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A Proposal For Strengthening of RC Structures By Using High Strength Lighweight Concrete Panels and Polyurethane Binders

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Abstract: In that study, the formerly proposed method for seismic retrofitting of RC frames with fiber reinforced lightweight panels and polymer binders were re-tested with full scale frames for the first time in a numerical analysis performed by Abaqus . Later, modelling was performed with beam elements and equivalent compressive strut approach in Sap2000 and a validation of modelling was done. Then ,the method was applied to 3 story and 5 story frames by using beam elements and equivalent compressive strut approach in Sap2000. Retrofitting method increased the lateral load capacity by 2.76 times if compared with traditionally infilled frame according to the analysis done in single story frames. In 3 story frames, the retrofitting with panels increased the lateral load capacity by 3 times and in 5 story frames by 3.18 times if the retrofitted specimens were compared with traditionally infilled structures. Also, the strengthening of infill walls with panels is effective for increasing stiffness of structures.

Betonarme Yapıların Yüksek Mukavemetli Hafif Beton Paneller ve Poliüretan Bağlayıcılar Kullanılarak Güçlendirilmesi için Bir Öneri

Anahtar Kelimeler;
Polimer bağlayıcı,
Abaqus,
Sap2000,
Güçlendirme,
Beton panel

Özet: Bu çalışmada, betonarme çerçevelerin fiber takviyeli hafif paneller ve polimer bağlayıcılarla sismik olarak güçlendirilmesi için daha önce önerilen yöntem, Abaqus tarafından gerçekleştirilen sayısal bir analizde ilk kez tam ölçekli çerçevelerle yeniden test edildi. Daha sonra Sap2000'de kiriş elemanları ve eşdeğer basınç çubuğu yaklaşımıyla modelleme yapılarak modellemenin doğrulanması yapılmıştır. Daha sonra yöntem Sap2000'de kiriş elemanları ve eşdeğer basınç çubuğu yaklaşımı kullanılarak 3 katlı ve 5 katlı çerçevelere uygulanmıştır. Tek katlı çerçevelerde yapılan analize göre güçlendirme yöntemi, geleneksel dolgulu çerçeveye göre yanal yük kapasitesini 2.76 kat artırmıştır. Eğer güçlendirilen çerçeveler geleneksel dolgulu çerçevelerle karşılaştırılsa, 3 katlı çerçevelerde panellerle güçlendirme, yanal yük kapasitesini 3 kat, 5 katlı çerçevelerde ise 3.18 kat artırmıştır. Ayrıca dolgu duvarların panellerle güçlendirilmesi yapıların rijitliğinin arttırılmasında etkilidir.

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1. Introduction

Destructive earthquakes caused many casulties around the globe in the past. In the last 30 years, the improvement of performance based methods, the improvements in computational methods, the improvements in construction practice decreased the risk in developed countries whereas the risky building stock of developing countries continued to threaten the human lives. Recent earthquakes in Kahramanmaraş region in 2023 revealed this fact once again. An earthquake with a magnitude of 7.8 happened in Pazarcik, Kahramanmaraş. Just after 9 hours, an earthquake with a magnitude of 7.5 happened around 100km away in Elbistan, Kahramanmaraş. This means the damaged buildings due to the first strong ground motion were forced to consume energy again due to the second ground motion. One can argue the approach followed in seismic codes because it's based on the ductility of RC structures and the existence of two powerful earthquakes in a short period demands high deformation from structures.

Unfortunately, Kahramanmaraş earthquakes will not be the last strong earthquakes. In a previous study, the probability of an earthquake with a magnitude of 7.0 or higher in the sea of Marmara between 2000 and 2030 was given as 62% (±15) [1]. More recent studies agreed with this expectation. In one study, it's said that the possibility of breaking in the Marmara fault in a single earthquake can not be disregarded [2]. These facts indicate that a strong earthquake can soon cause catastrophic damage in Istanbul.

Practical and feasible seismic retrofit methods are needed to strengthen risky building stock and/or to improve the performance of relatively new buildings which were constructed according to the codes. One of most practical method for strengthening of existing buildings is the application of CFRP wrapping of columns and beams or application of CFRP for strenghtening infill walls. For instance in a past numerical and experimental study, it's concluded that retrofitting the frame's and infill wall with CFRP material increased the load capacity of RC frames by a factor of 2 to 3 [3]. In a separate research study [4], a proposal was made to retrofit infill walls within reinforced concrete (RC) frames using CFRP (Carbon Fiber Reinforced Polymer) strips. In this context, a model to define the ductility ,stifness and load capacity of frame was developed for the retrofitted frames. The model's validity was confirmed through experimental verification, utilizing data from two independent studies. Subsequently, this model was employed to analyze a 3-story RC structure constructed in the 1970s [4]. The analysis concluded that, based on the pushover results, retrofitting infill walls with CFRP, significantly enhanced the structure's stiffness and strength, eliminating the need for additional retrofitting methods [4].

But the cost of CFRP material and neediness of careful application generally restricts its widely usage in construction practice in Türkiye. Still the traditional strengthening approach in practice is based on increasing the lateral rigidity of structures by adding RC shear walls. In 1985, after Vina Del Mar earthquake which is happened in Chile with a moment magnitude of 7.5, observations showed that the usage of RC shear walls in design of RC structures prevented a big catastrophy [5]. However adding shear walls to the structures requires labor and time. During the construction of new shear walls, residents often loses their access to the building. This fact restricts the application of method widely especially in residental buildings.

Previously, a method was proposed to strengthen existing infill walls by using precast concrete panels [6]. The method aimed to disturb residents not more than a simple painting process inside the building. Later that idea was improved by using high strength fiber reinforced lightweight concrete panels and polyurethane binders between panels and walls in a numerical study. Application of high strength lightweight panels does not require sophisticated workanship. Also their specific weight is 1966kg/m^3 which is 18% lighter than conventional concrete. The numerical study was performed by using scaled RC frames and it's seen that using polyurethane binders prevents the detachment of panels from the wall and the method increases the lateral load carrying capacity by 2 to 4.5 times if frames with hollow brick infills are thought as reference specimens [7]. Using polyurethane binder prevented the detachment of panels from wall even in 6% lateral drifts [7]. Polyurethane binders were first proposed in a previous study for [8] the purpose of masonry strengthening with and without CFRP and it was concluded that the deformation capabilities of rubber like material reduces the stress concentrations and provides ductility to masonry. In a recent study [9], the material was used between mortarless wall and RC frame members and the results showed that using polymer binder decreased the initial stiffness of frame and the stresses in the mortarless wall decreased.

In this study, the previously suggested idea was re-tested in numerical analysis performed in Abaqus [10] (to gain knowledge) and model it with simplier finite elements in Sap2000 [11] softwares. For that purpose, a full scale RC frame from a previous experimental and numerical study was chosen [12]. First, full scale RC frames were subjected to a quasi-static lateral loading in Abaqus and the behavior was understood. Later the numerical analysis was extended by using beam elements in Sap 2000 software. In Sap 2000 software the infill walls were modelled by using imaginary compressive struts which have pinned ends. The results were compared with the detailed finite element analysis performed in Abaqus and validation of compressive strut approach was done. Later, multistory frames were modelled and capacity curves were obtained. The results show that the proposed method is promising to strengthen risky building stock and/or to improve the performance of relatively new buildings which were constructed according to the codes.

2. Materials and Method

The dimensions of the frame were derived from a past study in which both experimental and numerical analyses were conducted using Abaqus [12]. This allows for a comparison of the results presented here with those obtained from previous experimental and numerical data. That comparison was performed in a separate study where metallic elements were proposed between frame and panel [13]. However, it's important to note that the steel and concrete materials were replaced with local equivalents, the differences in material strength fell within an acceptable range. In the previous study, the compressive strength of the concrete in Abaqus was 27.74MPa, whereas here it's considered to be 30MPa (cubic strength). Full-scale single-story RC frames were employed for the analysis [13].

For the steel used in the previous experimental and numerical analyses [12], the material's yield and tensile strengths were 472MPa and 656MPa, respectively. In this study, the values for yielding and tensile strength were 491MPa and 553MPa, respectively, for longitudinal rebars. In terms of confinement rebars, the previous study utilized steel with a yielding strength of 308MPa, whereas in this study, the yielding strength was 277MPa, as previously explained.

The column height was set at 2600mm, measured to the bottom of the beam, and the bay length was 2800mm. The assumed column cross-section was 350mm x 350mm, while the beam cross-section was 350mm x 400mm. The longitudinal rebars for columns were assumed to be 8φ16, and for beams, 4φ16. Confinement rebars for both beams and columns were of size φ8/100/150mm.

In the numerical analysis, the base of each column was considered as a fixed support, eliminating the need to separately model a foundation beam. A vertical load of 700kN (5.71 MPa applied to the column surface) was imposed on the columns. Subsequently, a second specimen was prepared with a wall panel and metallic elements inside the frame. Figure 1 displays the properties and rebar details of the RC frames obtained from the previous study[12] and the proposed retrofitting method . All measurements are presented in millimeters and Figure 1 also illustrates the proposed strenghtening system with panels. The panels which have interaction with frame elements were assumed to be anchored to the frame elements. The situation whether they are anchored or not changes the behavior as indicated in the previous study [7]. Between panels and wall polymer binder is sued rather than epoxy. Because as indicated in the previous study [7], polymer binder allows a ductile behavior after 3% lateral drift and even in 6% lateral drifts no detachment between wall and panels happen. However in epoxy situation detachment happens and frame shows a brittle behavior after 3% lateral drift.

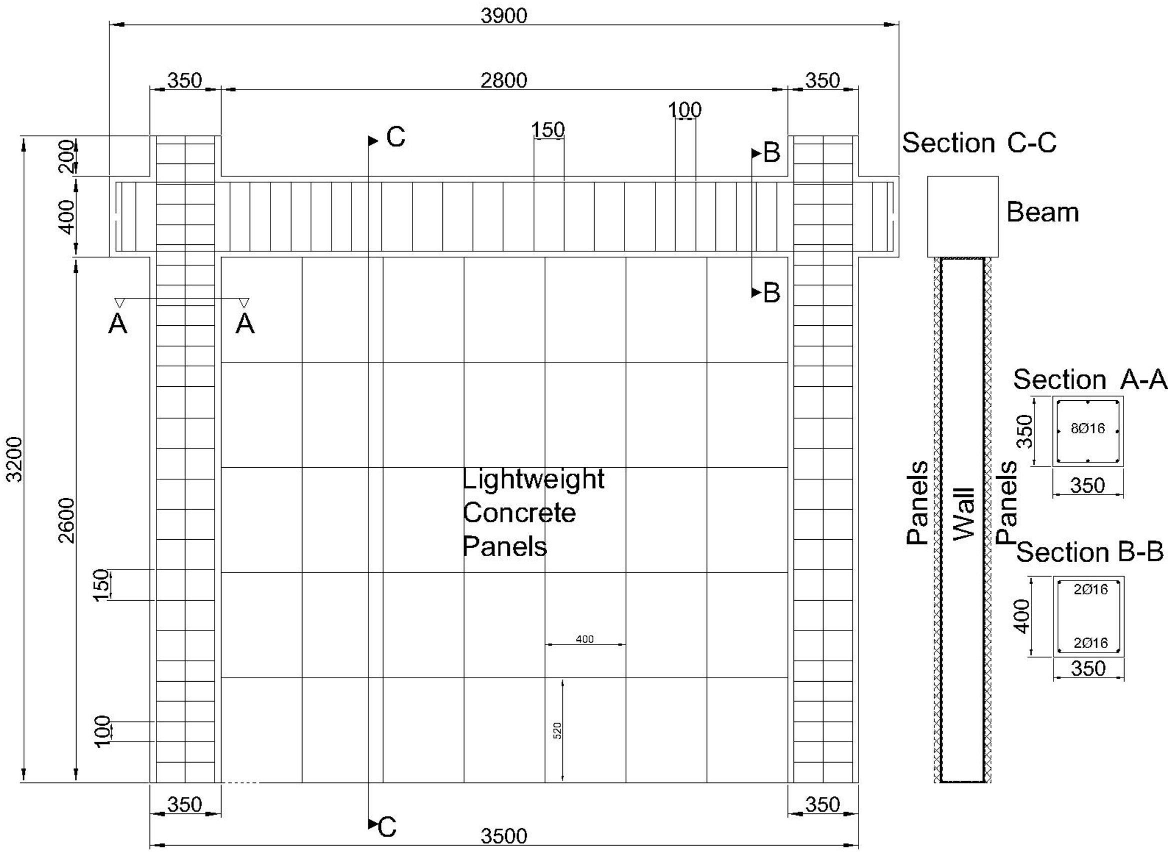


Figure 1. The properties of frame taken from past study [12] and proposed retrofitting method

2.1. Abaqus modelling

The frame was modelled in Abaqus sofware and Sap2000 software separately. In Abaqus software C3D8R elements were implemented for all kinds of solid elements except rebars. For the rebars of RC frame, the modelling was done by using "wire" element in the software. A two noded linear beam element called B31 was used in Abaqus to model rebars. All of the rebars were embedded inside the concrete frame in the software to define adherence between rebars and concrete. That way the frame was considered as a reinforced concrete cross section. The frame was pushed up to 0.026 lateral drift ratio in the past study [13] and load displacement curve was compared with the previous experimental and numerical study [12]. A good agreement was found. In this study, bare frame results were used again. Also a frame with a traditional infill wall is prepared. Traditional hollow clay bricks were used for the construction of wall. Thirdly, another frame was prepared in Abaqus by applying the proposed retrofitting method with high strength lightweight fiber reinforced panels.

Abaqus employs the finite element method to initially establish system stiffness, mass, and damping matrices. In the numerical solution of the equation of motion within explicit analysis, Abaqus utilizes the central difference method. In this method, stiffness, mass, and damping matrices are not needed to be rebuilt at each step, and displacements at the I+1 step are determined based on displacements at the I and I-1 steps [14]. In explicit dynamic analysis with Abaqus, computational costs are generally low. A substantial portion of computational power is dedicated to calculating the internal forces of elements.

When performing quasi-static analysis using explicit dynamic analysis in Abaqus, certain conditions must be met because a static problem is transformed into a dynamic one. To consider the problem as quasi-static, inertial forces need to be maintained at a certain level. The control of the kinetic energy's ratio to the total internal energy of the deforming material is essential after the analysis. If this ratio is less than 0.10, the analysis can be regarded as quasi-static [14]. The finite elements (mesh) of the systems can be seen in Figure 2. 50mm elements were used to model frame, mortar and panels and100mm elements were used to model traditional infill wall and polymer layer between wall and panels.

The column bases were assumed to be fixed support in bare frame analysis and in frame analysis with walls, wall bases were assumed to be fixed support also. The loading conditions can be seen in Figure 2.

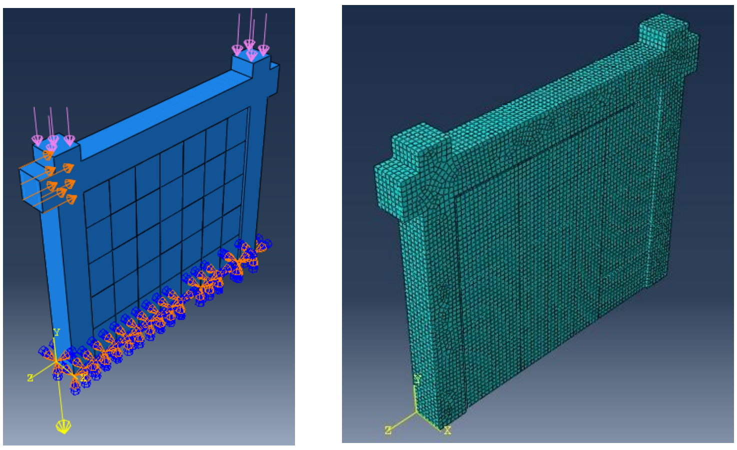


Figure 2. a. The loading conditions of strengthened frame **b.** The mesh of frame with proposed retrofitting method

2.1.1 Modelling the concrete of frame in abaqus

In the field of literature, various theories and models have been proposed to predict the plastic behavior of materials when subjected to multidirectional stresses. These stresses can be conceptualized as a combination of a hydrostatic component and a deviatoric component. The different theories consider either the hydrostatic part, the deviatoric part, or both of them to forecast material behavior. Von Mises theory asserts that the hydrostatic component of stress can be disregarded, assuming that only the deviatoric component is responsible for plastic deformation. In contrast to Von Mises theory, the Drucker-Prager theory takes into account the influence of both the hydrostatic and deviatoric components of stress. Within the Abaqus software, the Concrete Damaged Plasticity (CDP) model is available for concrete modeling, and it is a modification of the Drucker-Prager model. As depicted in Figure 3 [15], the K coefficient determines the modification. In the CDP model, the K coefficient is assumed to be 2/3, whereas in the Drucker-Prager model, it is assumed to be 1.

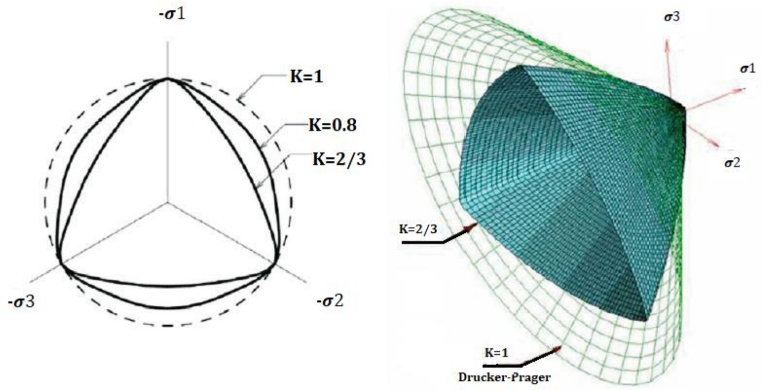


Figure 3. Demonstration of CDP model yielding surface [15]

Abaqus uses the method shown in Figure 4 [16] to establish the compressive stress and its corresponding strain for concrete. Figure 4 indicates the d_c coefficient to consider the impact of changes in the plasticity of concrete under compression. Abaqus relies on the relationship between plastic strain and corresponding stress values for this purpose.

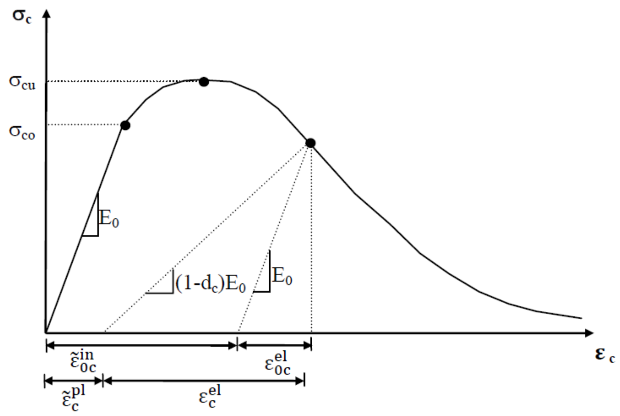


Figure 4. Stress-strain relationships of concrete [16]

The stress -strain values must be determined to follow the method expressed in Figure 4 in Abaqus. The strain corresponding to the maximum stress was assumed as $0.0022(\varepsilon_0)$ for C25 concrete class and the modulus of elasticity was assumed as 31000MPa. The cubical strength of C25 concrete (f_0) was taken as 30MPa. In Abaqus cubical strength value must be implemented in the software. A previous study [17] contains the following formulation to determine the values in the stress-strain graph of concrete.

$$\frac{f}{f_0} = 2.1\left(\frac{\varepsilon}{\varepsilon_0}\right) - 1.33\left(\frac{\varepsilon}{\varepsilon_0}\right)^2 + 0.2\left(\frac{\varepsilon}{\varepsilon_0}\right)^3 \quad (1)$$

The tensile strength of concrete is calculated as 0.7 times the square root of f_{ck} (characteristic strength of concrete). When examining the stress-strain behavior in a tension situation, it was considered to be linear until reaching the material's strength. Beyond that point, an exponential decrease was incorporated into the stress-strain curve. Plastic deformation values were employed and the parameter "d_t," which signifies the effects that diminish the elasticity of concrete in tension, was set to zero. For the CDP model in Abaqus, additional parameters were necessary. The assumptions for the dilation angle (representing the expansion angle in the p-q plane), eccentricity (the ratio of tensile strength to compressive strength of concrete), and fbo/fco ratio (the ratio of compressive strengths of concrete under two-directional and unidirectional loading) were 38 degrees, 0.1, and 1.16, respectively. These parameter choices were based on a prior experimental and numerical study, in which numerical simulations yielded results that closely approximated experimental findings [16].

2.1.2 Modelling the rebars of frame in abaqus

There are two kinds of steel material for longitudinal and confinement rebars in this study. The materials which had been used in the previous study [7] were used again. The steel used for longitudinal rebars was characterized by a yield strength of 491 MPa and a tensile strength of 553 MPa, and it was commercially known as B420C. For the stirr-ups, a steel with a yield strength of 277 MPa and a tensile strength of 387 MPa was used, and it was labeled as SAE steel. The steel's behavior was assumed to be consistent in both tension and compression. In Abaqus, stress-strain values must be implemented. In a previous study [16] steel's stress-strain curve was accepted as elastic-perfectly plastic for Abaqus analysis and the results were in good agreement with experiments. As indicated before "wire" elements were used for modelling. Using wire elements is not the only way to model rebars in Abaqus. As an alternative approach truss element or solid elements could be used however as indicated in a previous study [17] wire elements allows analyzing the structure when subjected to higher loads than the situation with solid elements. That's why wire element approach was chosen. In a study [18] where RC frames were analyzed experimentally and numerically, steel's stress-strain curve was accepted as linear plastic hardening, and the results gave good agreement with experiments. In this study, steel's stress-strain curve was considered by using the approach of previous study [7]

2.1.3 Modelling the high strength lightweight fiber reinforced concrete

The panels used for strenghtening the infill wall were considered to be produced with a high-strength, steel fiber-reinforced lightweight concrete which had been developed based on prior experimental research using expanded clay aggregates [19]. Expanded clay aggregates has a porous surface, thus, cement paste penetrates into the aggregate resulting a mechanical interlocking in the bond zone. This eliminates the weakness of coarse aggregate cement paste transition zone. A few different mixtures were used in theprevious study. 510kg/m^3 cement amount, 0.28 water/binder ratio, 2 percent steel fiber ratio, 50kg/m^3 silica fume was used in the production of high strength lighweight concrete with a compressive strength of 85.4 MPa (cubical). The concrete's density was obtained as 1966kg/m^3, its elasticity modulus was determined as 28000 MPa, and its Poisson's ratio was determined as 0.16 [19].

The parameters for the Concrete Damaged Plasticity (CDP) model were assumed to be the same as for ordinary concrete. These parameters included the dilation angle, eccentricity, and the fbo/fco ratio, which were set at 38 degrees, 0.1, and 1.16, respectively. To describe the stress-strain relationships of the concrete, the Hognestad model was used.

As previously mentioned, Abaqus necessitates the inclusion of plastic strains in the CDP model. Figure 5 illustrates the stress-strain graph for fiber reinforced lightweight concrete.

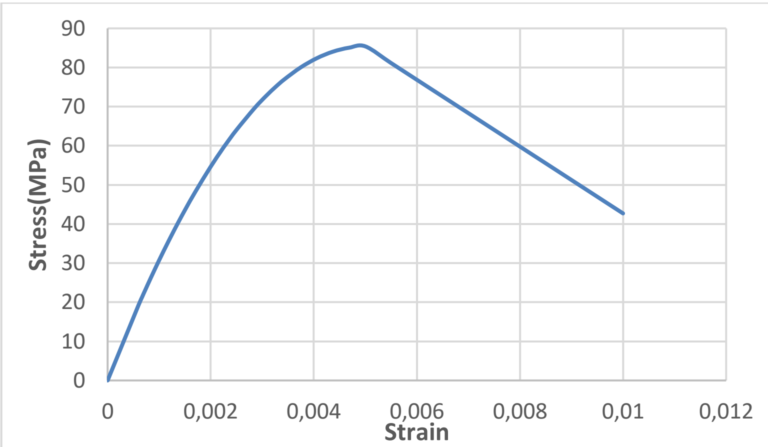


Figure 5. Stress-strain relationships of high strength lightweight concrete determined by Hognestad model [7]

In a study, an idealized and conceptual tensile stress-crack width (stress-strain) relationship curve of ultra-high performance fiber-reinforced concrete in a direct tension test was given [20]. After maximum load, the behavior was determined as an exponentially decreasing curve. The same method was followed for defining stress-strain relationships of concrete in tension here.

2.1.4 Modelling the polymer material between panels and infill wall

The polymer binder considered in the study is a polyurethane injection material produced by Sika company. It's a two component material. When applied, the two components were mixes, quickly the product hardens and became like a rubber material. That is why, here, rubber material modelling was considered to model the behavior. Rubber and rubber-like materials exhibit stress-strain curves that differ from those of metals. These curves include an ascending branch after some initial yielding. When modeling hyperelastic materials, stress-strain relationships are derived from the strain energy function, which quantifies the energy stored as the material deforms. Various models proposed in the literature define hyperelastic behavior by utilizing this strain energy. In a past study, where the material was used between CFRP and masonry bricks, Mooney-Rivlin theory was applied to model the hyperelastic behavior [21]. In that study the same material used in the previous study [7] was used again so C_{10} and C_{01} coefficients needed in Mooney-Rivlin Theory was considered as -0.05 and C10, as 0.47. The more detailed explanation can be found in the mentioned studies [7,21].

2.1.5 Modelling the interaction between binders and materials

The interaction between rubber like binder and materials can be defined in Abaqus in two ways. One of the approach is considering cohesive elements to model the binder. Another way is considering that the interaction between binders and materials has a surface-based cohesive behavior. This method allows for the modeling of connections with extremely thin interface thickness, employing the traction-separation constitutive model. Traction-separation laws are employed to define the behavior of joints in tension and shear failure modes. When the assembly is initially loaded, the joint exhibits linear elastic behavior. After reaching the peak traction value, the joint starts to have a plastic response.

There are some criterion to define the initiation of damage. Maximum nominal stress criterion was chosen in Abaqus. This criterion indicates that if the stress in tensile mode, or in shear mode of failure is equal to the strength of joint, damage starts. To characterize the behavior after joint failure, fracture energy of joint must be determined. The fracture energies of the joints were determined based on experimental results. In a preceding investigation [22], experiments involving masonry bricks were conducted to ascertain fracture energies in various failure modes. In a separate study using the results of experimental work [22] for joints containing mortar binder, the fracture energy was determined as 0.10 N/mm for mode 1 tension failure) and 0.183 N/mm for mode 2 shear failure [7]. In contrast, flexible joints constructed with a polymer binder exhibited fracture energies of 4.22 N/mm for mode 1 failure (tension) and 10.93 N/mm for mode 2 failure (shear) [23]. These values were employed in the numerical analysis. To address mixed-mode failure scenarios, the Benzeggagh-Kenane rule was implemented in Abaqus, guided by previous research [23] suggesting its effectiveness when critical fracture energies for second and third mode shear failures are identical. Additionally, an exponent of 2 for the Benzeggagh-Kenane rule was chosen in line with recommendations from the same study, specifically for brittle behavior. Following joint failure, a Mohr-Coulomb shear sliding model was established, incorporating a friction coefficient of 0.66. For a more detailed explanation, refer to the earlier study [7].

2.1.6 Modelling the hollow clay brick infill wall and mortar

Macro modelling approach was used to model the infill wall. For that purpose, the same wall modelling in the previous study [7] was used. In the macro modeling approach, the compressive and tensile strengths of the infill wall were determined in this study by using the following formulas based on Eurocodes as follows [7]:

$$f_{ck} = 0.4 \times 3.56^{0.75} \times 5^{0.25} = 1.57 MPa \quad (2)$$

$$f_{ctk} = 0.4 \times 0.9^{0.75} \times 0.257^{0.25} = 0.26 MPa \quad (3)$$

Here, f_{ck} indicates the characteristic compressive strength, 3.56 MPa is the strength of hollow brick and 5 MPa indicates the strength of mortar in compression. f_{ctk} indicates the tensile strength, 0.9MPa indicates the tensile strength of hollow brick, 0.257MPa indicates tensile strength of mortar [7]. The modulus of elasticity of hollow brick was 1111.49 MPa and the homogenised wall’s modulus of elasticity was 1012MPa [7].

The mortar used for constructing the wall was a mixture of cement, lime, and sand in a ratio of 1:2:9. In Abaqus, the Concrete Damaged Plasticity (CDP) model was employed to simulate the behavior of the mortar. In contrast to concrete modeling, the dilation angle for the mortar was determined to be 36.4 degrees. Experimentally, the mortar's compressive strength, Young's modulus, Poisson's ratio, and tensile strength were found to be 5MPa, 700MPa, 0.157, and 0.257MPa, respectively [7].

2.2. Sap2000 modelling

Sap2000 software is also based on finite element method. RC frame was considered to be constructed by using beam elements in the software. By using beam elements, frame can be analyzed in a linear analysis. However a nonlinear pushover analysis was needed to model the frame’s behavior in large lateral drifts. Pushover analysis is a nonlinear-static loading procedure in which an incremental lateral displacement loading is applied. That way it’s possible to know the lateral capacity curve of the structure. After some amount of increments of loading, frame loses its linear-elastic behavior and starts to yield. But it’s widely known that most of the nonlinear behavior happens in column-beam ends in the RC structures and in other regions of beams and columns, deformations still stay in linear elastic range. That's why rather than modelling the whole frame's elements nonlinearly, defining plastic hinges in column-beam end zones reduces the cost of computation. To model the plastic hinge, the moment curvature or moment-rotation graph of RC elements' cross-sections must be determined. Of course due to the existence of axial forces, moment capacity of column cross section will be dependent on axial force level. The axial force-moment interaction diagram of column must be taken into account also. If it's thought that second order effects do change the behavior, P-delta effects must be taken into account in the software.

In Sap2000 software, to model the plastic hinges, using lumped plastic hinges is one way , but in that study, fiber plastic hinge modelling is chosen. In fiber modelling approach, first the RC cross section is divided into fiber elements. Later by integrating the behavior of every fiber element , the moment carrying capacity and moment curvature relationship is determined. Then the curvature can be divided by the length of plastic hinge and rotation values can be found if it's needed. In that study, the element cross section was divided by 15x15 grids to form fiber elements. In the cross-section, the stress-strain relationships of concrete must be different in the confined region because the lateral pressure due to the existence of stir-ups(confinement rebars) changes the behavior of concrete. For that purpose, Mander model which is available in Sap2000 was used. Mander model can be seen in Figure 6 [24].

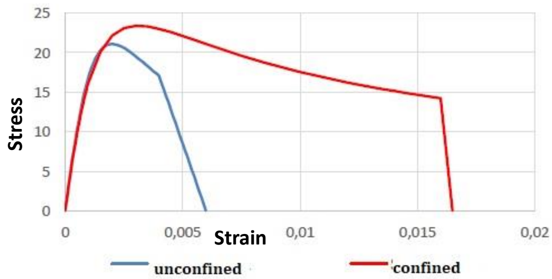


Figure 6. Mander model for defining stress-strain relations of concrete [24]

In Figure 6 [24], by the increase of displacement between confinement rebars, the model approaches to unconfined one. In this study, the distance between confinement rebars in plastic hinge regions were 100mm. In fiber modelling, for the concrete outside of confinements is assumed as unconfined concrete. The rebars are modelled by considering the strain hardening of steel as explained in section 2.1.2. For beam cross-section the plastic moment capacity was calculated as 76.85kNm. For column, 3D column interaction surface is obtained by the software. In the 3D curve, for any angle, Force-moment values can be seen. For instance for P-M3 surface of curve the following values in Table 1 are obtained.

Table 1. Interaction surface of column (P-M3)

Ax. Force KN	Moment(KNm)
-3806	0
-2490	129.47
-2168	150.2437
-1817	164.59
-1431	173.47
-996.43	179.18
-674.92	163.27
-306.27	140.35
-5.83	108.53
458.42	49.37
789.52	0

At the ends of members P-M2-M3 plastic hinges which had been derived from fiber modelling of cross section were assigned. To determine the place where plastic hinges were formed, the points calculated by 0.9 of member length and 0.1 of member length are used. Hinge length was assumed as 0.1 of the member length.

The frames which contains traditional infill wall and infill wall with strengthened panels were modelled by using equivalent compressive strut approach. In literature, many formulas were proposed to model the compressive strut. Some of the proposed formulas were the foundation of methods in seismic codes. In Eurocode 8 [25] for irregularity analysis it's said infills must exist in modelling and it's said that infill walls must be taken into account if they contribute to the lateral rigidity of structures however modelling approach is not explained in details. But in FEMA-356 [26] it's said that effect of infill walls must be taken into account with a FEM (finite element method) analysis or it must be modelled by using equivalent strut approach and the formulation is given. The formulation in FEMA-356 is based on the study of Mainstone [27]. But the method in FEMA-356 does not consider the level of vertical load. Also, in TSC 2018 [28], for the walls which are strengthened by using a special kind of reinforced mortar, equivalent strut approach is proposed. However, again the level of vertical loading is not taken into account. For this reason in a past study [29] a method considering the vertical load level was proposed. In the proposed method, the initial rigidity of actual system is considered to be equal to the system with equivalent strut by using the following formulation [29]:

$$\mathrm{Di}= \frac{k_d \cos^2\theta}{1+\frac{k_d}{k_c}+\sin^2\theta+0.25\frac{k_d}{k_b}\cos^2\theta} + 24\frac{E_f I_c}{h'^3}\left(1-1{,}5\left(3\frac{I_b}{I_c}\frac{h'}{l'}+2\right)^{-1}\right) \tag{4}$$

Here, Di represents the initial rigidity, while k_d, k_c, k_b denote the axial stiffness of the diagonal, column, and beam respectively. The variables h' and l' refer to the height and length of frame considering the midpoints of columns and beams in the calculations. The Θ angle is the angle of compressive strut inside the frame. Also k_d, k_c, k_b can be considered by following the approach outlined in the equation 5 [29]:

$$k_d = \frac{E_d t w}{d},\ k_c = \frac{E_f A_c}{h'},\ k_b = \frac{E_f A_b}{l'} \tag{5}$$

In equation 5, E_f represents the frame's modulus of elasticity, A_c is the cross-sectional area of the column, and A_b denotes the cross-sectional area of the beam, t indicates the thickness of infill wall and w indicates the width of infill wall. In a previous study [30] where equivalent struts were modelled inside RC frame, the width of equivalent struts were calculated by using different formulations proposed in literature and codes and it was seen that, by using the proposed approach explained in equations 4-5, the width of equivalent strut was determined 1.64 times of the width determined by Eurocode 8 approach and 2.23 times of the width determined by Mainstone's approach. In the same study [30], equivalent struts were modelled by assigning nonlinear hinges in the middle of struts. The modelling of hinge was described by Figure 7 [30]. In Figure 7, the ultimate force of nonlinear strut was calculated by multiplying the strength and the area of equivalent strut whereas the elastic force was calculated by multiplying elastic stress which can be assumed around 0.7 of ultimate strength and the area of compressive strut.

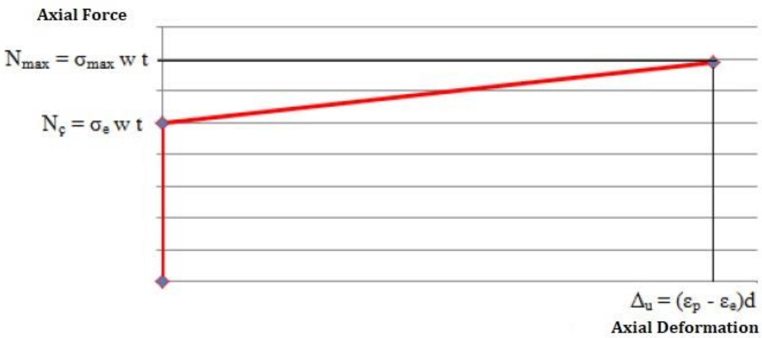


Figure 7. The axial force deformation relationship of nonlinear hinges in the middle of compressive struts [30]

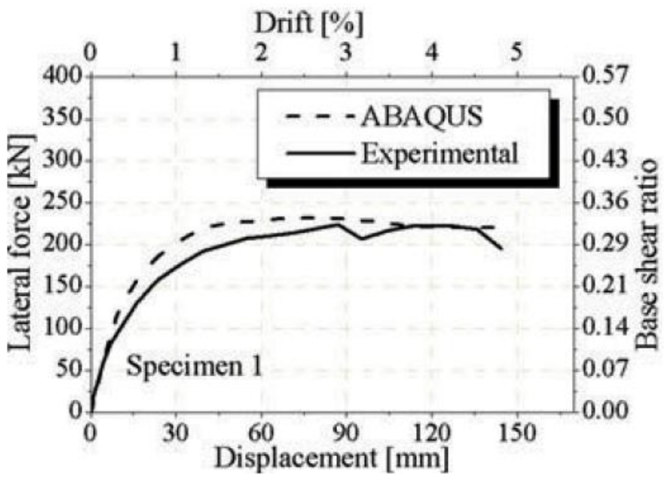
In the modelling of equivalent compressive struts, the equations 4-5 were used. For the modelling of nonlinear hinges in the middle of equivalent struts, the approach explained in Figure 7 was used. In equation 4, the initial rigidity of frames with infills were obtained by using the Abaqus results.

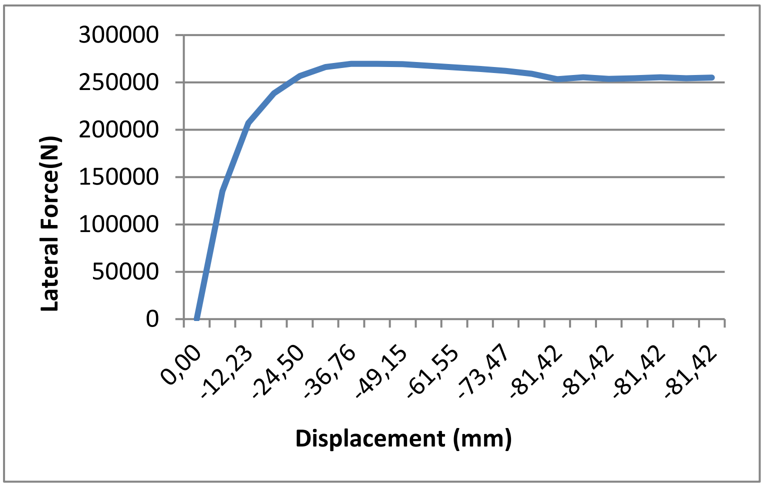
3. Results

3.1. Load displacement curves of frames modelled in abaqus

As explained before, bare frame was taken from an experimental and numerical study [12]. Also it's modelled in a previous study with equivalent Turkish concrete and steel materials in an another study [13]. The comparison and validation of Abaqus results with the materials used here were done in that study [13]. Here the Abaqus modelling of bare frame were performed by using the same approach. The results can be seen in Figure 8. If Figure 8 is observed it's seen that the frame's behavior is ductile. The maximum lateral load carried by bare frame is

269632.53N. The initial rigidity of bare frame can be calculated as 21886.06 N/mm from the graph by considering the first point where linear elastic behaviour finished. The discrepancy between the experiments and Abaqus graph can be related with differences in Chinese and Turkish materials. Also , analysis could be more precise if more elements were used.

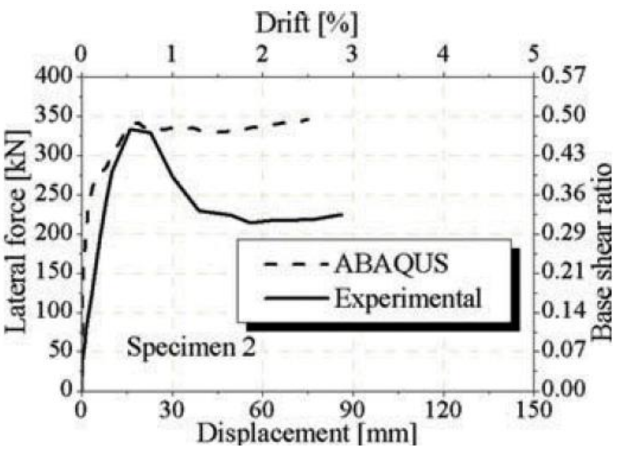


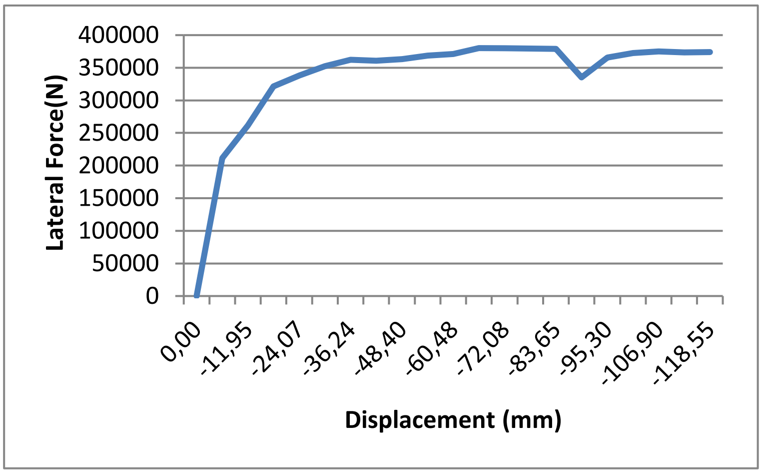


(a) **(b)**

Figure 8.a. The force displacement curve of bare frame in prior study [12] **b.** The force displacement curve of bare frame [13]

After the validation of Abaqus analysis by comparing with experiments, the frame with traditionally infilled wall can be analyzed. The frame is loaded around 120mm which indicates 4% lateral drift ratio to see the behavior in large drift. If the traditionally infilled frame is considered, the lateral load capacity is determined as 380109N which is similar with the lateral load capacity of frame with traditionally infilled wall in previous study [12] as seen in Figure 9. The initial rigidity of frame with traditionally infilled wall can be calculated as 35550.32N/m.





(a) **(b)**

Figure 9. a. The force displacement curve of traditionally infilled frame in prior study [12] **b.** The force displacement curve of traditionally infilled frame

In the third kind of frame, the infill wall was strengthened by high strength lightweight concrete panels. As seen in Figure 10, the capacity of frame increased to 1052406.12N level. This is 2.76 times of traditionally infilled frame's capacity and 3.9 times of bare frame's capacity. The initial rigidity can be calculated as 88575.30N/mm. The initial rigidity is approximately 4 times of bare frame and 2.5 times of the frame with traditionally infilled wall.

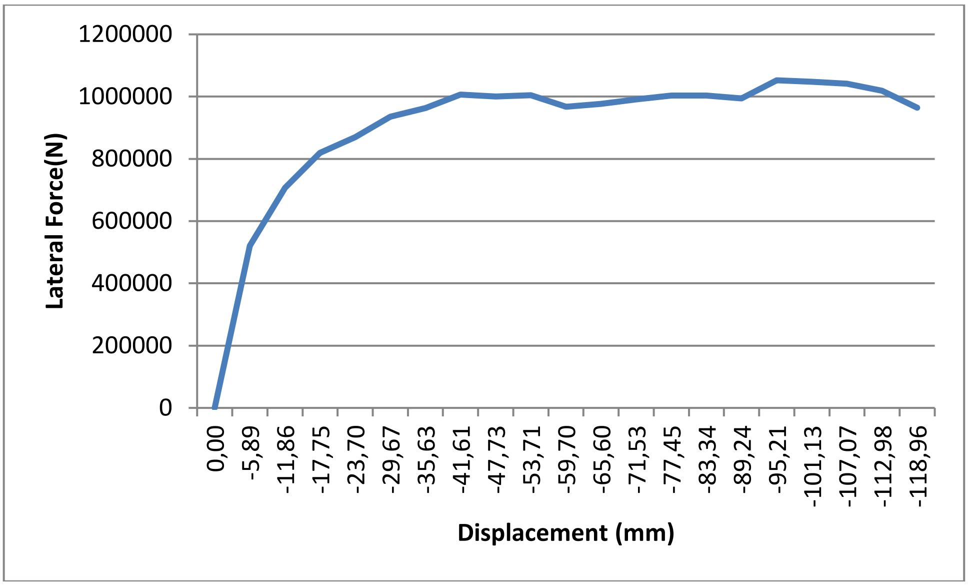


Figure 10. The force displacement curve of frame in which the infill wall was strengthened with panels

In Abaqus, dynamic explicit analysis was performed. In a previous study where a quasi-static loading of a historical wall was performed, the energy balance of the system in Abaqus was explained. Internal energy (EI) (Total strain energy of the material), energy absorbed by viscous damping (Ev), kinetic energy of deforming material (EKE), energy absorbed by frictional forces (EFD), work done by external forces (EW), and the total energy of the system can be written in the same equation as follows [14]:

$$EI + Ev + EKE + EFD + EW = Etotal \tag{6}$$

As stated earlier, in equation 6, “the kinetic energy of deforming material” / “total internal energy” ratio must be observed after analysis to check if the analysis can be accepted or not. In Abaqus after the analysis, the energy values for whole model during loading process can be seen. As a general rule, the kinetic energy should not exceed 5% -10% of the internal energy throughout most of the process [14]. As seen in Figure 11, throughout the loading, kinetic energy values are very low for the strengthened frame with lightweight concrete panels. Also, for other frames the ratio is checked and it’s concluded that the analysis can be accepted as a quasi-static loading. In other words, the whole frame models were not accelerated enough to be accepted as a dynamic loading. So, the results can be compared with a quasi-static experiment.

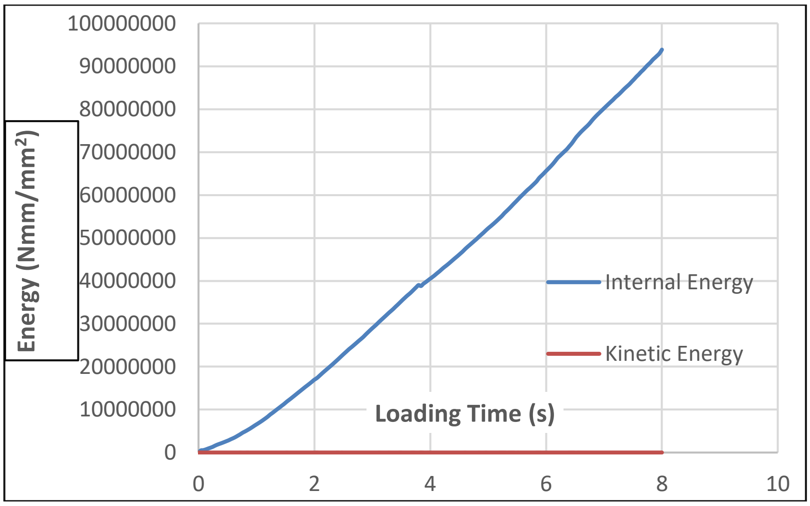


Figure 11. Internal energy and kinetic energy of frame strengthened with panels

3.1. Load displacement curves of frames modelled in sap 2000

The frames were modelled in Sap2000 according to the approach explained before in section 2.2. Bare frame is modelled by using plastic hinges. In the frames with traditional and strengthened infill walls, the properties of equivalent compressive strusses were determined by using equations 4-5. In equations 4-5, every quantity except the width of compressive strut is known. The initial rigidity of frames were derived from Abaqus analysis results. By using the equations 4-5, width of compressive strut of the frame with traditional infill wall is determined as 370mm. To determine the width of the compressive strut of the frame with strengthened infill, first an equaivalent axial modulus of elasticity for the strengthened infill was considered as 7344MPa according to equation 7. Later by using this value the width of equivalent strut was calculated as 335mm approximately according to equatons 4-5.

$$A_1E_1 + A_2E_2 = (A_1 + A_2) \text{ x } E_{eq} \quad (7)$$

In equation 7, A_1 and E_1 indicates the cross-sectional area of brick infill wall and modulus of elasticity of brick infill wall whereas A_2 and E_2 indicates the cross sectional area of high strength lightweight concrete panels. E_{eq} is the equivalent modulus of elasticity.

The nonlinear hinges in the middle of compressive struts with pinned ends must be determined for modelling. For that purpose, the method explained in Figure 7 was used. For traditionally infilled frame, the elastic force in hinge was determined as 76960 N, and the maximum force was determined as 113950N. Deformation value was calculated as 11.46mm.

For the frame with strengthened infill, again the nonlinear hinge was determined according to Figure 7. For that purpose , first, the ultimate axial strength of strengthened infill must be determined. To determine the ultimate axial strength of infills, equations 8-10 which are taken from a previous experimental study [6] were considered.

$$F_{infillwall} + F_{panels} = F_{strengthened\ wall} \quad (8)$$

$$F_{infillwall} = f_{infill}\ x\ ti\ x\ wi \quad (9)$$

$$F_{panel=} 7\ x\ fp^{0.25} x\ tp\ x\ wp \quad (10)$$

In equations 8-10, ti and tp indicates the thickness of infill wall and panel, wi and wp indicates the width of infill and panels. Thickness of infill wall is 200mm, width of infill is 335mm, the strength of infill wall is 1.54 MPa and the strength of panels are 85.4 MPa as explained previously. Equation 10 is derived by fitting a curve to the real panel strength of specimens in the previous experiments [6]. Here in the determination of panel strength a calibration was done and panel strength found by equation 10 was increased by a factor of 1.5. In an experimental study, some specimens can have strengths which are lower than expected. The ultimate strength was calculated as 744758N and elastic force in the nonlinear hinge was assumed as 70% of ultimate force. The compressive struts which have pinned ends were modelled linearly with nonlinear hinge in the middle. The load displacement curves and their comparison with abaqus results can be seen in Figure 12,13,14.

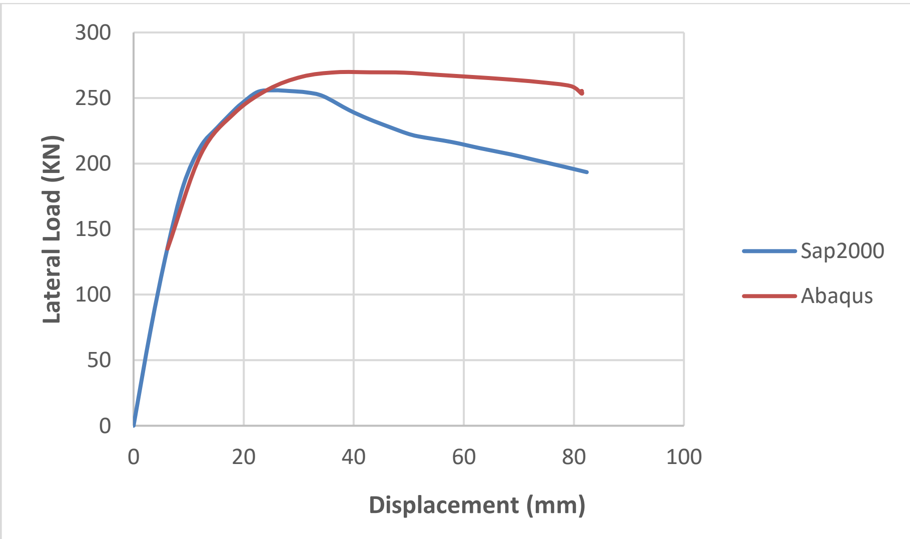


Figure 12. The force displacement curve comparison of bare frame

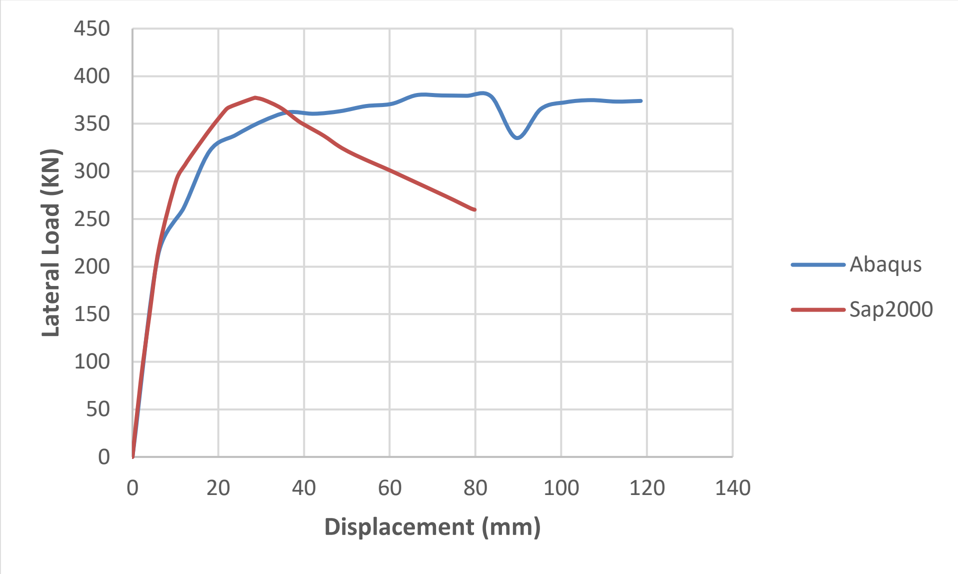


Figure 13. The force displacement curve comparison of frame with infill wall

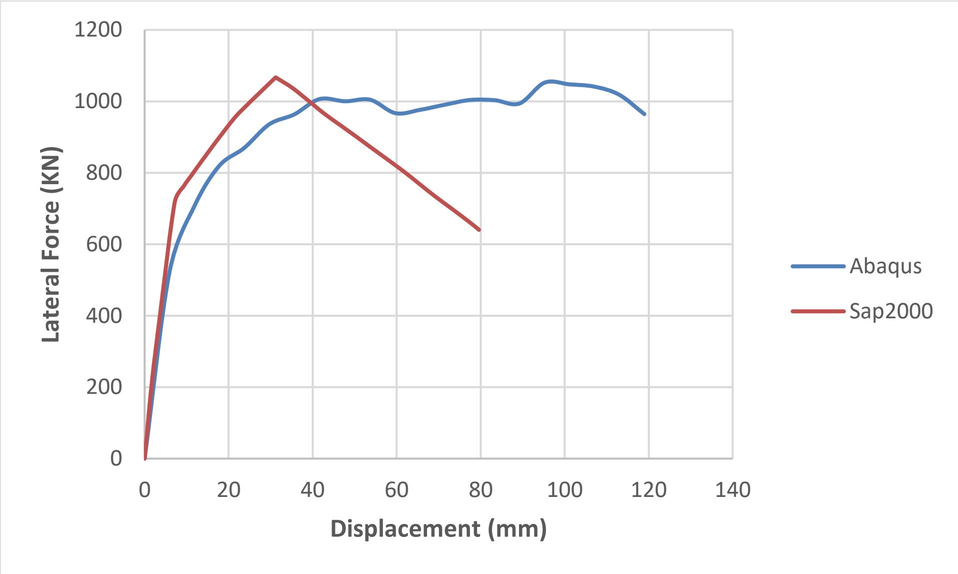


Figure 14. The force displacement curve comparison of frame with strengthened infill wall

3.2. Load displacement curves of multistory frames modelled in sap 2000

The Sap 2000 analysis was verified by comparing Abaqus results and considered to be representative for the real phenomenon. As explained in section 2.2, modelling of infill walls in multistory structures as equivalent compressive struts is adopted in seismic design codes. Also, in a previous study [31] where nonlinear dynamic analysis of infilled single and multistory RC frames were performed by using equivalent struts and shear spring, the displacements and base shears were compared with experiments and a good agreement was found both for single and multistory frames. In other study [6], the equivalent compressive strut approach for modelling was applied in the nonlinear static (pushover) analysis of 2 story frames and results showed good agreement with experiments. So, the validated modelling for single story frames can be used for multistory frames. Nonlinear static (pushover) analysis were performed on 3 and 5 story frames in order to analyze the effect of proposed strengthened method. In multistory frames, rebars, member dimensions, bay length and story height were taken as same with single story frames. Because the single story frame in the previous experimental study [12] was a representative of RC frames in China. In the multistory bare frame modelling, dead loads excluding the selfweight of RC frame members were considered as 30KN/m and live loads were considered as 20KN/m. In the frame with traditionally infilled wall, wall weight on the beam was assumed as 10KN/m. In the frame with strengthened infill, wall weight was taken as 15KN/m. On the frames, first, a vertical pushover analysis were taken into account by using G+nQ combination as explained in TSC-2018 [28], later incremental nonlinear static loading starting from the final results of G+nQ loading was applied n number was assumed to be 0.3.

The same nonlinear hinge approaches which were used to model single story frames were used in the modelling. For 3 story specimens in bare frames no wall is considered. The frames with traditional infill and strengthened infill were modelled by assuming walls in every frame in every story. In Figure 14 and in Figure 15, the pushover curves can be seen. The frame without any walls carried 392.15KN load maximum load at 84mm displacement, the frame with traditional infill wall carried 711.08KN maximum load, the frame with strengthened infill carried 2105.18KN maximum load.

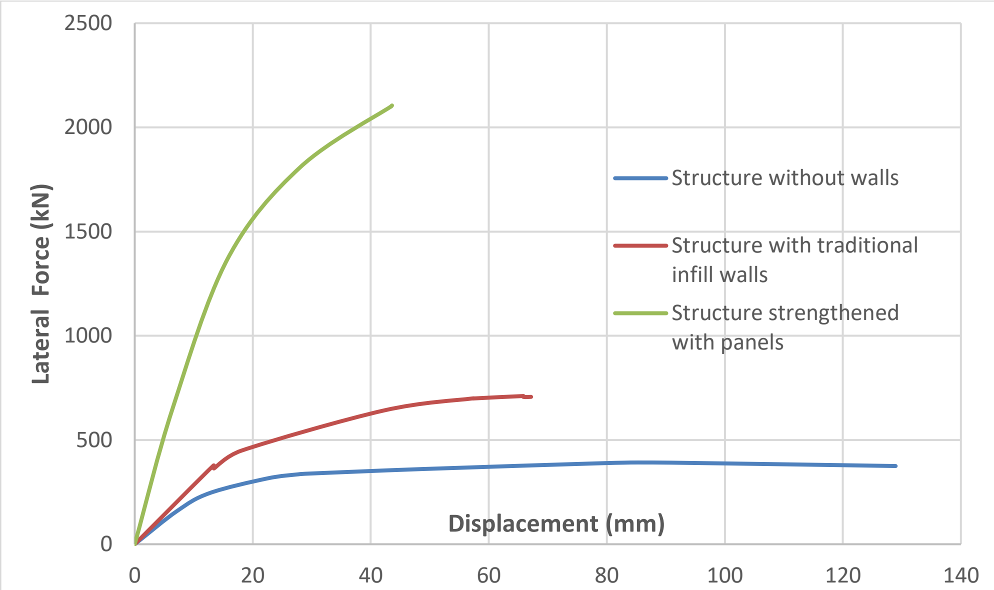


Figure 14. The force displacement curves of 3 story frames

If the pushover curves of 3 story frames were observed, in the 3 story frames, traditional infill walls increased the lateral load capacity by 1.81 times when compared with frame without any infill wall. Strengthening infill walls with panels increased the capacity by 3 times when compared with the frame which contains traditional infill walls. However, despite the increase in stiffness and strength, the proposed strengthening method with panels reduced the ductility of structure if the behavior was compared with bare frame without any infill walls.

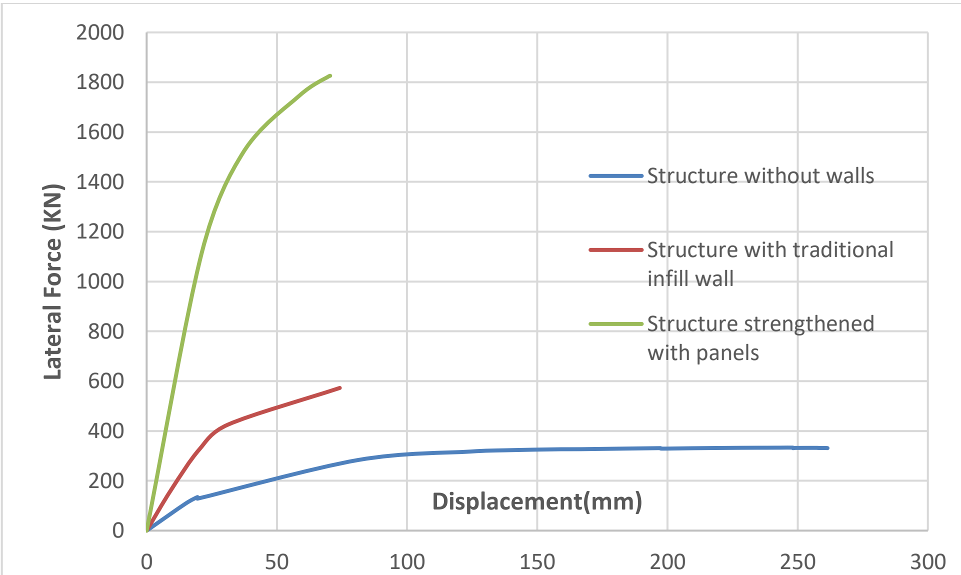


Figure 15. The force displacement curves of 5 story frames

If the pushover curves of 5 story frames were observed, the existence of traditional infill wall increased the lateral load capacity from 332.75KN to 572.75KN . This indicates an increase by a factor of 1.72. The panel strengthening affected the frames and maximum load carrying capacity increased from 572.75 KN to 1825.81KN. This indicates an increase by a factor of 3.18 if the frame with traditional walls and frame with strengthened walls are compared. However, despite the increase in stiffness and strength, the proposed strengthening method with panels reduced the ductility of structure if the behavior was compared with bare frame without any infill walls.

4. Discussion and Conclusion

In this study, the method proposed in previous studies [6,7] was investigated by using full scale frames in Abaqus. Later a validation was performed by using Sap2000 software. With the highlight of these analysis, multistory frames were analyzed in pushover analysis in Sap2000.

In single story full scale frames, it was seen that bare frame carried a max. load of 269632.53N. Traditionally infilled frame carried 380109N which is similar with the lateral load capacity of frame with traditionally infilled wall in previous study [12]. Traditional infill wall increased the load capacity of frame by 40%. The proposed strengthening method for the infill wall increased the load carrying capacity to 88375.30N level. This is 2.76 times of traditionally infilled frame's capacity and 3.9 times of bare frame's capacity. In a prior investigation [7], the application of the proposed method to scaled reinforced concrete (RC) frames, using 7.5mm-sized panels and a polymer binder between the panels and infill wall, resulted in an approximately 2 times increase in lateral load capacity compared to frames with traditional infill. In this study, the retrofitted frame supported 2.76 times more load than its traditionally infilled counterpart. The consistency and even improvement in results suggest that differences may be attributed to variations in concrete strength between the two studies and the influence of the scaling process.

In a previous numerical and experimental study [3], retrofitting frames and infill walls with the proposed method using Carbon Fiber Reinforced Polymer (CFRP) material increased the load capacity of RC frames by a factor of 2 to 3 when compared to non-retrofitted traditionally infilled specimens. Thus, the effectiveness of the strengthening method with high-strength lightweight panels and polymer is comparable to retrofitting with CFRP. In previous study [6], precast panels were employed for strengthening, leading to an increase in lateral load capacity from 65.5-86.6 KN to 148.9-254.7 KN, marking a 3.88-fold improvement in the best case. Although epoxy binder was used in that study, a higher load increase could be anticipated, as reported in the prior study [7]. However, the use of a polymer binder prevents panel detachment from the wall during large drifts and provides a ductile response. The initial rigidity of the bare frame is calculated at 21,886.06 N/mm, while the frame with a traditionally infilled wall has an initial rigidity of 35,550.32 N/m, indicating a 1.62 times increase, as expected. The well-known influence of infill walls on a structure's rigidity and period is evident. The initial rigidity of the retrofitted frame is calculated at 88,575.30 N/mm, approximately four times that of the bare frame and 2.5 times that of the frame with a traditionally infilled wall. This promising result suggests the effectiveness of the proposed method in strengthening structures with insufficient lateral rigidity.

After the validation of single story frame results in Sap2000, it's seen that the modelling of traditionally infilled frame in Sap2000 software is more consistent with real experiments in previous study [12] after 0.01 lateral drift ratio. Validation of Sap2000 results with Abaqus highlighted the way for modelling of multistory frames by using equivalent compressive struts. If the pushover curves of 3 story frames were observed, in the 3 story frames, traditional infill walls increased the lateral load capacity by 1.81 times when compared with frame without any infill wall. Strengthening infill walls with panels increased the capacity by 3 times when compared with the frame which contains traditional infill walls. If the pushover curves of 5 story frames were observed, the existence of traditional infill wall increased the lateral load capacity from 332.75KN to 572.75KN . This indicates an increase by a factor of 1.72. The panel strengthening affected the frames and maximum load carrying capacity increased from 572.75 KN to 1825.81KN. This indicates an increase by a factor of 3.18 if frame with traditional walls and frame with strengthened walls are compared.

In a recent study [14], high strength lightweight concrete wall and metallic elements were porposed to increase the stiffness, ductility, lateral load capacity of structures. The wall was assumed to have interaction with frame also. It was seen that; in the case when wall had interaction with frame, adding high strength lightweight wall and metallic elements to bare frame increased the lateral load capactiy of bare frame by 112%. However the proposed method in this study increased the lateral load capacity by 3.9 times when compared with bare frame.

The pushover curves obtained here can be compared with a previous study [32] where frames were modelled in a detailed finite element analysis and later pushover analysis were conducted to multistory frames by using a more practical software using super element approach. In the previous study, adding unreinforced masonry infill walls to multistory structures increased the lateral load capacity of structure around 4 times when compared with bare frame, but ductility decreased. In this study the behavior of multistory frames showed a similar behavior. The proposed method here increased the lateral load capacity by 3.9 times. So the pushover analysis results are consistent with previous studies.

The proposed method can be applied to strengthen the buildings which has insufficient lateral stiffness and strength. Traditionally, addition of RC walls to the structure was used in practice to enhance the stiffness and strength. But addition of RC walls to structures requires labor and time. Residents must leave the structure during the constructional work. In contrast, the proposed method here aims not to force the residents to be out of the building during the application process. Panel sizes are selected such as that the passage of panels through doors of buildings is possible and practical retrofit can be applied to walls. The polymeric binder between wall and panels is a two-component binder which gets hardened immediately after application. In this study during the in plane loading of RC frames the infill wall's central axis was thought to be same with the central axis of columns and infill wall had interaction with inner surfaces of frame elements as seen in Figure 1. In real applications eccentricity can be observed, thus it's advised here to extend the investigation in future studies. The strengthened infill walls are modelled as equivalent compressive struts here, however they can be modelled as equivalent columns in other investigations.

As a conclusion, the practical method used here is promosing for increasing the lateral load capacity and stiffness of existing structures and the proposed modelling approach performed with Sap 2000 will highlight the way to model the behavior for practicing engineers.

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