G)

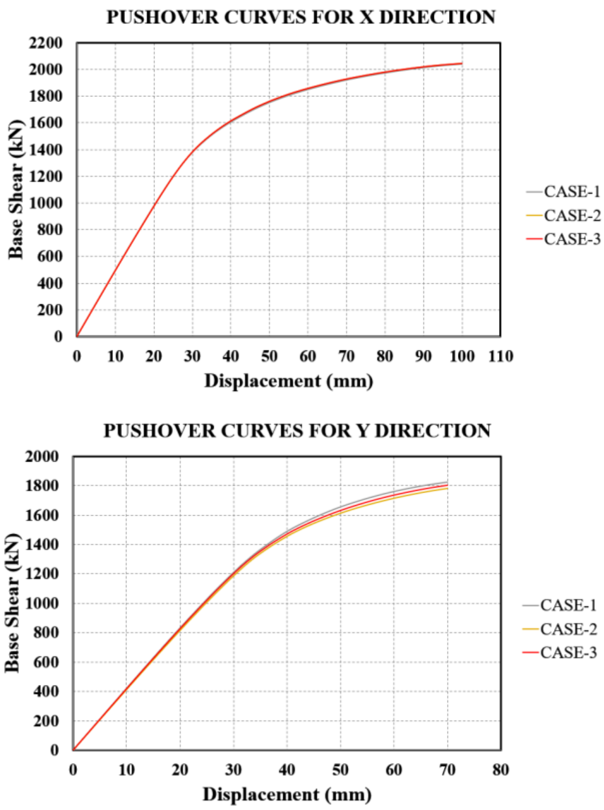

Figure 10. Pushover curves

At the end of vertical pushover analysis (Push-G), the example beam exceeded its yield moment capacity (around 85kNm) and plastic rotations are formed as 0.001699 rad in cases 2 and 3. However, when the same beams' plastic hinge state is considered for the undamaged case, the moment on the section is far below than the yield moment value (around 6.89 kNm) and there is not any plastic rotation occurred. In other words, for the undamaged model (case 1), the hinge states of the structural elements in the related region have remained in the elastic zone.

It is understood from the pushover curves that in X direction, all of three cases have given nearly same results. However, case 1 has higher performance than case 3 which is also higher than case 2 in Y direction. The higher performance of case 3 than case 2 shows the effect of strengthening on the overall seismic performance of the building. When the base shear values are compared, the difference between three cases' results in Y direction is around 1.3%. Since the trend of the three curves is nearly similar, this value may not be considered as a significant difference. The maximum base shear for three cases are found around 2040 kN in X direction and 1840 kN in Y direction. The building's overall performance is not enough to reach the target displacement value. It can be said that the local damaged column does not have a significant role in the seismic performance of the building with respect to these results.

5. Conclusion

One of the most important risks in the urban transformation projects is the damages which may occur on the adjacent buildings during the demolition. Concerning this issue, a low-strength building in Istanbul is investigated which has been damaged by the demolition of an adjacent building. The effects of these damages to the seismic performance level of the building are analyzed with three different numerical models. First two models are for evaluating the seismic performance levels of undamaged (case 1) and damaged (case 2) states with nonlinear analysis. The last model (case 3) is generated to strengthen the damaged zone by a new column which is added to the damaged analysis model, keeping the damaged state's vertical pushover results. For each case, it is certain that the building could not reach the desired target displacement in both orthogonal directions. In other words, the building is not structurally capable enough to deform until the required seismic performance level achieved. This situation shows the urgent need for an overall rehabilitation or reconstruction of the related building.

The plastic hinge states at the end of vertical pushover analyses show that in the damaged case (case 2), irreversible plastic deformations occurred on the members connected above the damaged region. Thus, a localized rehabilitation cannot bring back the initial state even if the building is capable enough to reach its seismic performance level. The effect of strengthening can be gathered from the comparison of lateral pushover curves for each case. This arises by a 1.3% increase in base shear only in Y direction. It is believed by the authors that this difference can't be accepted as a significant effect. The building should be strengthened by an overall approach. More clearly, the capacity of columns and beams should be increased with an appropriate application of steel jackets, FRP sheets or concrete jackets. This will improve the structural behavior of the system by avoiding high plastic deformations on the plastic hinges while achieving the desired seismic performance level. The structural damage occurred in the related column can be repaired as described in this study for only to temporarily hold the related region against possible increase of vertical displacements in time due to environmental conditions until the overall strengthening is completed. It is also believed that if this type of damage occurs on a column located in the basement floors, the difference shown on the pushover curves will be more distinctive. The approach presented in this study can be used to investigate the effects of various types of damages on structural behavior and seismic performance of buildings for further studies related to similar situations or incidents.

Conflict of Interest
No conflict of interest was declared by the authors.

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