Strengthening of Brick-Infilled RC Frames with CFRP

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ABSTRACT

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Intensive studies regarding the rehabilitation of reinforced concrete structures by introducing reinforced concrete infill walls were carried out in the past. Many structures were also repaired using this technique after the recent earthquakes. However, the feasibility of this method is questionable as far as the rehabilitation of a large number of structures is concerned. This procedure requires evacuation of the entire buildings during the rehabilitation. In general it is not feasible to evacuate a building in use for a few months. Thus, a faster and easier method, which would not interrupt the use of the building, should be developed to strengthen the large number of buildings which do not possess seismic safety. Externally bonded fiber reinforced polymers (FRP) might be the solution of this problem owing to their light weight, high strength and ease of application. Of the available types of FRPs (carbon, glass, and aramid), carbon fiber reinforced polymers (CFRP) seems to be more feasible due to their higher strength.

In this study, retrofitting of undamaged reinforced concrete frames using CFRP is discussed in detail. The main objective of the experimental program is to reinforce the hallow clay tile infill walls, which are known to contribute to the seismic performance of the reinforced concrete structures significantly.

The scope of the study includes testing of seven one bay, two story, 1/3 scale specimens were constructed and tested under reversed cyclic loading. The specimens were constructed with the most common deficiencies observed in practice.

The test results were evaluated in terms of strength, stiffness, interstory drift, and energy dissipation capacity characteristics.

A model for composite material was derived using the test results. This model was used to develop design criteria for strengthening of structures using CFRP.

Keywords: Seismic strengthening, CFRP, brick infill, reinforced concrete frames, reversed cyclic loading, seismic response

ÖZ

Tuğla Dolgu Duvarlı Betonarme Çerçevelerin CFRP Kullanarak Güçlendirilmesi

Ülkemizdeki betonarme yapılar genellikle yeterli yanal dayanım ve rijitliğe sahip olmayan, donatı detayları deprem davranışı açısından yetersiz, beton dayanımları düşük çerçevelerden oluşmaktadır. Bunun yanında bu yapılarda yumuşak kat, kısa kolon, kuvvetli kiriş zayıf kolon gibi sistem yetersizliklerinin de bulunması, deprem güvenlikleri yeterli olmayan büyük bir yapı stokunu gündeme getirmektedir. Bu zayıflıklara sahip yapıların kuvvetli bir depremde sağlıklı bir davranış sergilemesini beklemek mümkün değildir. Bu nedenle, bir öncelik sırası belirlenerek, mevcut yapı stokunun deprem güvenliğinin artırılması gerekmektedir.

Güçlendirilmesi gereken bina sayısı gözönüne alındığında, bu binaların hepsinin deprem sonrası kullanılabilirliğini koruyacak şekilde güçlendirilmelerinin ekonomik olarak mümkün olmadığı görülmektedir. Diğer taraftan, can ve mal kayıplarının en aza indirilmesi için bu binaların büyük bir depremde göçmelerinin engellenmesi de gerekmektedir.

Kullanımda olan binaların güçlendirilmesini mümkün kılmak üzere, onarılacak binanın boşaltılmasını gerektirmeyen, hızlı ve binanın kullanımını aksatmadan uygulanabilen, ekonomik yöntemlerin geliştirilmesi gerekmektedir.

Yanal ötelenmeler belirli düzeyi geçmediği sürece, boşluklu tuğla duvarlarların betonarme çerçevelerin hem yanal rijitliğini hem de dayanımını önemli oranda artırdığı bilinmektedir. Ne var ki, yanal ötelenmeler belirli bir düzeyi aştığında söz konusu duvarlar ezilerek devre dışı kalmakta ve betonarme çerçevenin davranışına tüm deprem süresince katkıda bulunamamaktadır.

Bu çalışmada yapıdaki tuğla duvarların tümünün veya bir kısmının karbon lifli polimerler (CFRP) kullanılarak güçlendirmesi araştırılmıştır. Bu çalışma kapsamında, ülkemizde sıklıkla görülen zayıflıkları içeren 1/3 ölçekli, iki katlı, tek açıklıklı yedi adet çerçeve üretilmiş, çerçeve gözleri 1/3 ölçekli delikli tuğla duvar ile kapatılmış, ülkemizdeki genel uygulamaya uygun olarak duvarların her iki yüzü de sıvanmıştır. Bu şekilde üretilen deney elemanları, daha sonra değişik CFRP örtü uygulamaları ile güçlendirildikten sonra denenmiştir.

Bu raporda yapılmış olan deneysel bir araştırmanın detayları sunulmakta ve ulaşılan sonuçlar, mevcut yapıların deprem güvenliklerinin artırılması açısından irdelenmektedir.

Anahtar Kelimeler: Güçlendirme, CFRP, tuğla dolgulu duvar, betonarme çerçeve, tersinen tekrarlanan yük, deprem davranışı

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1. INTRODUCTION

1.1 GENERAL

In the last decade, six major earthquakes caused significant casualties and extensive structural damage in Turkey. Besides, most of the population and industry is under the threat of a possible major earthquake as they are located in the seismically active zones. Despite these facts, the structures in Turkey are far from possessing qualities that would ensure satisfactory seismic performance. Thus, these structures must be strengthened (pre–earthquake rehabilitation) to reduce losses that might occur in the future.

In order to have satisfactory seismic performance, a structure must possess adequate lateral stiffness, strength and ductility. The 1998 Turkish Earthquake Code [1] was prepared to ensure that all structures have adequate stiffness, strength and ductility, but most of the structures in Turkey do not have these properties. The main reason underlying this fact is the lack of inspection and supervision at the design and construction stages.

The most critical problem in reinforced concrete frames in Turkey is the detailing of the reinforcement. In general, the anchorage length of the bottom reinforcement of beams is insufficient. For columns, lap splices made at floor levels with inadequate length and inadequate confinement are the main problems. Due to the insufficient lap splice length provided, the yield strength of the longitudinal bars cannot be developed. Moreover, plastic hinges forming at the member ends cannot dissipate sufficient energy if they are not properly confined. These two factors lead to frames with inadequate ductility which cannot withstand major ground motions.

These reinforced concrete structures can be rehabilitated by strengthening the beams and columns. However, if the structure has the following deficiencies, member strengthening may not be economically feasible:

- There are too many members to be strengthened.
- The lateral rigidity of the structure is not adequate.
- There are important system deficiencies, like soft stories, weak stories, or short columns.

In such cases “system rehabilitation” would be more feasible. Very intensive studies regarding the rehabilitation of reinforced concrete structures by introducing reinforced concrete infill walls were carried out in the past. Also, many structures were repaired using this technique after the recent earthquakes. However, the feasibility of this method is questionable as far as the rehabilitation of a large number of structures is concerned. This procedure requires evacuation of the entire buildings during the rehabilitation. In general it is not feasible to evacuate a building for a few months. Thus, a faster and easier method, which would not interrupt the use of the building, should be developed to strengthen the large number of buildings which do not possess seismic safety. Externally bonded fiber reinforced polymers (FRP) might be the solution of this problem owing to their light weight, high strength and ease of application. Of the three types of FRPs (carbon, glass, and aramid), carbon fiber reinforced polymers (CFRP) seems to be more feasible due to their higher strength. In this study, retrofitting of existing buildings with brick infilled reinforced concrete frames using carbon fiber reinforced polymers will be discussed in detail.

1.2 PREVIOUS STUDIES

1.2.1 System Strengthening Studies at METU

System strengthening studies at METU Structural Mechanics Laboratory date back to the early seventies. In 1971 Ersoy and Uzsoy [2] reported the results of their experimental studies on reinforced concrete infilled frames. They tested nine one-story-one-bay frames under monotonic lateral loading. The lateral load capacity of the frame was increased approximately by 700 percent with the introduction of the infill. In addition, infills reduced the lateral displacements by 65 percent.

In 1990 Altın et al. [3] tested fourteen one-bay-two-storey specimens to investigate the behavior of reinforced concrete infilled frames. The frames, in which the infills had been introduced, were undamaged, detailed in accordance with the seismic code and were properly constructed. The main test variables were the pattern of infill reinforcement, the connection of the infill to the frame, effect of axial load and strength of the frame columns. At the end of his study Altın et al. reached the following conclusion:

- the infills, which were properly connected to frame members, increased both the strength and stiffness significantly.
- column strength and axial load on the columns improved the behavior and increased lateral load capacity.
- the most important variable that affects the behavior of reinforced concrete infilled frames was the type and details of the connection between the frame and the infill.

Marjani et al. [4] tested one bay, two story brick infilled reinforced concrete frames. As a result of this experimental study, they stated that brick infills increased the lateral load capacity by about 240 percent. Plastering the brick infills resulted in an additional 60 percent increase in strength. Plaster was also effective in increasing the ductility of the specimens since it delayed the cracking of the infills.

Sonuvar [5] tested five one-bay-two-story 1/3 scale, poorly designed, detailed and constructed reinforced concrete frames. The frames were heavily damaged prior to the introduction of the reinforced concrete infills. The following conclusions were reached:

- Strength and stiffness of the frames significantly increased as a result of the rehabilitation made by introducing reinforced concrete infills,
- the anchorage of dowels was influenced by the quality of concrete of the frame members,
- the amount of column reinforcement had a significant effect on the strength of the infilled frame,
- rehabilitation using reinforced concrete infills resulted in a significant increase in energy dissipation capacity.

Canbay et al. [6] tested a two-story, three-bay reinforced concrete frame. After the frame was significantly damaged, reinforced concrete infill was introduced to the middle bay. The conclusions drawn from his study can be summarized as follows:

- The stiffness of the infilled frame was fifteen times the stiffness of the corresponding bare frame.
- The strength of the frame was increased by 400 percent as a result of the introduction of the infill.

- Energy dissipation capacity was increased significantly.
- The reinforced concrete wall formed by the infill carried 90 percent of the lateral load.

1.2.2 Strengthening of Masonry Infills with FRP

In the last decade, fiber reinforced polymers (FRP) have become a popular material in the rehabilitation of reinforced concrete structures. However, FRPs are mostly used in member strengthening such as wrapping of columns to increase ductility or increasing the shear strength of beams. In addition, structures with unreinforced masonry walls had also been strengthened using FRPs. No work has been reported on rehabilitation of brick infilled reinforced concrete frames till now. Studies related to strengthening of wall bearing structures using FRPs will be briefly summarized in this section.

Priestley and Seible tested a full scale five story masonry building [7]. First, the building was damaged (original testing), then repaired by means of structural carbon overlays to the walls of the first two stories. At the end of this test program, the following observations were made by the authors:

- The load-displacement envelopes show that a single layer of carbon fabric overlay (t=1.25 mm) applied on each face of the walls with two layers in the toe regions, contributed significantly to doubling the inelastic deformation capacity.
- In the first phase, the building had been damaged to the inelastic stage. The specimen repaired by using FRP reached the base shear capacity of the undamaged specimen. It is also noted that there was a significant decrease in the initial stiffness.
- Measured shear deformations in the overlaid wall panels were reduced to half the shear deformations in the original five story building test.
- Very thin layers of composite material (one or two layers) can show significant seismic improvements for in - plane wall response.

Marshall, Sweeny and Trovillion [8] tested twenty wall panels and twenty double-wythe-brick wall panels by using four different FRP composite systems,

- to evaluate the capability of FRP composite systems to hold unreinforced masonry wall sections together once seismic damage has occurred in the mortar joints and through the masonry units themselves.
- to quantify the strengthening and/or pseudo ductility or overall system ductility increase to the walls with FRP composites adhered to them.

The conclusions drawn by the authors can be summarized as follows:

- Some increase in the pseudo ductility (area under the load - deflection curve) was observed in the walls with FRP compared to reference specimens. However, this observation was not consistent for all of the walls with the same type of overlay.
- These tests demonstrated the capability of FRP composite materials to hold a wall together once failure had occurred.

Roko, Boothby and Bakis tested twenty five masonry prisms to determine the failure modes of sheet bonded FRP applied to brick masonry [9]. They used two types of masonry (high and low porosity) and two types of CFRP reinforcement (low and high modulus). They reported that, all molded specimens with both low and high modulus had failed by shearing of the brick where FRP reinforcement had terminated, whereas extruded brick specimens failed by the debonding of FRP, thus had not yielded significant moment resistances. At the end of these tests authors concluded that;

- FRP, when applied in tapes, greatly increases the strength and ductility of masonry prisms subjected to out-of-plane bending.
- Brick type directly affects the bond performance and failure mode of a masonry prism. The porosity of the masonry is the direct cause of this variation.

Triantafillou tested twelve identical small masonry wall specimens (wallettes) [10]. Six of these specimens were tested in out-of-plane bending and the remaining ones in in-plane-bending. Four specimens in each group were reinforced with epoxy bonded unidirectional CFRP laminates (1mm thick and 50 mm wide) whereas the other two were used as unreinforced specimens. The author reached to the following conclusions at the end of these tests.

- Wallettes tested in in-plane bending failed prematurely through debonding of the CFRP laminates. In the case of specimens with more than one layer of reinforcement, peeling-off initiated at the bottom layer and progressed to the upper layers. The achievement of full in-plane flexural strength depends on proper anchorage of the laminate to the masonry.

1.3 OBJECT AND SCOPE OF THE STUDY

With developing technology, composite materials provide an alternative solution for seismic strengthening of structures. However, only a limited number of studies exist on their use in seismic strengthening of reinforced concrete buildings. Although the research in this field is scarce, the composite materials look very promising for rehabilitation of existing reinforced concrete buildings that are identified as seismically vulnerable.

This study is a part of a major project on the strengthening of existing reinforced concrete structures and it is funded by NATO (through SfP977231) and TUBITAK - The Scientific and Technical Research Council of Turkey (through İÇTAG I575). The main objective of this study was to increase the seismic performance of poorly designed and constructed reinforced concrete frames with minimum disturbance to the occupants. Collapse prevention during a major earthquake was established as the level of the aimed seismic safety. In the first phase of the project, the feasibility of strengthening using CFRP sheets and strips was investigated.

The main objective of this first phase was to develop a new strengthening technique to improve the seismic behavior of reinforced undamaged frames with hollow clay tile infills. This new technique would be based on strengthening of hollow clay tiles by using CFRP sheets and strips. The undamaged frames tested had the following deficiencies:

- The end zones of beams and columns were inadequately confined.
- The ends of the ties were bent 90°.
- No transverse reinforcement was used in joints.
- The bottom reinforcement of the beams did not have sufficient anchorage length.
- Concrete strength was low.

Test specimens in this study were one-bay, two-story frames similar to the ones which were tested at METU with reinforced concrete infills [3], [5]. However, in this study the infill was made of hollow clay tiles.

Using the test results, it was aimed to develop design criteria for strengthening of existing reinforced concrete buildings with CFRP sheets.

2. TEST SPECIMENS

2.1 GENERAL

In this experimental study a total of seven specimens were tested under reversed cyclic loading. All specimens consisted of one-bay-two-story reinforced concrete frames with hollow clay tile infills. Six of these specimens were strengthened by using CFRP sheets prior to testing. The remaining one specimen was not strengthened. This specimen served as a reference specimen.

2.2 TEST SPECIMENS

As stated above, test specimens were two-story, one-bay reinforced concrete frames with hollow clay tile infills which are commonly used in residential buildings in Turkey. The frames were designed and constructed with deficiencies commonly encountered in residential buildings. These deficiencies are discussed in Section 2.2.1.

2.2.1 Reinforced Concrete Frame with Brick Infill Walls

The reinforced concrete frame specimens consisted of two one-bay, two-story frames connected with a rigid foundation beam (Figure 2.1). The philosophy behind the design of this specimen was to prevent (or minimize) the rotation at the base of the columns. The specimens were tested in a horizontal position.

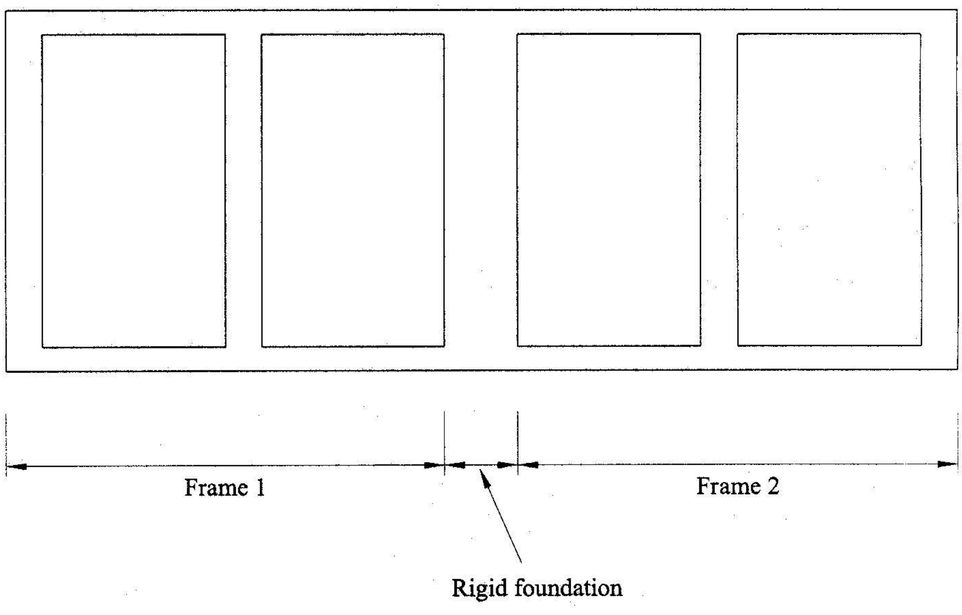


Figure 2.1 General view of the specimen

The specimens were designed and constructed with deficiencies commonly observed in existing reinforced concrete residential buildings. Therefore, they did not conform to the Turkish Earthquake Code [1].

In beams and columns ties were uniformly spaced as shown in Figure 2.2. No confinement was provided at member ends by decreasing the tie spacing. The ends of ties were not bent into the core as required by the seismic codes. The ends of ties had 90 degree bents and were overlapped at the corner. In addition, there were no ties at beam column joints.

Since lapped splices made in column longitudinal bars at floor levels are known to reduce the strength of infill frames [5], no splices were made at the base of first story columns at the foundation level. However longitudinal bars in columns of the second story had lapped splices at the base. The lap length was 300 mm.(about 38 bar diameters). It was intended to study the adverse effect of lapped splices in the later phases of the project.

The dimensions and the reinforcement of the test frame are shown in Figure 2.2. A photo of the reinforcement is given in Figure 2.3.

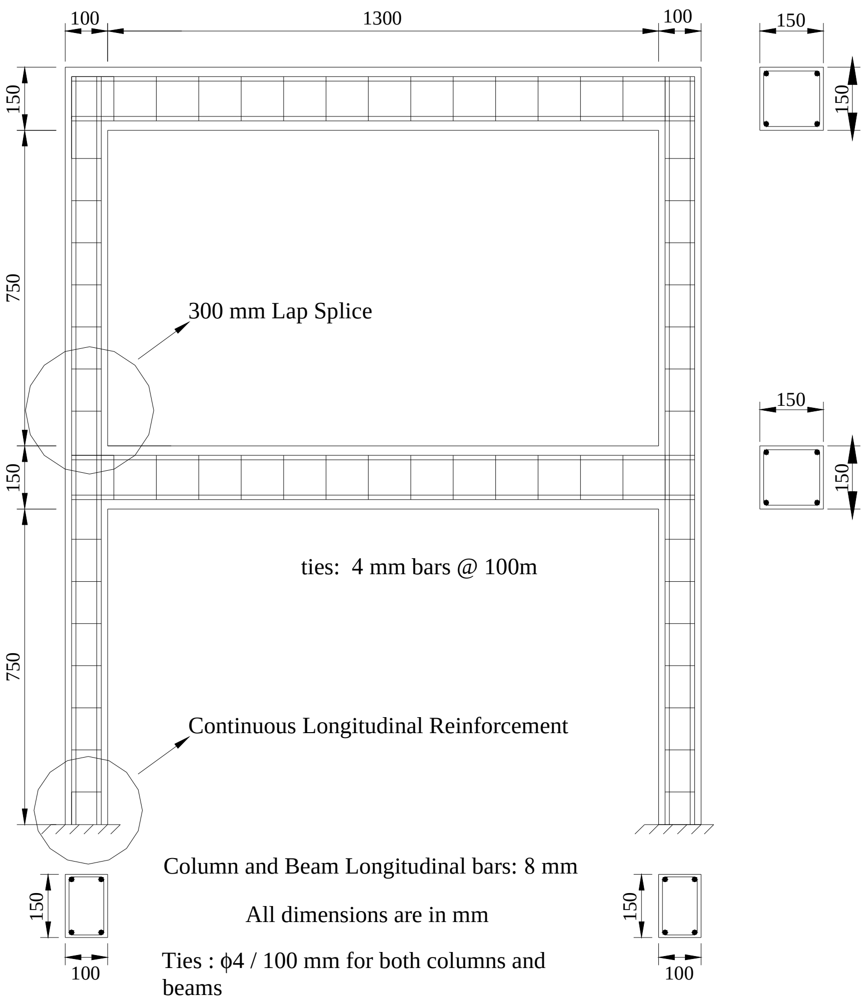


Figure 2.2 Reinforcement Details of the Test Specimens

One of the common problems in the residential buildings in Turkey is the anchorage of beam bottom bars. These bars usually extend into the end columns without adequate anchorage. This detail was duplicated in the beams of the test specimens as shown in Figure 2.4.

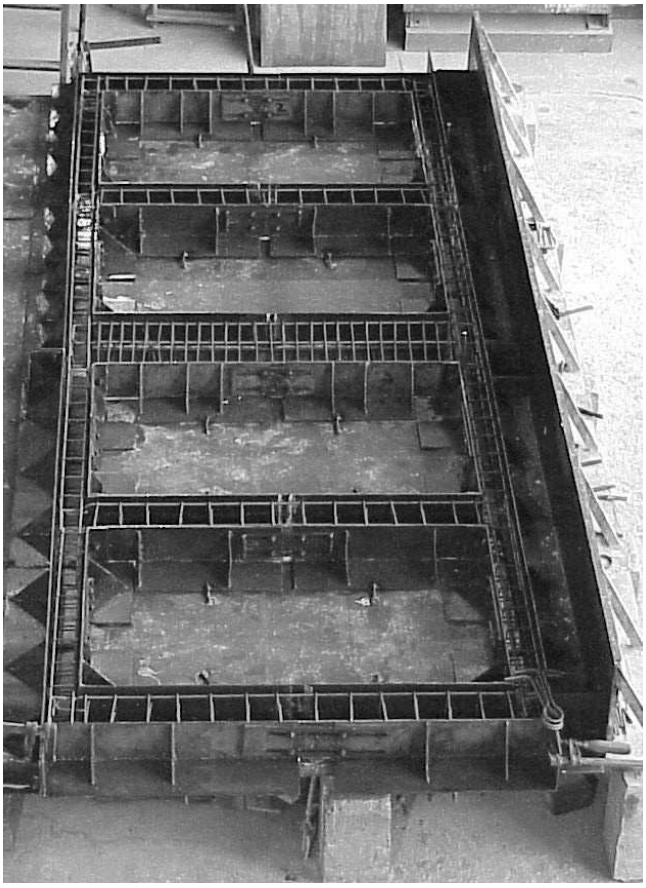


Figure 2.3 Reinforcement of a specimen in the formwork

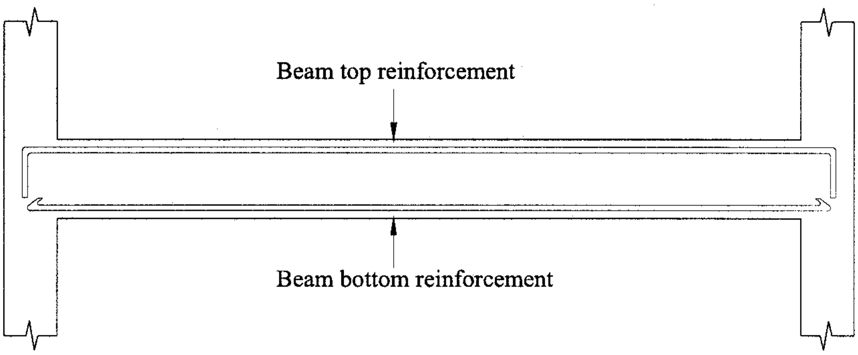


Figure 2.4 Detailing of Beams

The foundation beam of the frame was constructed monolithically. The foundation beam was heavily reinforced to prevent local failures. Moreover, the ties were bent 135° into the core to ensure the confinement (Figure 2.5). Also two intermediate bars were provided in the foundation beam as shown in Figure 2.5.

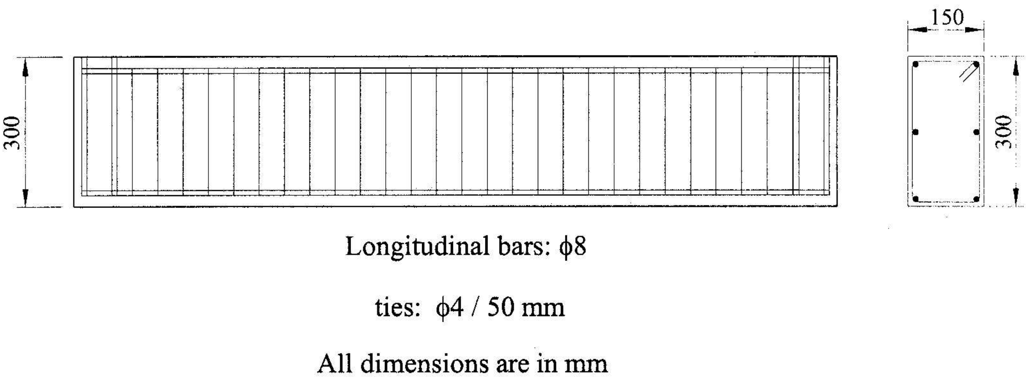


Figure 2.5 Reinforcement of Foundation Beam

Although extreme care was taken to ensure symmetric loading of the twin frames, it was not possible to fully eliminate the rotations at the base of the first story columns, especially when the specimen was loaded into the inelastic range. These rotations were measured and taken into account in the related calculations.

Hollow-clay tile was used as the infill material. Standard 18 x 18 x 8.5 hollow tiles were cut into 4 – 9 x 7.5 x 8.5 tiles in order to scale them to simulate the real tile (Figure 2.6)

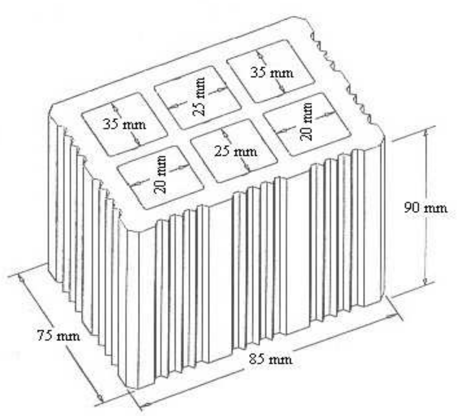


Figure 2.6 Hollow Clay Tiles

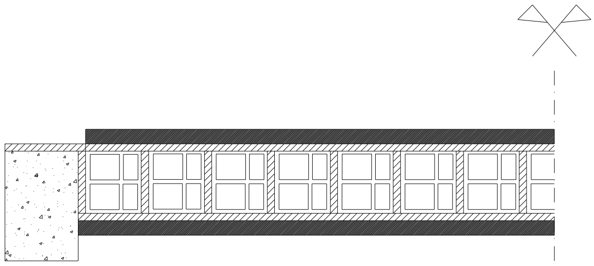
The infill was made by an ordinary construction worker. The mortar between the tiles was made by mixing sand, cement, lime and water. The tiles were laid when the specimen was placed in a vertical position. The hollow clay tile wall was plastered on both sides. The thickness of the plaster was about 10 mm.

2.2.2 Test Specimen 1 (SP-1)

The first one of the seven identical specimens was an unstrengthened specimen. This specimen was intended to serve as a reference specimen.

2.2.3 Test Specimen 2 (SP-2)

The second specimen was the first strengthened specimen tested within the scope of this study. Only the brick infill walls of SP-2 were strengthened using two orthogonal layers of CFRP covering the whole infill. No CFRP was applied on the reinforced concrete frame members and no connection was made between the CFRP sheets and the frame members. No anchor dowels were used to anchor the CFRP sheets to the wall. CFRP sheets were fixed to the wall using a special adhesive. CFRP detailing of SP-2 is presented in Figure 2.7.

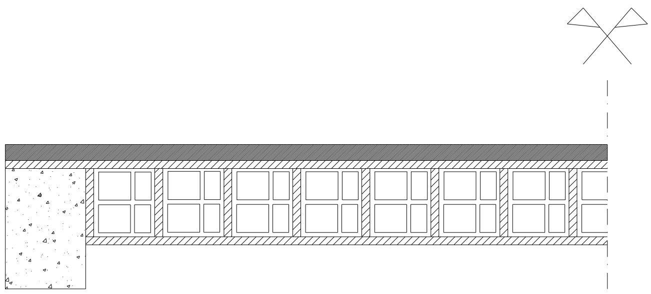


CFRP
Mortar & Plaster

Figure 2.7 CFRP Applied to Specimen SP-2

2.2.4 Test Specimen 3(SP-3)

The third specimen was strengthened by two orthogonal layers of CFRP applied on the exterior face of the infill. CFRP reinforcement was applied to the brick infill walls and was extended to the reinforced concrete frame members (Figure 2.8). During the test of the second specimen, CFRP layers had easily delaminated from the brick. To prevent such a premature failure, special anchor dowels were designed and used to anchor CFRP to reinforced concrete members. Those dowels were developed upon the suggestion of Dr. Murat Saatcioglu. The locations of the anchor dowels used in SP-3 are presented in Figure 2.9. No such anchors were used in the infill. The CFRP applied covered one face of the infill totally.



CFRP
Mortar & Plaster

Figure 2.8 CFRP Applied to Specimen SP-3

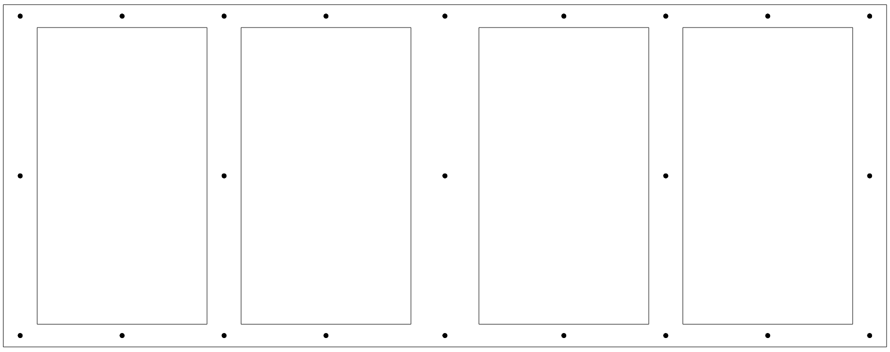


Figure 2.9 Location of Anchor Dowels in Specimen SP-3

2.2.5 Test Specimen 4 (SP-4)

Specimen SP-4 was strengthened by two orthogonal layers of CFRP applied on both faces of the infill. CFRP reinforcement was extended to the reinforced concrete members on the exterior face (Figure 2.10). Although, the anchor dowels used in SP-3 made a favorable contribution to the behavior, it was observed that the amount number? of the anchor dowels used was insufficient. Thus additional anchor dowels were used in SP-4, Figure 2.11. In addition, CFRP layers were also anchored to brick infills.

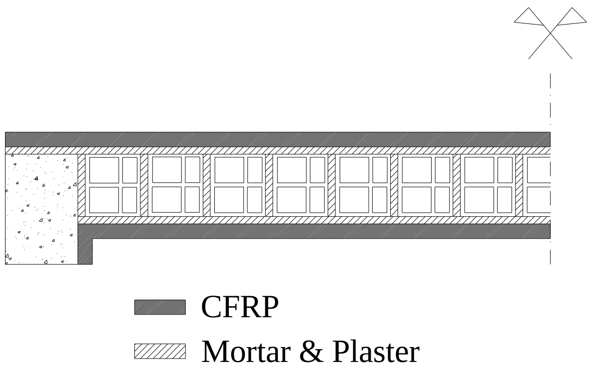


Figure 2.10 CFRP Applied to Specimen SP-4

Failure of test specimens SP-2 and SP-3 took place near the beam-column joint at the first story level. This showed that the lap-splices made at floor levels caused problems although the lap splice length was sufficient according to the Turkish Seismic Code. To overcome these problems, the lap-splice regions were confined using two layers of CFRP reinforcement in SP-4. The length of the confined zone was 400 mm, a little bit longer than the splice length.

2.2.6 Test Specimen 5 (SP-5)

The CFRP detail used in SP-4 resulted in a significant increase in strength and improved the behavior. However the strengthening made was far from being economical due to the amount of CFRP used, which covered both faces of the infill.

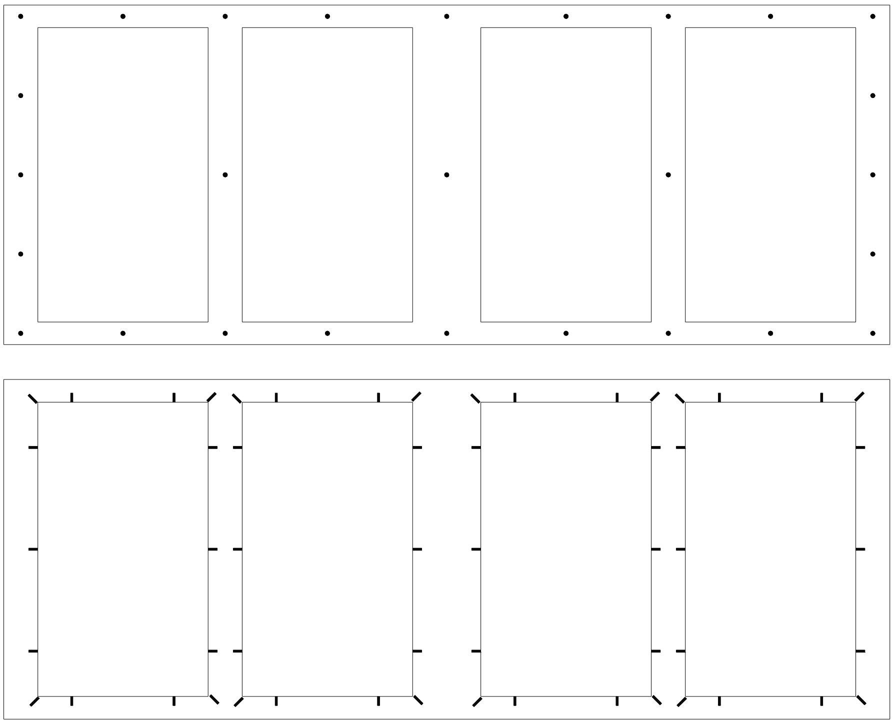
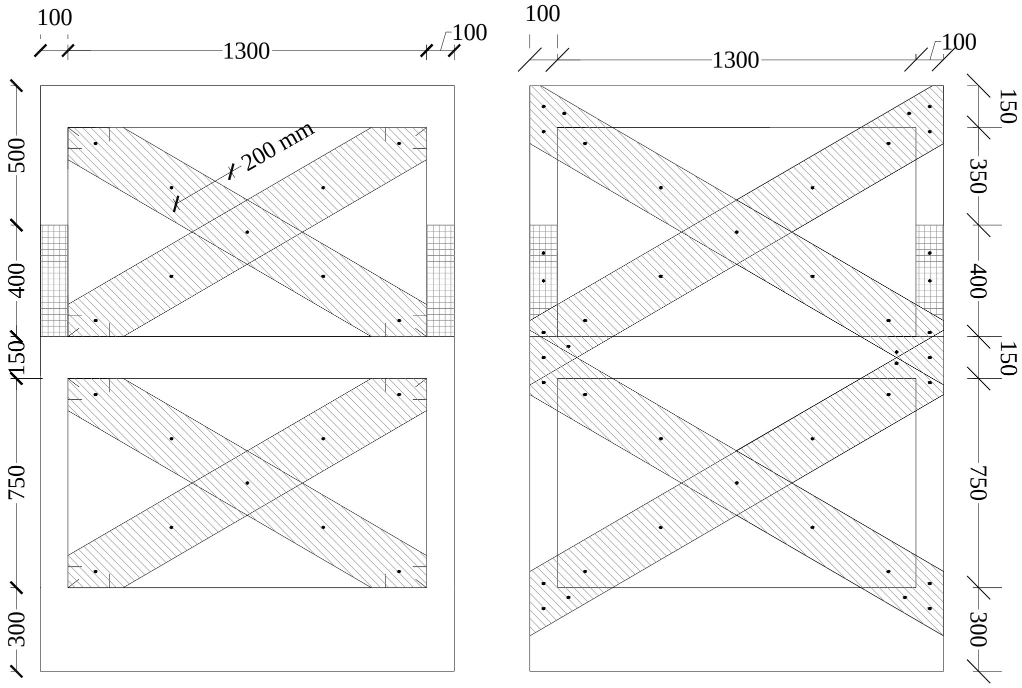


Figure 2.11 Location of Anchor Dowels in Specimen SP-4

CFRP layers were observed to work as cross struts during this test. To decrease the amount of CFRP used, in regions where CFRP did not seem to be effective were taken out. Furthermore, since CFRP is a unidirectional material with very high tensile strength, it was decided to use one layer of CFRP that would form a tensile strut (Figure 2.12 and Figure 2.13).

The width of each CFRP strap applied was 200 mm. On the exterior face of the specimen, CFRP extended along the reinforced concrete frame above the first story level. Lap-splice regions were confined using 2 layers of fibers in orthogonal directions in order to eliminate the problems that had occurred in the previous tests. No confinement was provided at the foundation level since there were no lap splices in this region. At the interior face, the corners of the straps were bent 90^0 and extended to the sides. In order to prevent the stress concentrations that might occur in the CFRP straps due to the sharp edges at the corners, these edges were rounded.

Previous tests made in this study have shown that one of the most important parameters affecting the improvement in both strength and behavior was the connection of the CFRP to the infill and to the concrete members. The anchorage system used in SP-4 proved to be effective. Thus, the same anchorage system was also used in this specimen with slight modifications. The number of anchor dowels was increased from five to nine on the infills. On the interior face, the length of the anchor dowels was increased from 50 mm to 60 mm. The properties of the anchor dowels used are summarized in Table 2.1 and are shown in Figure 2.14.



(a) Interior face (a) Exterior face

Dimensions are in mm

Figure 2.12 CFRP Arrangement and Location of the Anchor Dowels in Specimen SP-5

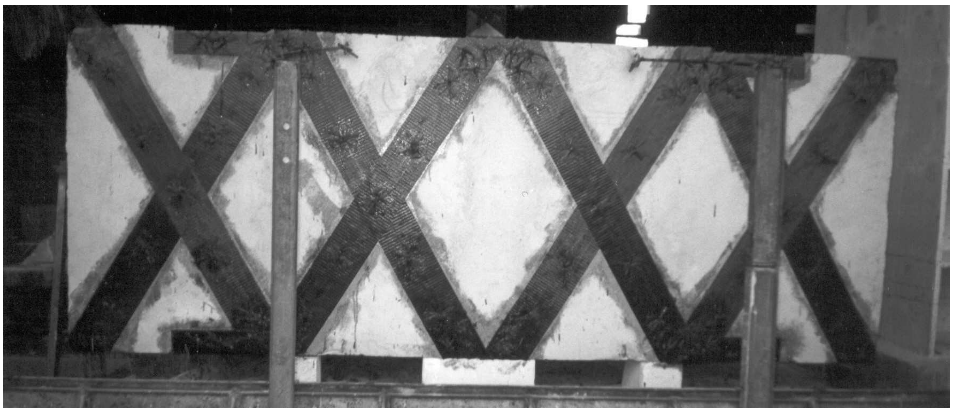


Figure 2.13 Specimen SP-5

Table 2.1 Properties of Anchor Dowels of Specimen SP-5

Anchor Type	Applied to	Depth in RC (mm)	Width of Strip (mm)	Diameter of Hole (mm)
A	Infill		50	10
B	Concrete	50	25	10
C	Concrete	60	25	10
D	Concrete	60	30	12

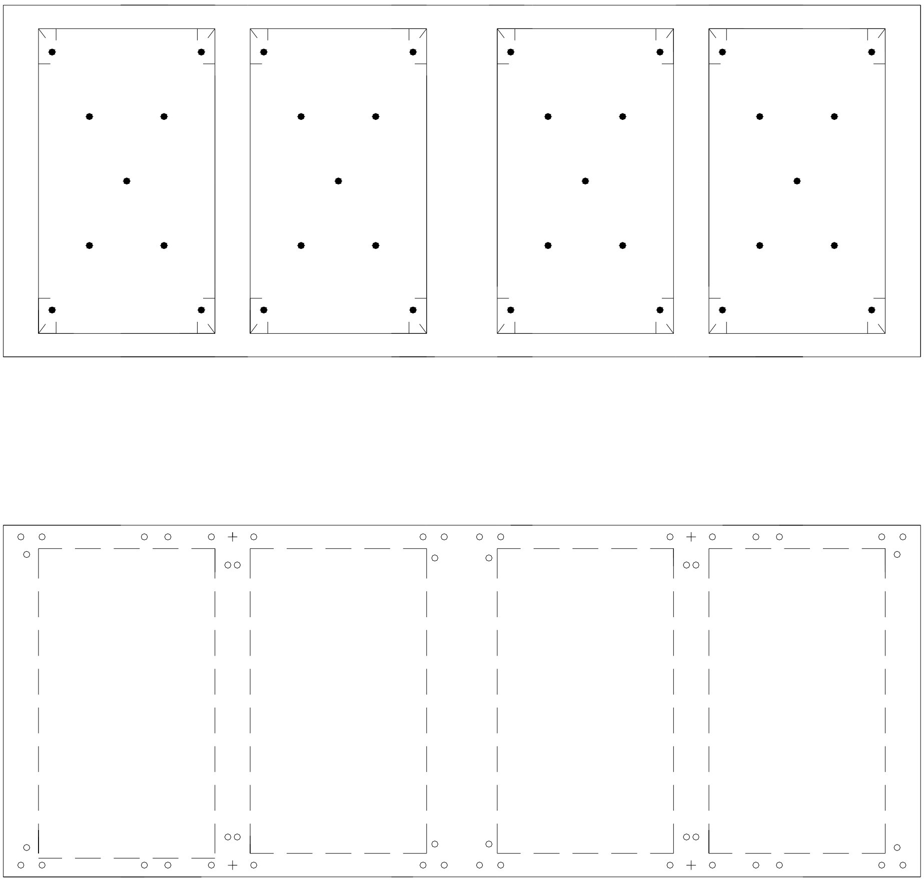


Figure 2.14 The Location of Anchor Dowels in Specimen SP-5

2.2.7 Test Specimen 6 (SP-6)

Strengthening done in Specimen SP-5 improved the strength and lateral stiffness significantly. During the test, it was observed that failure was initiated by debonding of the CFRP strap at the foundation level. In addition, yielding of reinforcement and crushing of concrete was observed at the base of the first storey columns. To prevent these local failures in specimen SP-6, the base of the first storey columns was confined with two CFRP layers placed in orthogonal directions. The length of the confinement zone was limited to the expected plastic hinge length (150 mm) (Figures 2.15 and 2.16).

Some of the anchor dowels at the foundation level of the SP-5 failed by the fracture of CFRP, indicating that the fibers used was insufficient. Thus, the width of strips of those anchor dowels was increased from 25 mm to 40 mm. Properties of the anchor dowels used in this specimen are given in Table 2.2 and are shown in Figure 2.17.

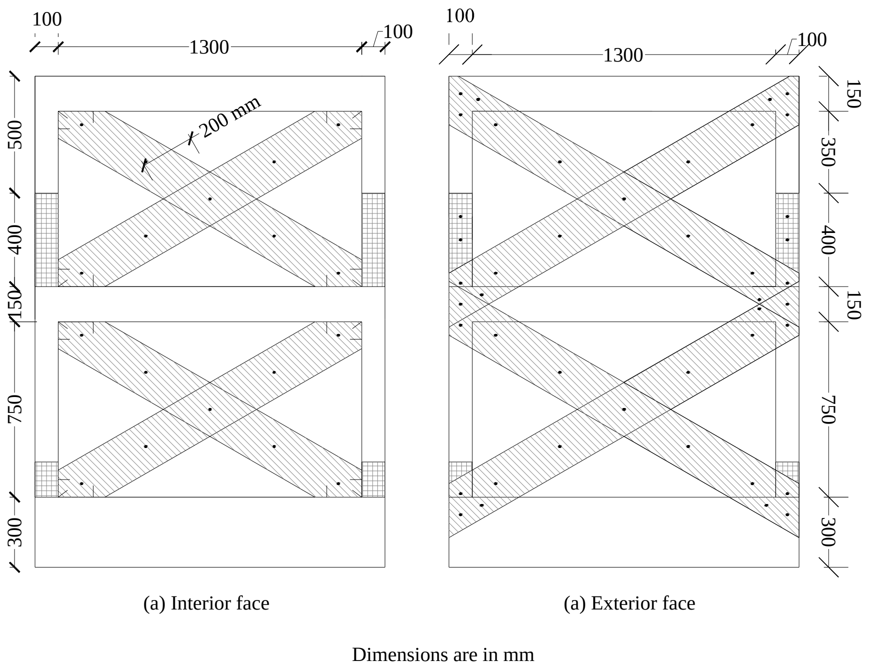


Figure 2.15 CFRP Strengthening Applied to Specimen SP-6

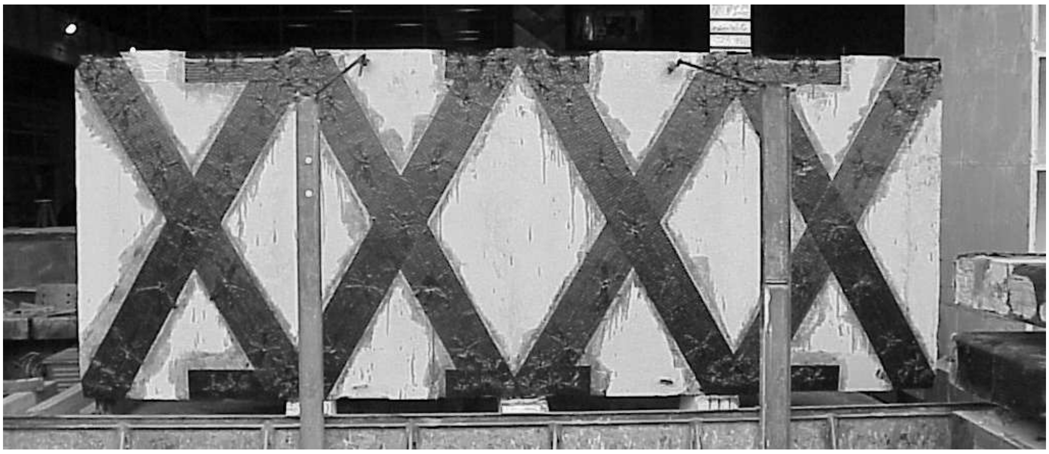
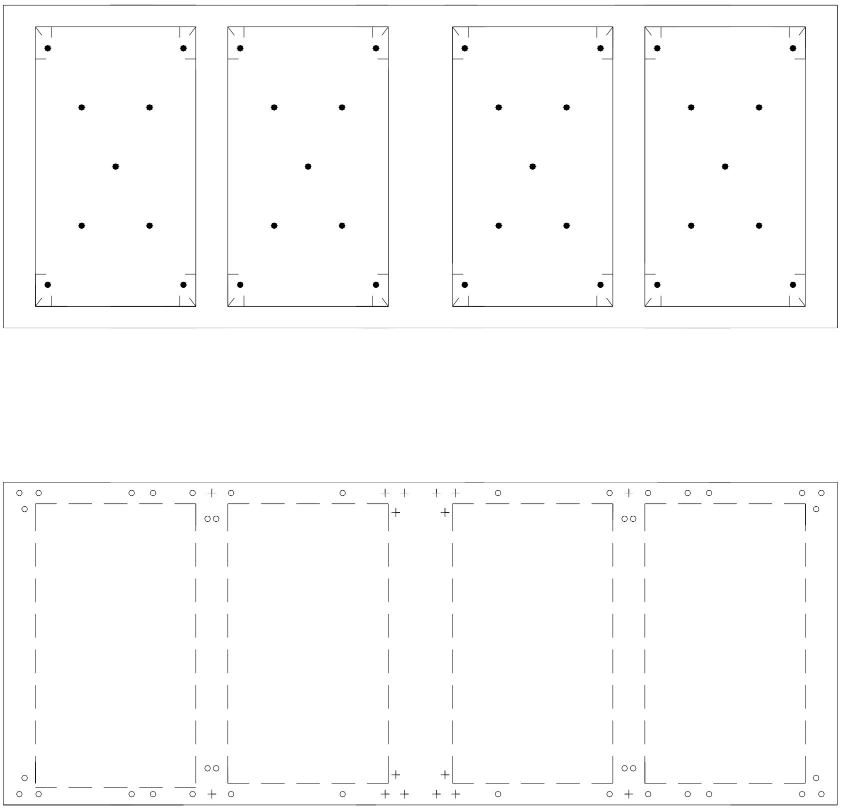


Figure 2.16 Specimen SP-6

Table 2.2 Properties of Anchor Dowels of Specimen SP-6

Anchor Type	Applied to	Depth in RC (mm)	Width of Strip (mm)	Diameter of Hole (mm)
A	Infill		50	10
B	Concrete	50	25	10
C	Concrete	60	25	10
D	Concrete	60	40	12



- • TYPE A
- – TYPE B
- ○ TYPE C
- + TYPE D

Figure 2.17 Location of Anchor Dowels in Specimen SP-6

2.2.8 Specimen 7 (SP-7)

Specimen SP-6 reached its capacity by shear failure which took place at the beam-column joint at the first story level. This failure was initiated by the failure of anchor dowels resulting in a sudden unloading in the tension strut. In designing the CFRP detailing of the last specimen (SP-7), it was aimed to strengthen the first story joints for shear by using CFRP. For this, the size of the anchor dowels used at the first story joints was increased. These dowels were expected not only to anchor CFRP struts to the specimen but also to increase the shear strength of the joints since Type-E anchors penetrated into the joints 150mm. The locations and properties of the anchor dowels are summarized in Figure 2.18 and Table 2.3.

Table 2.3 Properties of Anchor Dowels of Specimen SP-7

Anchor Type	Applied to	Depth in RC (mm)	Width of Strip (mm)	Diameter of Hole (mm)
A	Brick		50	10
B	Concrete	50	25	10
C	Concrete	60	25	10
D	Concrete	60	40	12
E	Concrete	150	60	14

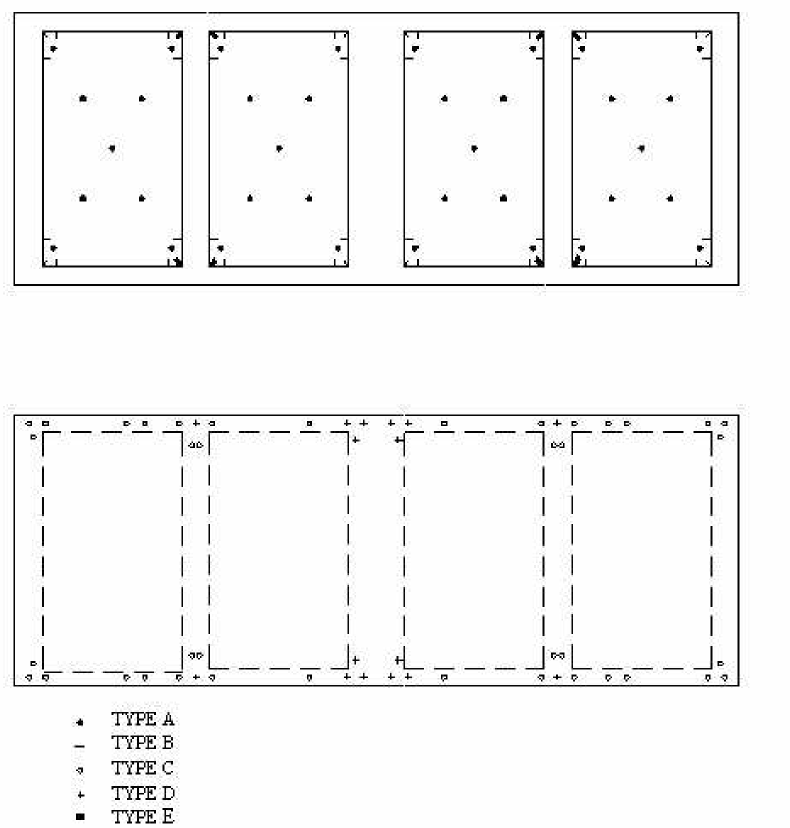


Figure 2.18 Location of Anchor Dowels in Specimen SP-7

2.3 MATERIALS

Concrete

C10 concrete was aimed at in the construction of the frame specimens. However, the strength of concrete was slightly higher than 10 MPa on the test day. The main reason for this lay in the fact that the tests were carried out about 150 days after the concrete had been cast. The mix design of concrete is given in Table 2.4.

Table 2.4 Mix Design of Concrete

Material	**Amount in 1 m^3 of Concrete (kg)**	**Amount in 1 m^3 of -Concrete (%)**
0-3 aggregate	457.1	19.0 %
3-7 aggregate	914.3	38.1 %
7-15 aggregate	485.8	20.2 %
Cement	285.7	11.9%
Water	257.1	10.8%
Total	**2400**	**100%**

2.3.2 Mortar and Plaster

The mix design for the mortar used in the construction of the brick wall and for the plaster were identical. Mix proportions are given in Table 2.5.

Table 2.5 Mix Design of Mortar and Plaster

Material	**Percentage by Weight (%)**
0-3 Aggregate	61.0%
Cement	10.5 %
Clime	10.5 %
Water	18.0 %
Total	**100 %**

Plastering both faces of the infill by using mortar completed the second phase of the fabrication. The thickness of the plaster was about 10 mm.

2.3.3 Steel

Plain bars were used as both longitudinal and transverse reinforcement in frame members. Properties of reinforcing bars used in frame members are given in Table 2.7.

2.3.4 CFRP Reinforcement

MBrace FRP "wet lay-up" application was used as CFRP reinforcement. This system is composed of four materials:

- Concresive 1305, to improve the bonding of the composite to the substrate.
- MBrace Rasatura (Putty), to even out any imperfections in the base.
- MBrace Adesivo (Saturant), main adhesive of the system.
- MBrace Fiber, fiber reinforcement of the system.

Mechanical properties of the composite are summarized in Table 2.6.

Table 2.6 Mechanical Properties of CFRP

Tensile Strength • characteristic, f_{ct} (MPa) • overlapping (l_s>20 cm), (MPa)	3430
Characteristic Tensile Modulus of Elasticity E, (MPa)	230000
Ultimate Strain, ε_u (%)	1.5

The material properties of the test specimens are given in Table 2.7.

Table 2.7 Properties of Reinforcement in RC Frame Members.

		SPECIMEN						
		SP-1	SP-2	SP-3	SP-4	SP-5	SP-6	SP-7
Longitudinal Reinforcement	Yield Strength (MPa)	388	388	388	388	388	388	388
	Ultimate Strength (MPa)	532	532	532	532	532	532	532
	Diameter (mm)	8	8	8	8	8	8	8
Transverse Reinforcement	Yield Strength (MPa)	279	279	279	279	279	279	279
	Ultimate Strength (MPa)	398	398	398	398	398	398	398
	Diameter (mm)	4	4	4	4	4	4	4
Number of Bars	Beams and Columns	4	4	4	4	4	4	4
	Foundation Beam	6	6	6	6	6	6	6
Detailing of Transverse Reinforcement (Beams &Columns)	Spacing (mm)	100	100	100	100	100	100	100
	Hook Angle (°)	90	90	90	90	90	90	90
Detailing of Transverse Reinforcement (Beams &Columns)	Spacing (mm)	50	50	50	50	50	50	50
	Hook Angle (°)	135	135	135	135	135	135	135

Table 2.8 Properties of Test Specimens

Compressive Strength of Concrete (MPa)			19.5	15.3	12.9	17.4	12.0	14.7	17.5
Compressive Strength of Mortar (MPa)			4.3	4.3	3.1	2.9	4.1	4.2	4.3
CFRP	Infill	Application Side	None	Both	Exterior	Both	Both	Both	Both
		Type	-				Strut	Strut	Strut
		Anchorage	-	-	Yes	Yes	Yes	Yes	Yes
	RC Frame	Application Side	None	None	Ext.	Ext.	Ext.	Ext.	Ext.
		Type					Strut	Strut	Strut
		Anchorage			Yes	Yes	Yes	Yes	Yes

Note: Blanket covers the whole infill surface

2.4 FABRICATION OF THE SPECIMENS

Test specimens were fabricated in three phases. First, the reinforced concrete frame was constructed. After the reinforcement of the specimen was prepared, the concrete was cast in the horizontal position. When the concrete gained adequate strength, the specimen was placed in a vertical position. The hollow clay tiles were laid while the specimen was in a vertical position (Figure 2-19). When infill of the upper frame was completed, the specimen was turned upside down and the infill of the second frame was constructed.

In the third phase, CFRP reinforcement was bonded to the specimens. CFRP reinforcement consists of four materials.

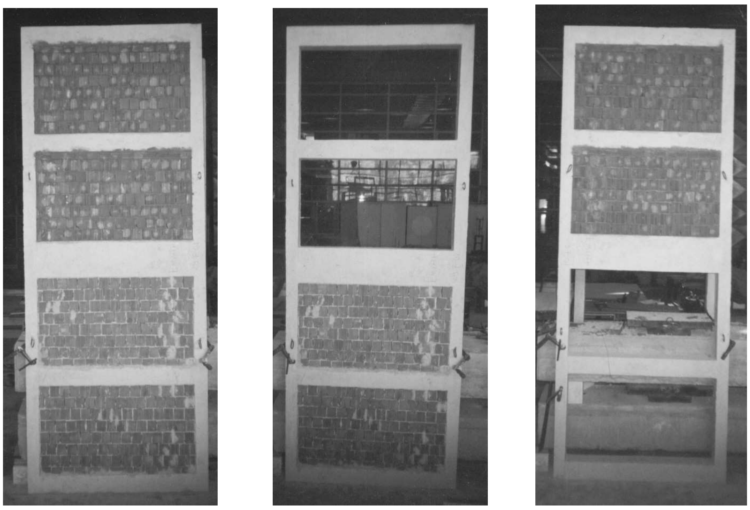


Figure 2.19 Construction of Hollow Tile Infills

First, the Concresive coat was applied to the surface on which CFRP sheets would be applied. The main function of Concresive is to improve the bonding of the composite to the substrate. After the concresive coat applied to the surface completely dried, the surface was leveled by applying Rasatura (Putty). Then, the components of the Adesivo, the main adhesive of the system, were mixed to have the Saturant prepared. Approximately two hours after the application of the Rasatura, the Saturant was evenly applied to the concrete surface using a roller. Immediately after the application of the Saturant, the fiber sheet layer was applied. The sheet was strongly pressed in the longitudinal direction of the fibers using an iron roller to make the Saturant penetrate into the sheet and to eliminate air. Then, the Saturant and iron roller were applied once again to finalize the application of one layer of CFRP sheet. If another layer was to be applied, the Saturant would be re-applied on the first layer and after the placement of the second layer the same procedure would be followed to end the application.

The last phase of the fabrication of specimens was the preparation of the anchor dowels. First, the holes of specified length and diameter were drilled in the specified locations. The dust in the holes was removed using pressurized air. To make the anchor dowels, CFRP was cut into pieces of specified length. The pieces were rolled and tied at three locations (middle and two ends). Next, these pieces were folded into two and a 150 mm long 1 mm diameter wire was inserted into them, after which they were tied together. Preparation of anchor dowels is shown in Figure 2.20.

After the application of CFRP reinforcement, each hole was filled with the Saturant using a medical syringe. The fibers of the anchor dowel that remained outside the hole were separated using a knife and bonded to the surface of the CFRP reinforcement by using the Saturant. As the last step, the Saturant was re-applied to the hole. The cross section of the infill and of concrete with anchor dowels is shown in Figure 2.21.

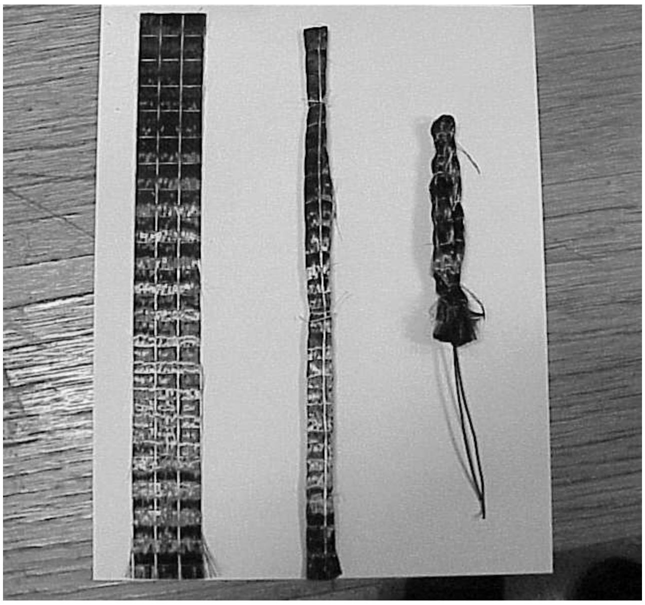
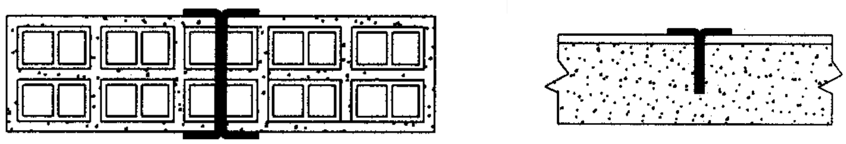


Figure 2.20 Preparation of Anchor Dowels



(a) Type A (b) Types B, C, and D

Figure 2.21 Type of anchor dowels in (a) the infill (b) concrete

3. TEST PROCEDURE

3.1 GENERAL

In this study, seven one-bay two-story frames were tested. These frames were assumed to be fixed at the base. To simulate this behavior, two identical frames connected with a rigid foundation beam were cast together. Apart from the symmetry of the test specimen, the loading also had to be symmetric with respect to the foundation beam in order to minimize the rotation of the base of the columns.

3.2 TEST SETUP AND LOADING

All specimens were tested in a horizontal position. The loading frame consisted of two reinforced concrete reaction beams connected by steel sections. The load was applied at the foundation beam level. The reactions at each end represented the lateral load applied to each of the twin specimens at the second story levels. The test setup is shown in Figures 3.1 and 3.2.

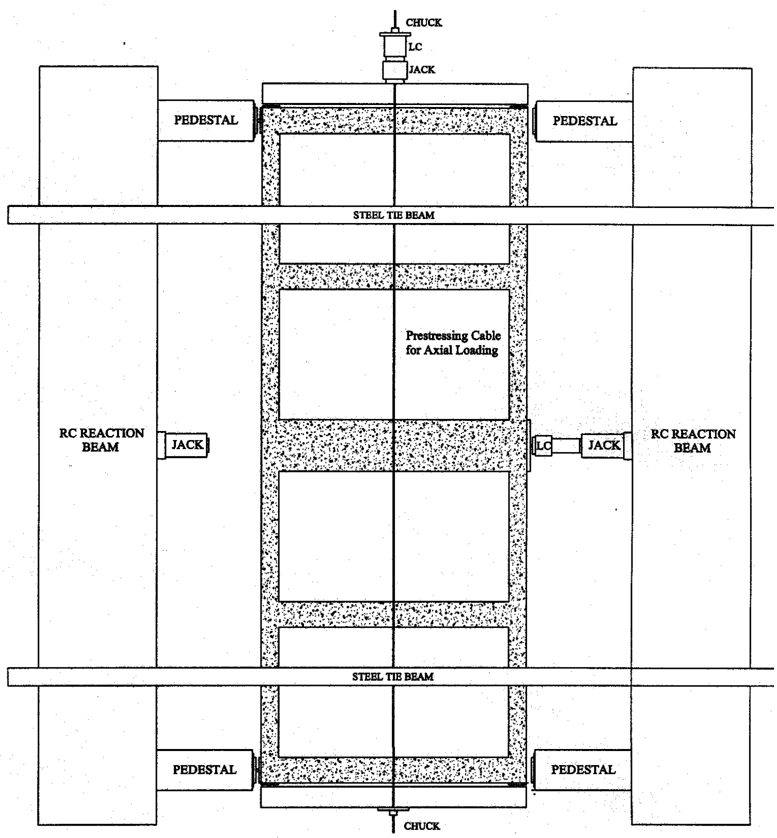


Figure 3.1 Test Setup

To simulate the real behavior, axial load was applied on the columns by post tensioning two cables. The post tensioning force was applied with the help of a hydraulic jack to the mid span of a simply supported steel beam. The support reactions of the steel beam were the axial loads on the columns, Figure 3.3.

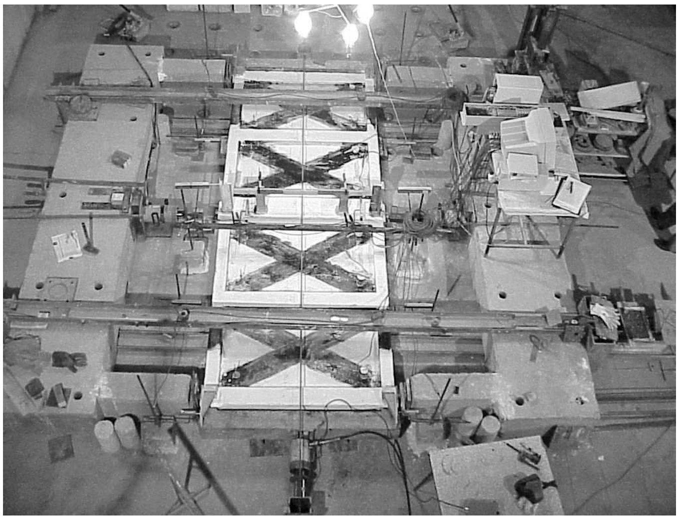


Figure 3.2 Test Setup and the Specimen

Ball bearings were placed under the first story beam-column joints to hold the specimen horizontal without restraining the displacements.

The specimens were tested under reversed cyclic lateral loading. For this purpose, lateral load was applied at the foundation beam level through hydraulic jacks. The reaction forces acting through the pedestals were the lateral loads applied at the top story level of each of the twin frames. The forces acting on the specimen are shown in Figure 3.3. To simulate the reversed cyclic loading, the direction of the applied force was reversed in each cycle. The load applied at the foundation level was measured by a load cell.

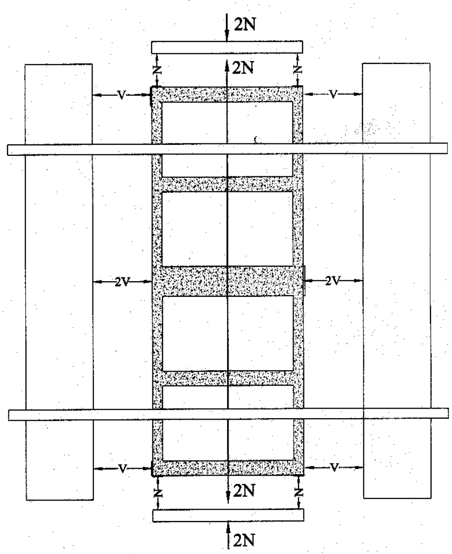


Figure 3.3 Forces Acting on the Test Specimen

3.3 INSTRUMENTATION

The load applied at the foundation level, which corresponded to twice the lateral load applied to each of the twin specimens, was measured by a load cell. Displacements and rotations of the test specimens were recorded during the test using dial gauges and Linear Variable Differential Transducers (LVDT). The locations of the dial gauges and LVDTs are shown in Figure 3.4. These transducers sent voltage signals to the data acquisition system, which then converted these signals to displacement and load values. Signals from the load cell were also sent to the data acquisition system.

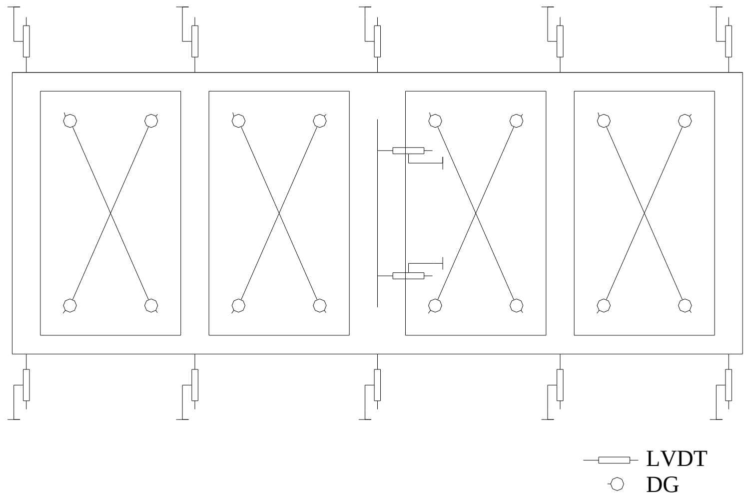


Figure 3.4 Instrumentation

3.3.2 Measurement of Lateral Displacements

The lateral displacements were measured using LVDTs. For this purpose, LVDTs were mounted at the second floor, first floor, and foundation levels, in the loading direction. Also two LVDTs were mounted to the foundation beam to measure the rotation of the foundation beam (Figure 3.4). The beams were assumed to be inextensible in the longitudinal direction.

The calculations of the lateral displacements are presented in Appendix A.

3.3.3 Measurement of Shear Deformations

The shear deformations in the infill were measured using diagonally placed electrical dial gauges. The gauges were placed at some distance away from the corners of the infill to avoid local effects. Calculations of the shear deformations are presented in detail in Appendix B.

3.4 TEST PROCEDURE

After the construction of a test specimen, it was moved to the test rig and placed on the ball bearings. Then the specimen was whitewashed in order to be able to monitor the cracks clearly. After whitewashing, the hydraulic ram and the load cell were placed at the foundation level. LVDTs and dial gauges were mounted on the specimen and connected to the data acquisition system. Then the instruments were checked and calibrated. The standard cylinder tests were performed to determine the strength of concrete on the day of testing.

The specimens were subjected to reversed cyclic lateral loading. One half cycle consisted of the loading of the specimen to a specified load level and unloading. The direction of the load was reversed after each half cycle. The same loading history was applied in both tests. The loading histories, story drifts and shear deformations are presented in the following chapter.

3.5 CRITICAL REMARKS ON THE TEST SPECIMEN AND TESTING

Some assumptions and simplifications had to be made in this experimental study. Test results and conclusions should be evaluated in the light of these assumptions and simplifications which are given below:

- Lateral loads were applied at the second story. In reality, application of earthquake forces to structures is not that simple.
- Specimens were cast in one go preventing the formation of cold joints at the floor levels.
- Since the test specimens are one bay frames, one of the columns is subjected to direct tension under the lateral load applied. Axial tension changes the mode of failure and decreases the moment capacity of the column.
- The tests were carried out under quasi – static loading. This type of loading is a heavy punishment to the specimens as far as the evaluation of dynamic response is concerned.
- Since the specimens were tested in a horizontal position, the self weights of members were not applied as loads.
- There were no lap splices in column longitudinal bars at the foundation level and the lap splice length in the first story level was in accordance with the code.

4. TEST RESULTS

4.1 GENERAL

In this chapter, test results and the behavior of the test specimens are discussed.

4.2 OBSERVED BEHAVIOR OF TEST SPECIMENS

4.2.1 Specimen SP-1

Specimen SP-1 was an unstrengthened hollow clay tile infilled reinforced concrete frame. SP-1 was tested under the load history given in Figure 4.1. The applied axial load on the columns was 60 kN which was kept constant during the test. This axial load corresponded to 18% of the axial load capacity of the column.

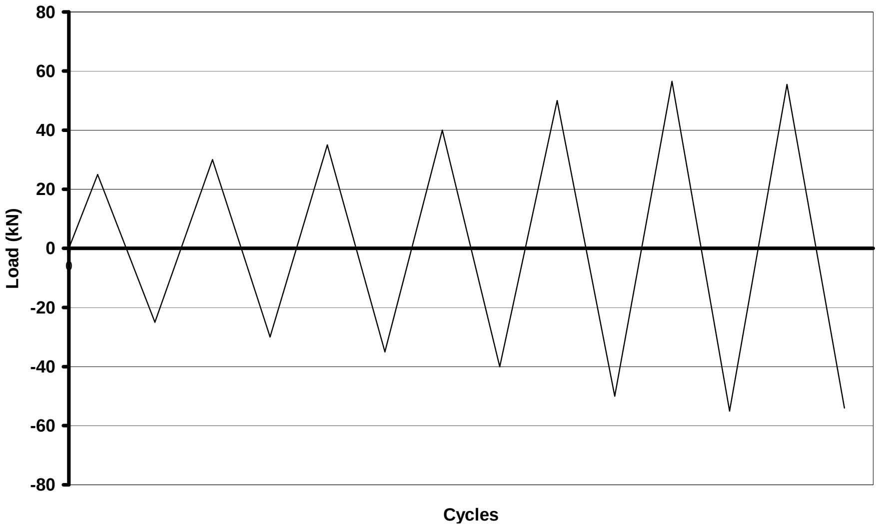


Figure 4.1 Lateral Load History of SP-1

The lateral load–drift ratio curves for all stories are given in Figures 4.2 and 4.3.

The behavior of SP-1 during the test is summarized below:

- In the first two cycles, hairline cracks were observed on the plaster at the base of the first story columns.
- In the third and fourth cycles, the lateral load level was 35 and 40 kN respectively. In these cycles, the cracks observed on the plaster widened.
- When the lateral load level was increased to 45 kN, the cracks observed on the plaster extended to the foundation beam. Diagonal plaster cracks formed on the brick infills.
- In the sixth cycle the lateral load level was 50 kN. At this load level serious damage occurred in the reinforced concrete frame. A crack occurred and widened at the top of the first story column. The lateral load capacity of the frame decreased due to this

crack. The infill of the first story started to separate from the frame at the foundation level.

- The lateral load was 55 kN in the seventh cycle. At this level the infill panels were completely separated from the frame. From this stage on, the test specimen behaved almost like a bare frame. The specimen failed due to the crushing of the first story beam column joint.

In Figure 4.4 photographs taken after the test are given.

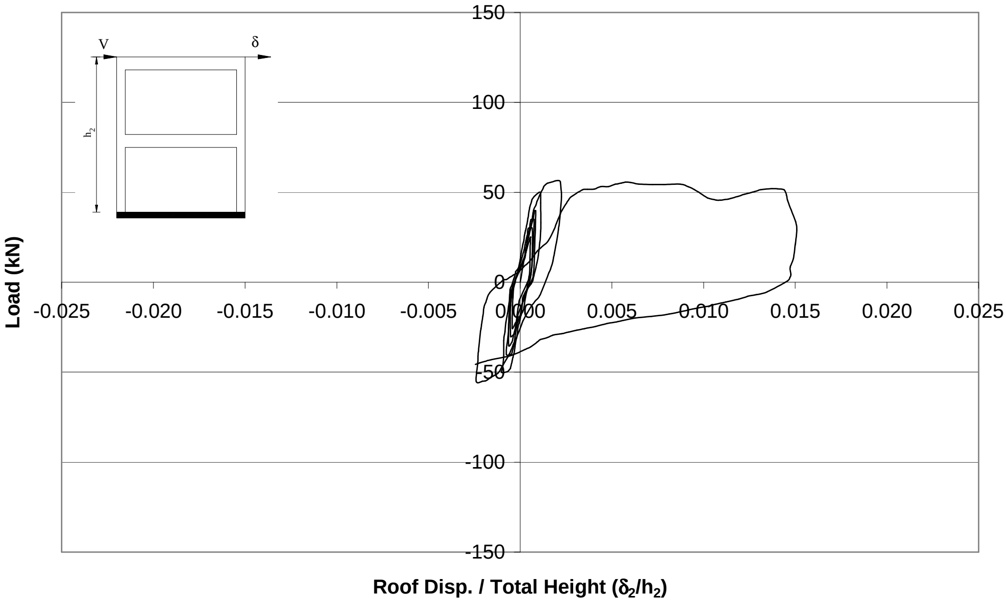


Figure 4.2 Lateral Load – Roof Drift Ratio Curves for SP-1

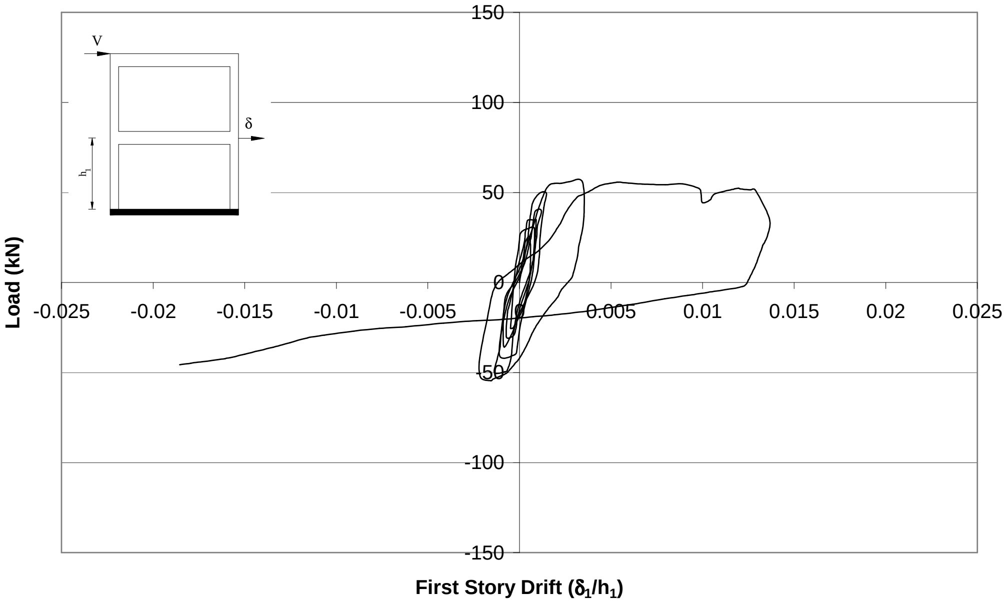


Figure 4.3 Lateral Load – First Story Drift Ratio Curves for SP-1

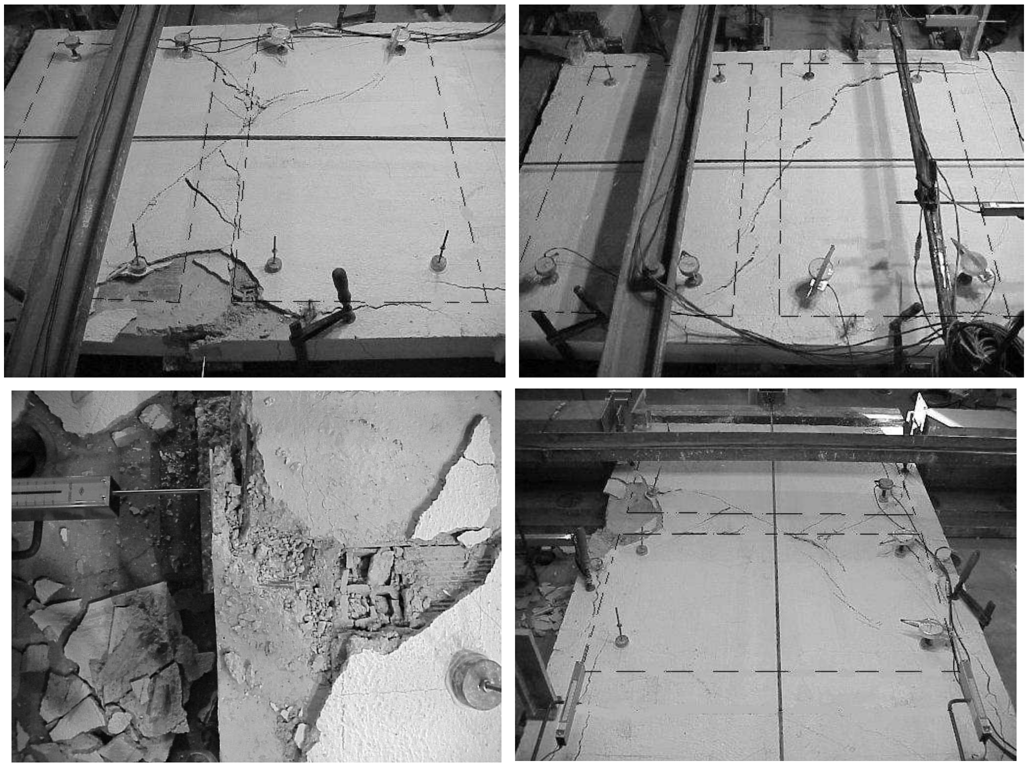


Figure 4.4 Specimen SP-1 after the Test

4.2.2 Specimen SP-2

The second specimen was strengthened by two orthogonal layers of CFRP reinforcement applied to both faces of the infill. CFRP covered the whole surface of the infill. The CFRP reinforcement was neither extended nor anchored to the reinforced concrete frame members.

SP-2 was tested under the loading history presented in Figure 4.5. The applied axial load on the columns was again 60 kN, which corresponded to 22% of the axial load capacity.

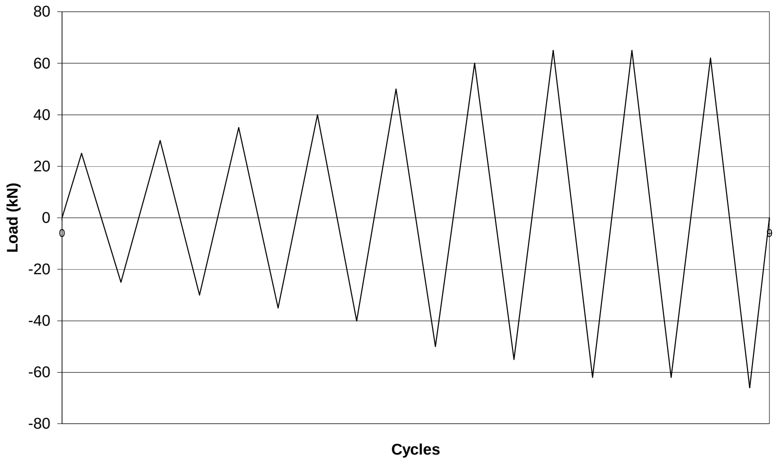


Figure 4.5 Load History of SP-2

The lateral load – drift ratio curves for the roof and the first story are given in Figures 4.6 and 4.7 respectively.

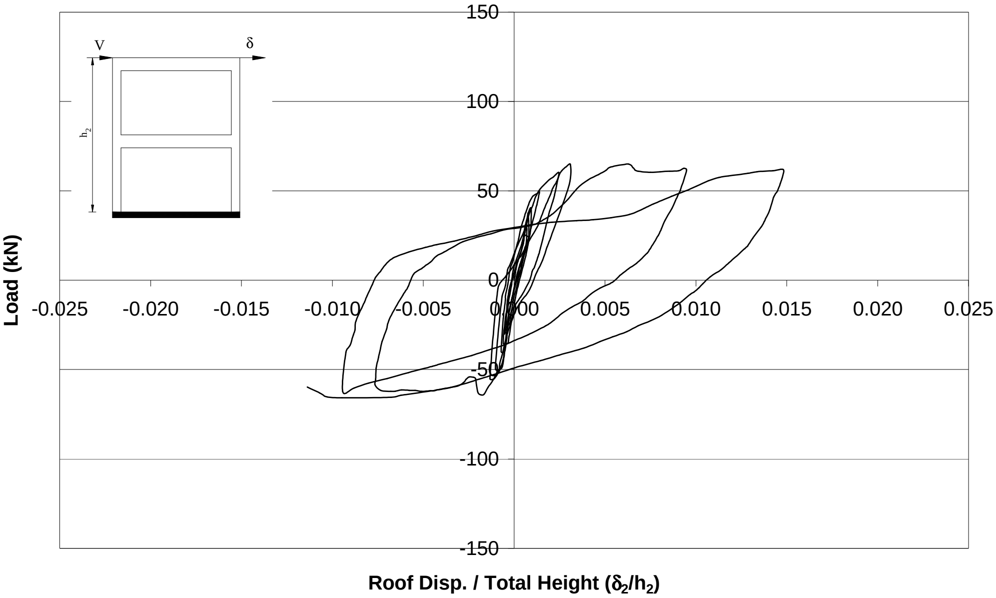


Figure 4.6 Lateral Load – Roof Drift Ratio Curves for SP-2

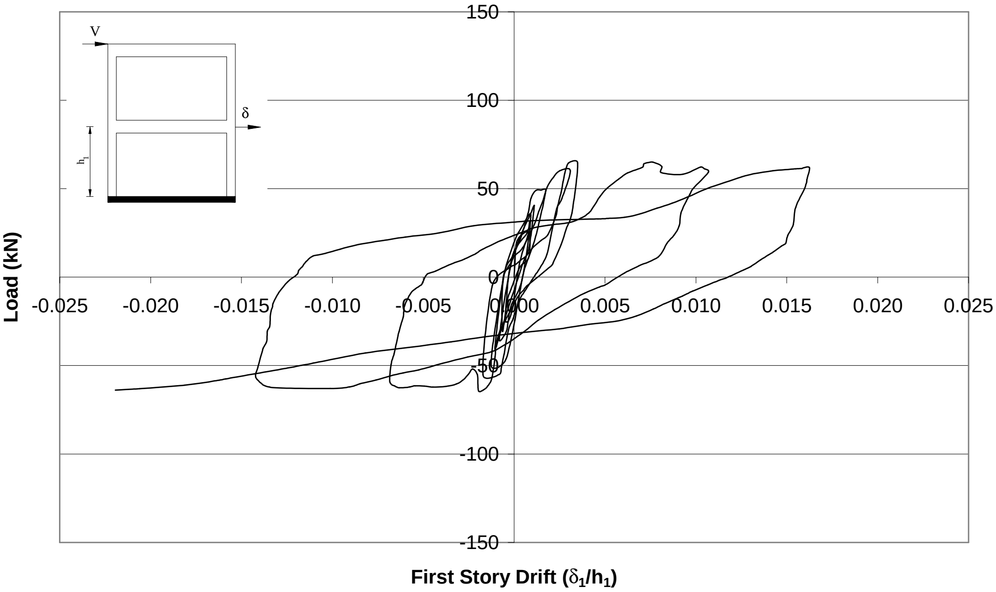


Figure 4.7 Lateral Load – First Story Drift Ratio Curves for SP-2

The observations made during the test are summarized below:

- In the first two cycles no cracks were observed.
- In the third cycle the maximum lateral load level was 35 kN. First hairline cracks were observed at the bottom of the first story columns.
- In the fourth and fifth cycles, the cracks observed widened.

- In the sixth cycle, new cracks were observed at the top and middle of the first story columns.
- In the seventh cycle, maximum load reached was 65 kN. At this level, the cracks observed so far widened and extended. The frame was so damaged that the load capacity of the specimen could not exceed 65 kN in the following cycles.
- From this point on, the test was carried out on displacement controlled basis. In the eighth cycle, there was severe damage at the top of the first story columns.
- The specimen failed due to the crushing of the first story beam-column joints in the ninth cycle.

The damage which occurred in the specimen during the test is shown in Figure 4.8

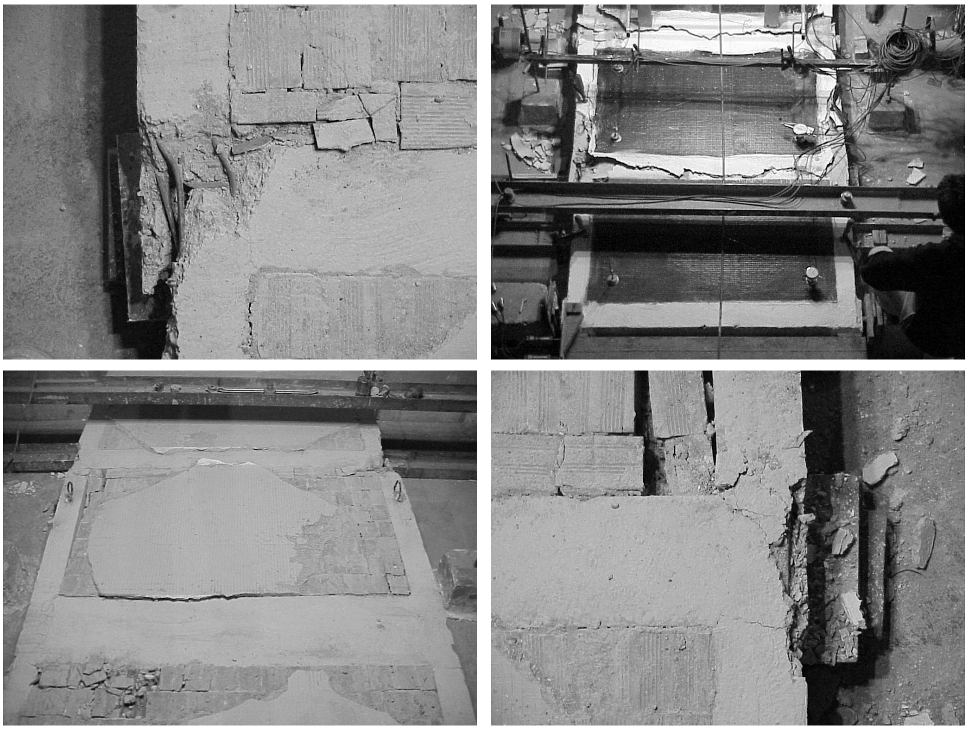


Figure 4.8 SP-2 after the Test

4.2.3 Specimen SP-3

This specimen was strengthened by the application of CFRP reinforcement only on the exterior face of the specimen. CFRP was applied both on the infills and the frame members.

The axial load applied to each column (60kN) corresponded to 25% of the axial load capacity. SP-3 was tested under the loading history given in Figure 4.9.

The lateral load – drift ratio curves of the second and first stories are presented in Figures 4.10 and 4.11 respectively.

- When the lateral load level was increased to 60 kN, delamination of CFRP was observed at the foundation level. The cracks observed so far widened at this level.
- In the ninth cycle the maximum load was 65 kN. At this load, the infill of the second story separated from the frame. Anchor dowels at the second story joints failed. As a result, debonding of CFRP was observed at second story level.

- After the ninth cycle, it was not possible to increase the load. Thus, the test was conducted under displacement control after this point. At the first half cycle the maximum load attained was 50 kN and in the negative half cycle it was 40 kN. At this cycle, the anchor dowels at the middle of the second story columns failed. The second story beam-column joints failed by the crushing of concrete.
- In the last cycle, the stiffness of the second story was far less than the stiffness of the first story. Thus, second story of the frame behaved like a one bay one story frame.

The state of SP-3 after the test is shown in Figure 4.12.

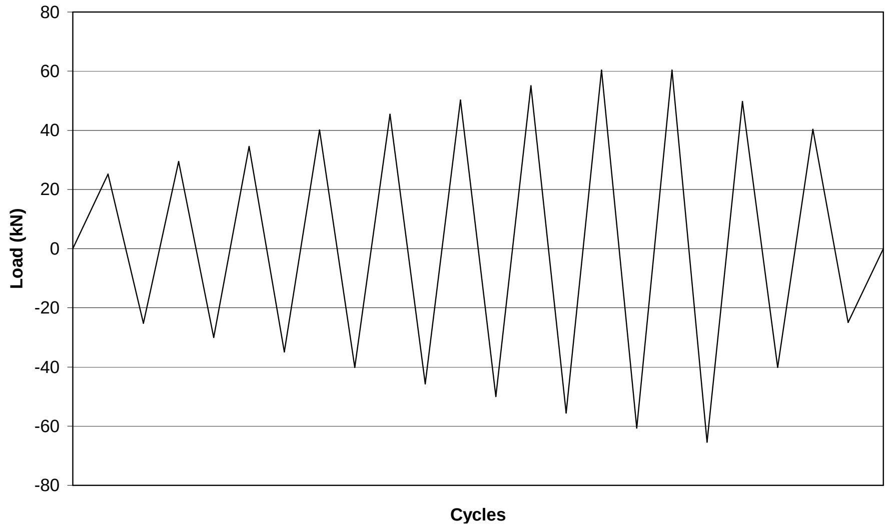


Figure 4.9 Load History of SP-3

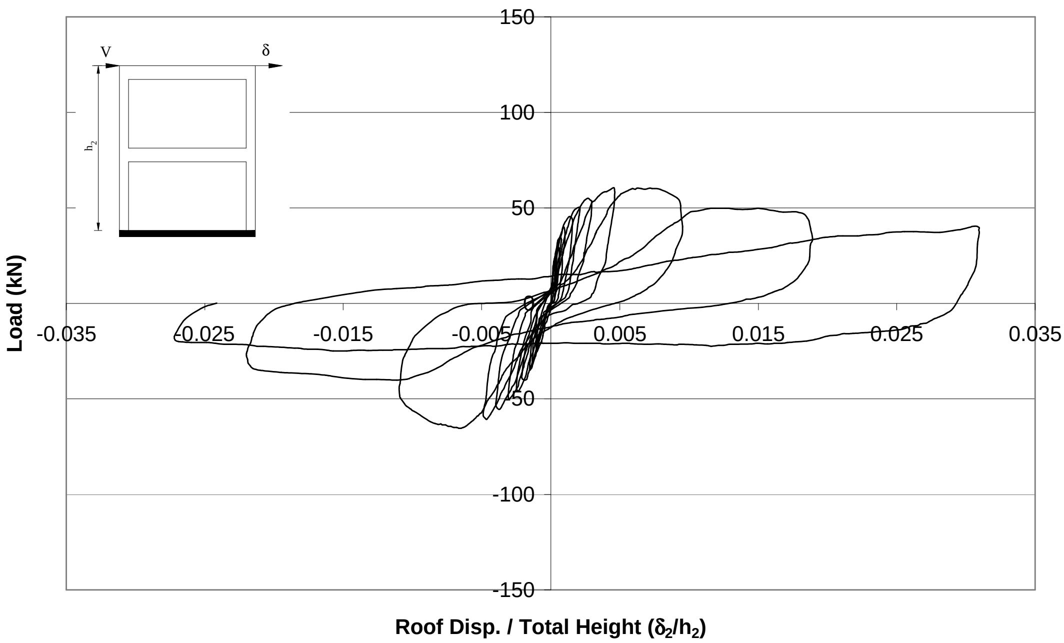


Figure 4.10 Lateral Load – Roof Drift Ratio Curves for SP-3

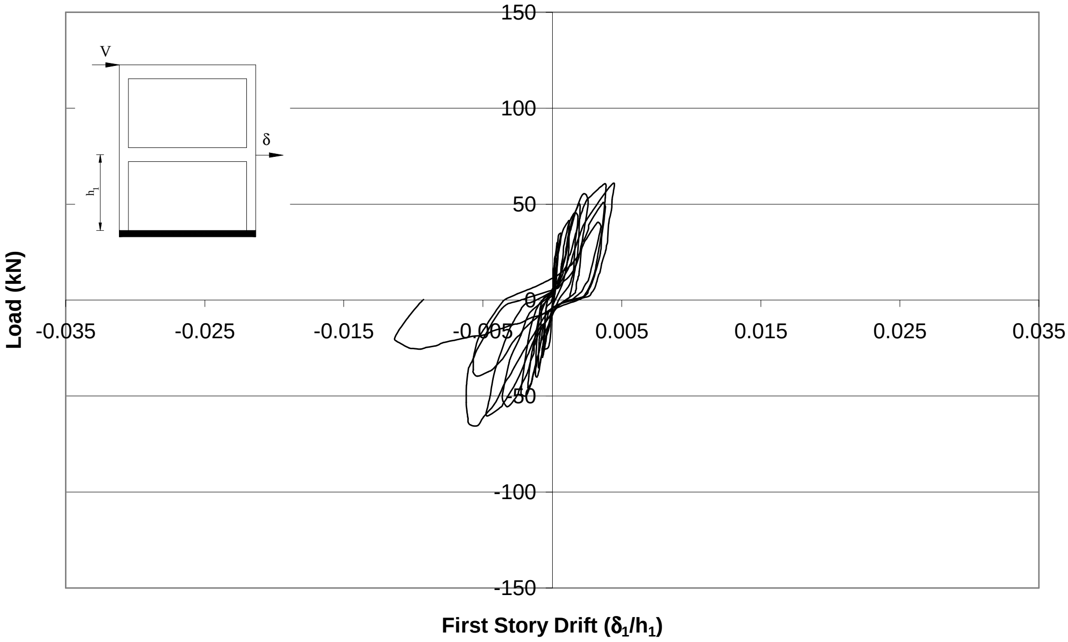


Figure 4.11 Lateral Load – First Story Drift Ratio Curves for SP-3

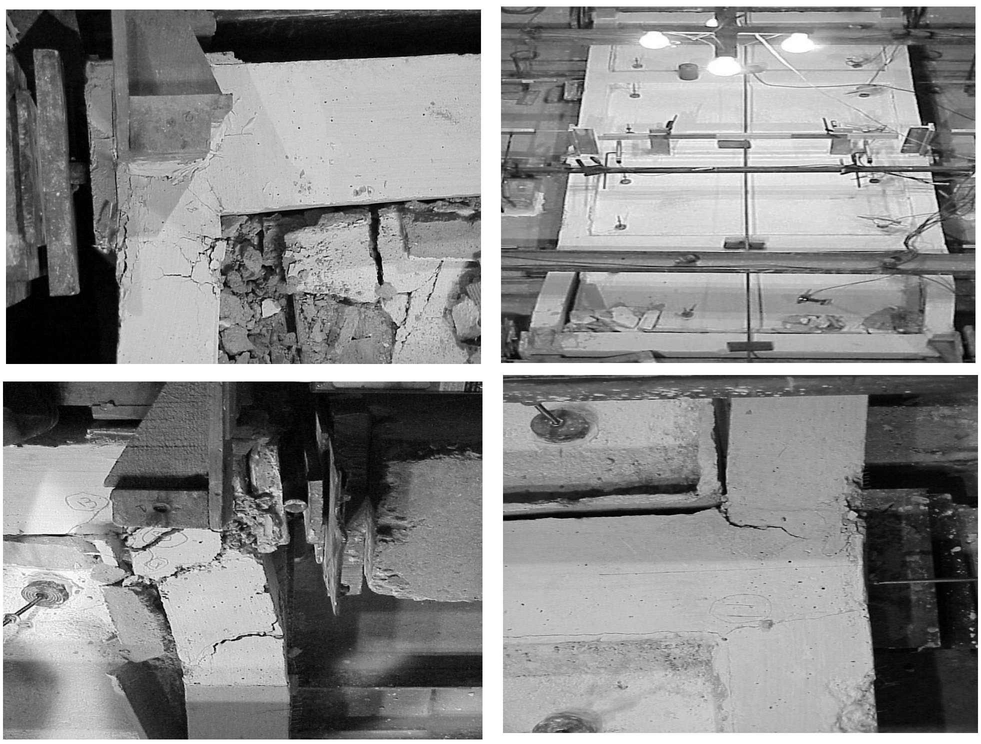


Figure 4.12 SP-3 after the Test

4.2.4 Specimen SP-4

In this specimen CFRP was applied to the infill on both faces. CFRP covered the whole surface of the infill. CFRP was extended to the frame members. Anchor dowels were used to anchor the CFRP to the infill and to the frame members.

In this test, the axial load applied to each column corresponded to 20% of the axial load capacity. SP-4 was tested under the loading history given in Figure 4.13.

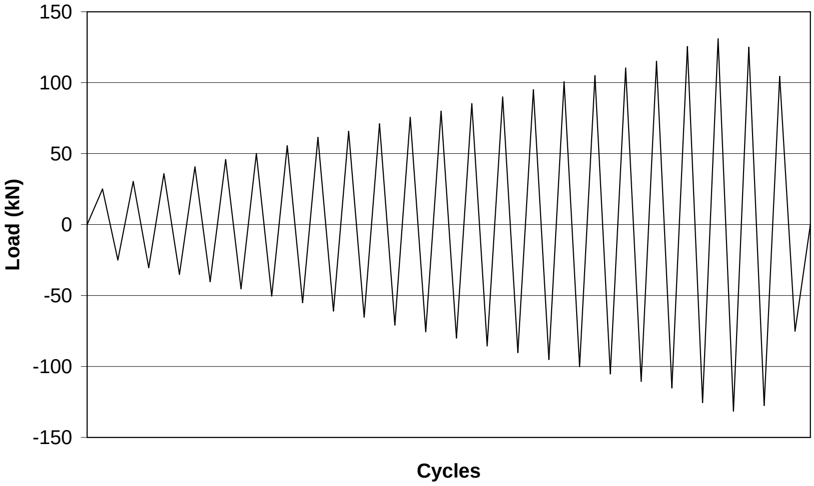


Figure 4.13 Load History for SP-4

The lateral load–drift ratio curves are presented in Figure 4.14 for the second story and in Figure 4.15 for the first story.

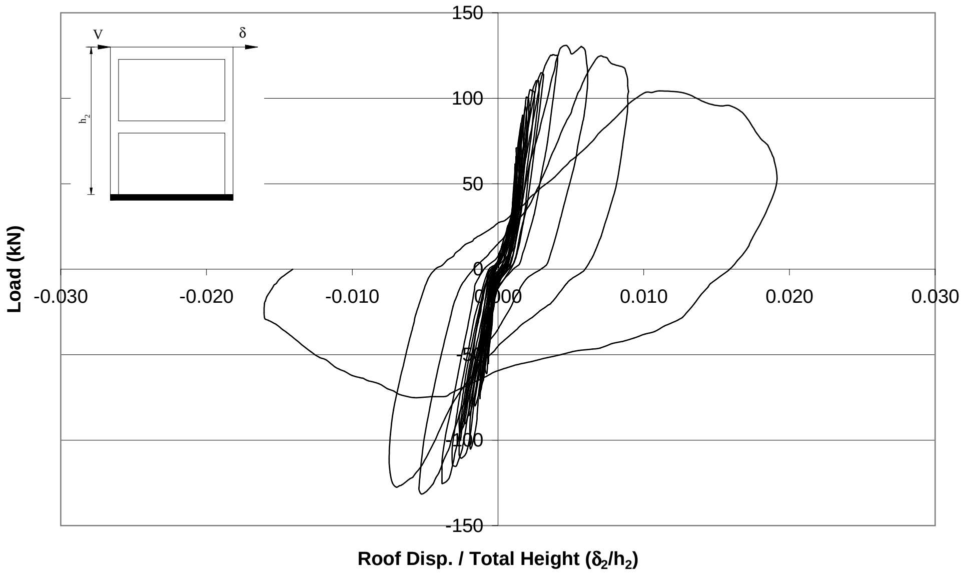


Figure 4.14 Lateral Load – Roof Drift Ratio Curves for SP-4

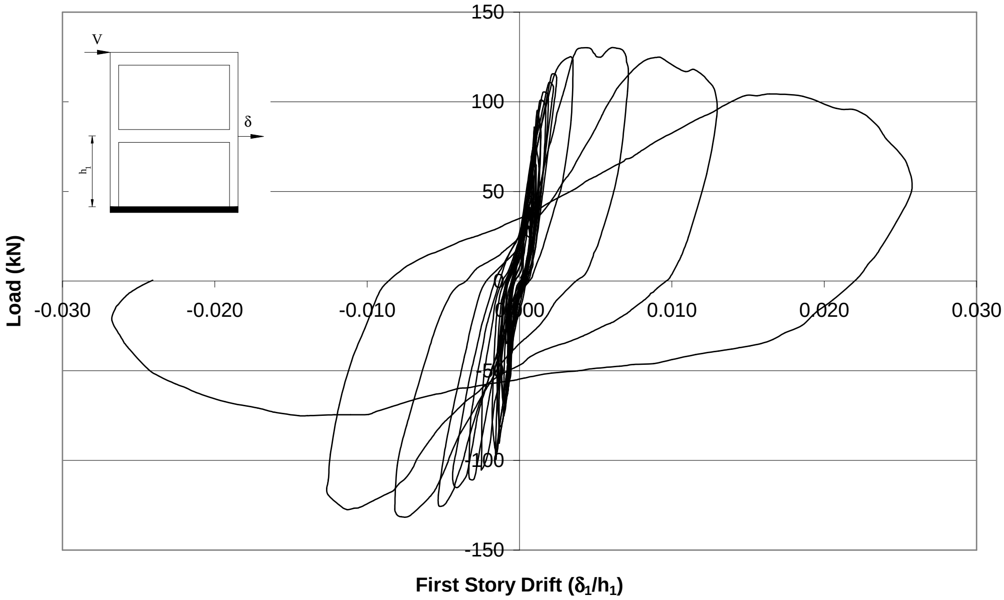


Figure 4.15 Lateral Load – First Story Drift Ratio Curves for SP-4

The behavior of the specimen during the test is summarized below:

- In the first thirteen cycles no cracks were observed.
- A hairline crack was observed at the bottom of the first story columns at a lateral load of 90 kN.
- In the fifteenth cycle, the load was 95 kN. In this cycle, the crack observed in the previous cycles elongated, but cracks were still hairline cracks.
- When the load was 100 kN, the cracks observed previously extended and widened.
- In the seventeenth cycle diagonal hairline cracks were observed at the first story beam-column joints. Some hairline cracks were initiated at the top of the first story columns.
- In the eighteenth cycle, the maximum load was increased to 110 kN. At this level, plaster started to separate from the frame at the foundation level.
- The maximum load reached was 115 kN at the nineteenth cycle. New hairline cracks were observed at the first story joints and at the middle of the first story columns.
- In twentieth positive half cycle it was intended to go up to 120 kN. Nevertheless the specimen could not reach this load. Maximum lateral load achieved was 118 kN. In this cycle, complete delamination of plaster on the bottom side was observed with the failure of the anchor dowels.
- When the lateral load was increased to 125 kN, cracks occurred at the ends of the beams due to the tensile forces on the anchor dowels. At this load level there were several hairline cracks on the first story joints and columns.
- In the twentieth cycle, the ultimate load carrying capacity of the specimen was reached. The maximum load was 131 kN in this cycle. At this load level CFRP buckled at the edge of the first story joint. The cracks at the bottom of the first story columns widened significantly. The forces in the anchor dowels at the foundation level were so high that, concrete at the corner of the first story infills failed due to these high tensile forces resulting in the delamination of CFRP at the foundation level.
- In the twenty-second cycle, the maximum loads were 125 kN in the positive half cycle and 127 kN in the negative half cycle. At this load level, the anchor dowels at the bottom of the first story column failed. CFRP was completely delaminated at the

foundation level. Crushing of concrete at the bottom of first story columns occurred, indicating the flexural failure of the specimen.

- In the last cycle, the maximum loads were 104 kN and 75 kN in the positive and negative half cycles respectively. Longitudinal bars of the columns buckled at the foundation level. A tie at this level broke due to buckling.

Figure 4.16 demonstrates the state of SP-4 after the test.

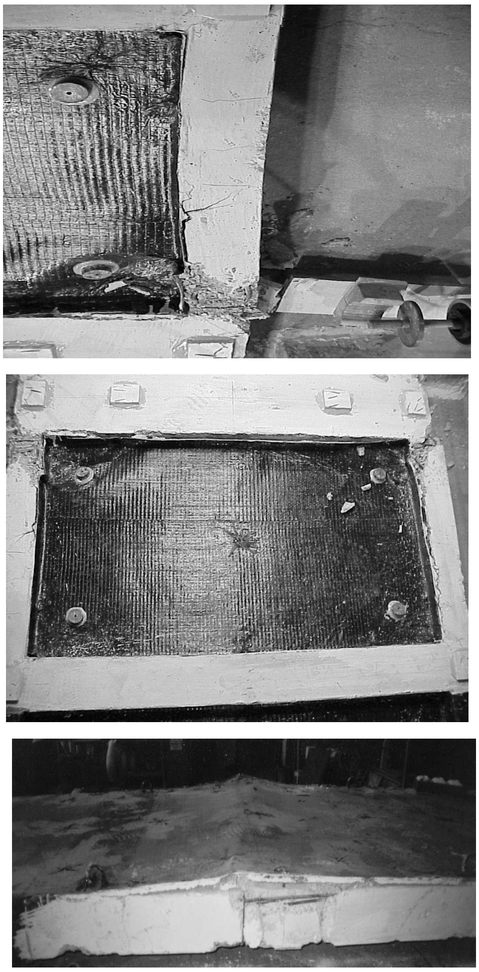


Figure 4.16 Specimen SP-4 after the Test

4.2.5 Specimen SP-5

In this specimen CFRP applied consisted of strips arranged as cross-bracing members (Figure2.12). Strips were anchored to the both the infill and the frame members. The axial load applied (60kN) corresponded to 25% of the axial capacity of the column. The lateral load history of SP-5 is presented in Figure 4.17.

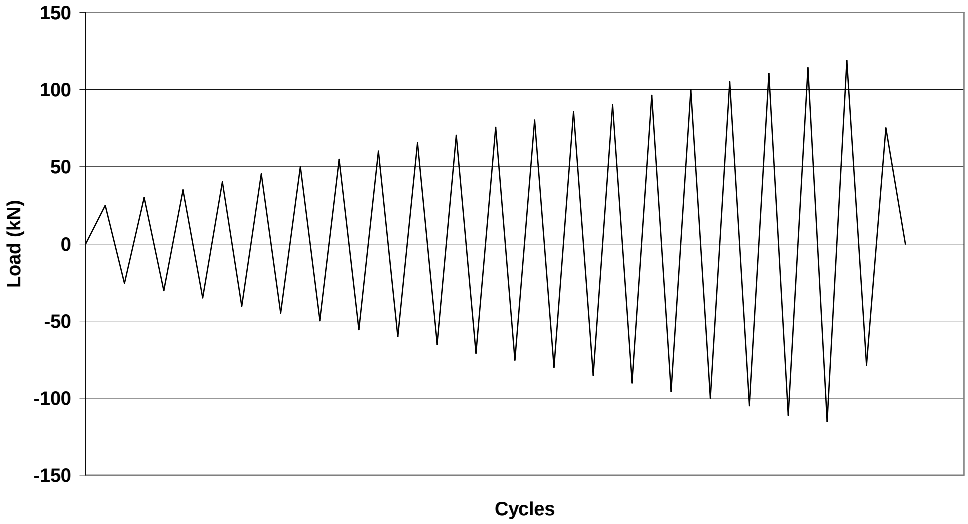


Figure 4.17 Load History of SP-5

Figure 4.18 and 4.19 show the lateral load – drift ratio curves for second and first stories respectively.

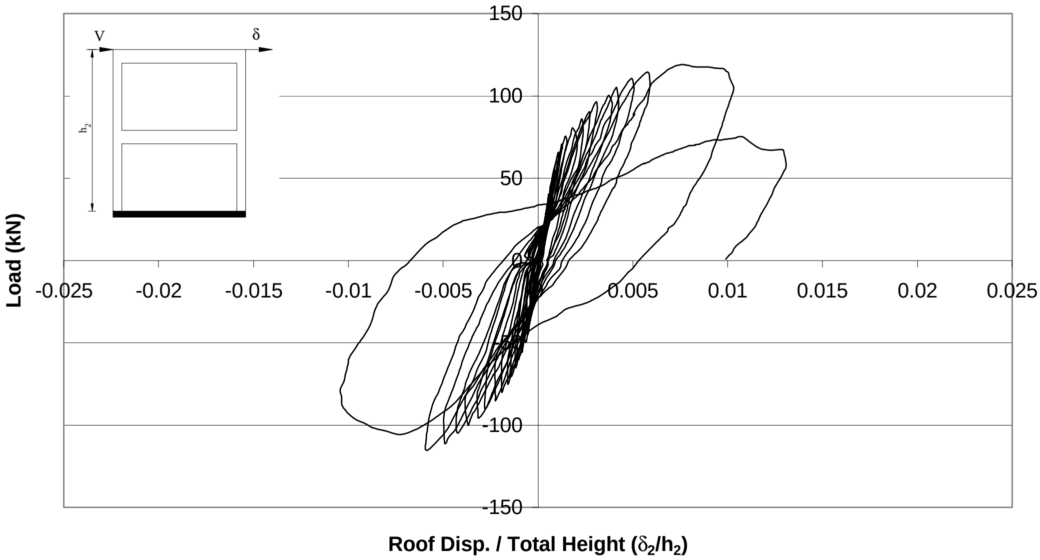


Figure 4.18 Lateral Load – Roof Drift Ratio Curves for SP-5

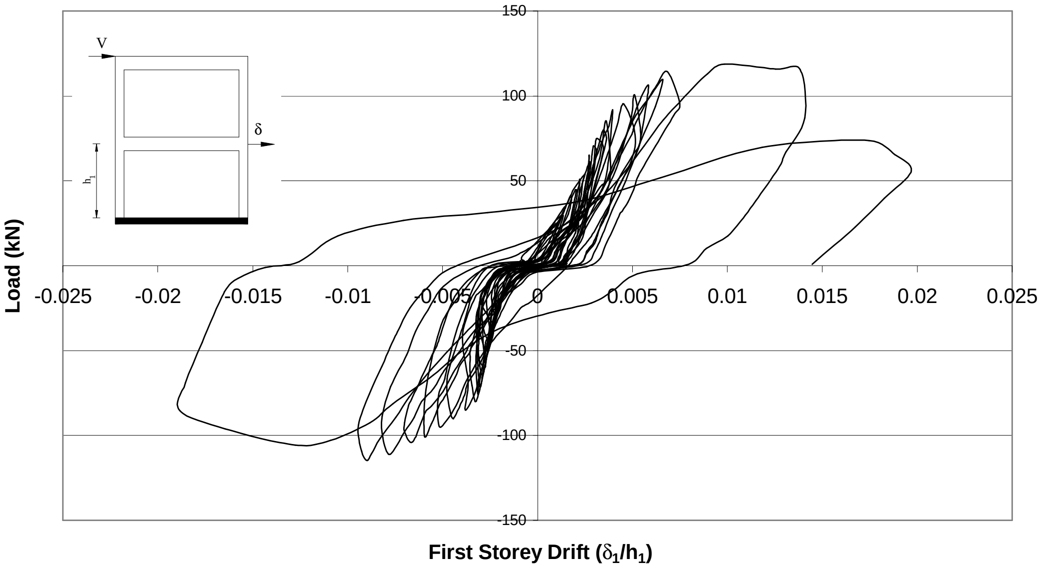


Figure 4.19 Load – First Story Drift Ratio Curves for SP-5

The behavior of SP-5 during the test can be summarized as follows:

- In the first six cycles no cracks were observed on the frame or on the infill panels.
- In the sixth forward cycle maximum lateral load was 55 kN. At this half cycle a hairline crack occurred at the bottom of the first story column on the tension side.
- In the following three half-cycles, no new cracks were observed.
- In the ninth forward cycle the lateral load was increased to 65 kN and new hairline cracks were observed on the tension face of the first story columns.
- The width of cracks observed previously began to increase in the tenth positive half cycle at which maximum load was 70 kN. In the negative half cycle new cracks occurred on the first story columns.
- There were a number of new hairline cracks on the tension face of the columns in the eleventh and twelfth cycles.
- When the lateral load was increased to 85 kN in the thirteenth cycle, cracks were initiated on the first story infills. The cracks observed so far could still be considered as hairline cracks.
- In the fourteenth cycle, the maximum lateral load was 90 kN. At this level plaster started to delaminate from concrete at the foundation level.
- The lateral load was 95 kN in the fifteenth cycle. Significant cracking on the corners of the tension struts occurred indicating that the load on the CFRP anchor dowels was quite high. Also pre-formed cracks at the bottom of the first story columns widened significantly.
- In the sixteenth positive half cycle CFRP layer on the foundation beam started to delaminate. In the negative half cycle separation between the infill of the first story and concrete was observed. Some cracks were observed on the foundation and first story beams.
- In the seventeenth cycle the maximum lateral load was 105 kN. Several cracks were observed on the infill in both stories.
- When the load was increased to 110 kN, delamination of the plaster at the foundation level was significant. Previously observed cracks widened. New cracks were observed on the first story infill.
- The delamination of plaster at the foundation level continued to widen in the nineteenth cycle. At this cycle the lateral load level was 115 kN. At the end of this

cycle the bond between CFRP and the foundation beam was only provided by the anchor dowels.

- In the twentieth negative half cycle fracture of CFRP was observed in the compression strut due to buckling at 100 kN. As a result of the compression strut failure, the load was released and the load of the tension strut increased drastically. Thus, the anchor dowels at the foundation level either failed or delaminated. Hence, the reinforced concrete frame had to carry a load which was far beyond its capacity. Crushing of concrete at the bottom of first story columns resulted in the failure of the specimen.
- As a last try, the specimen was loaded once more in positive direction to see the reserve capacity of the specimen. The load level attained was limited to 75 kN and further damage occurred in the specimen.

After the test, the plaster was removed and it was observed that almost no damage occurred in the infill. The state of SP-5 after the test is shown in Figure 4.20.

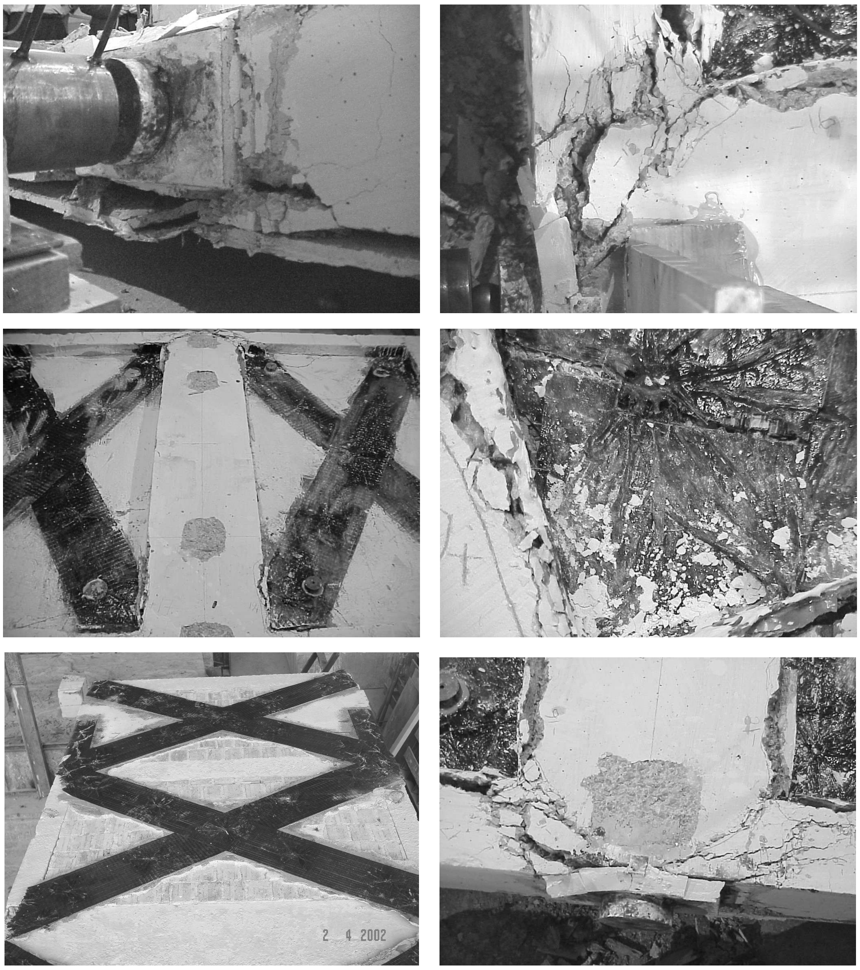


Figure 4.20 Specimen SP-5 after the Test

4.2.6 Specimen SP-6

In this specimen CFRP application was similar to the one applied to SP-5. However extra anchor dowels were provided. The axial load applied to columns (60 kN) correspond to 22% of the axial load capacity. The lateral load history is shown in Figure 4.21.

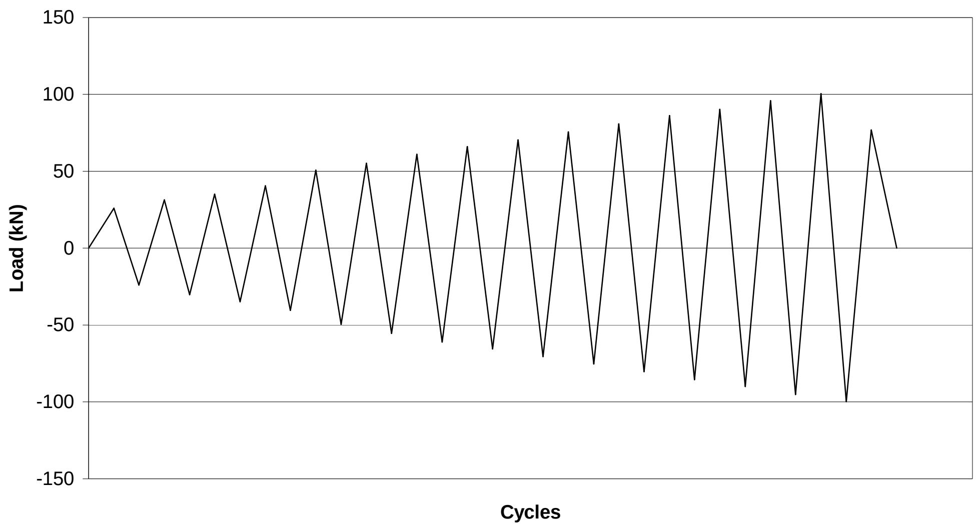


Figure 4.21 Lateral Load History of SP-6

The load displacement curves for roof and first story levels were given in Figures 4.21 and 4.22.

The experimental observations are summarized below:

- In the first seven cycles no cracks were observed on the specimen.
- In the eighth cycle maximum lateral load was 60 kN. The first hairline crack was observed in the middle of the first story column.
- In the negative ninth half cycle a crack was observed on the tension face of the first story column.
- In the tenth negative half cycle the aimed lateral load was -70 kN. In this cycle two new hairline cracks occurred.
- When the maximum lateral load was 75 kN, a crack occurred in the first story beam-column joint. Also the crack observed in the ninth cycle extended.
- In the twelfth cycle cracks occurred at the top of the first story column near the anchor dowels on the tension strut. The lateral load was 80 kN in this cycle. The cracks observed so far all remained at hairline level.
- The infill of the first story started to separate from concrete when the load was increased to 85 kN. Also cracks occurred at the ends of the foundation beam due to the high tensile force in the anchor dowels.
- In the fourteenth cycle the load was increased to 90 kN. Several cracks occurred in this cycle on different places of the specimen. The cracks were hairline cracks.
- In the fifteenth positive half cycle a crack formed in the second story joint due to the tensile stress imposed by the anchor dowels. Delamination of plaster started at the foundation level. Nevertheless, this delamination was not significant at this load level.
- When the lateral load was -100 kN, anchor dowels of the tensile strut failed suddenly and the load on that strut was released. Thus, the load level on the compressive strut

increased significantly and shear failure occurred in the first story beam-column joint. The failure of the specimen was quite sudden and brittle.

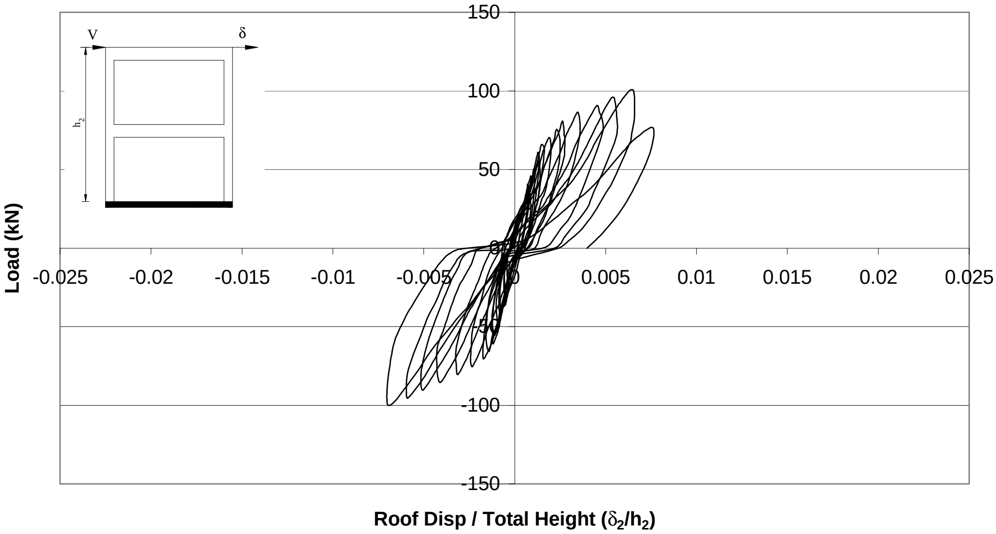


Figure 4.22 Lateral Load – Roof Drift Ratio Curves for SP-6

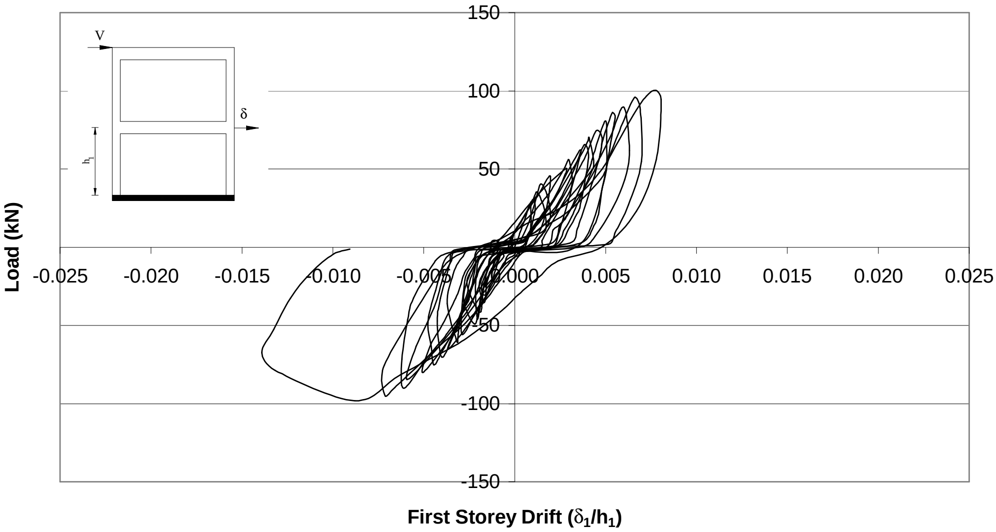


Figure 4.23 Lateral Load – First Story Drift Ratio Curves for SP-6

After this failure, the specimen was loaded in the positive direction. Since the first story joint had crushed, the anchor dowels of the tension strut could carry no load and the same failure occurred at the other joint.

Figure 4.24 presents the state of the specimen at the end of the test.

Figure 4.24 Specimen SP-6 after the Test

4.2.7 Specimen SP-7

Specimen SP-7 was also strengthened by CFRP strips arranged diagonally. In this specimen the lengths of anchor dowels at beam-column joints were increased.SP-7 was tested under the load history given in Figure 4.25. The axial load of 60 kN on each column correspond to 20% of the axial load capacity.

The lateral load-displacement curves for the second and first stories of Specimen SP-7 are given in Figures 4.26 and 4.27 respectively.

The behavior of SP-7 was very similar to SP-6. SP-7 failed by the shear failure of the first story beam-column joint.

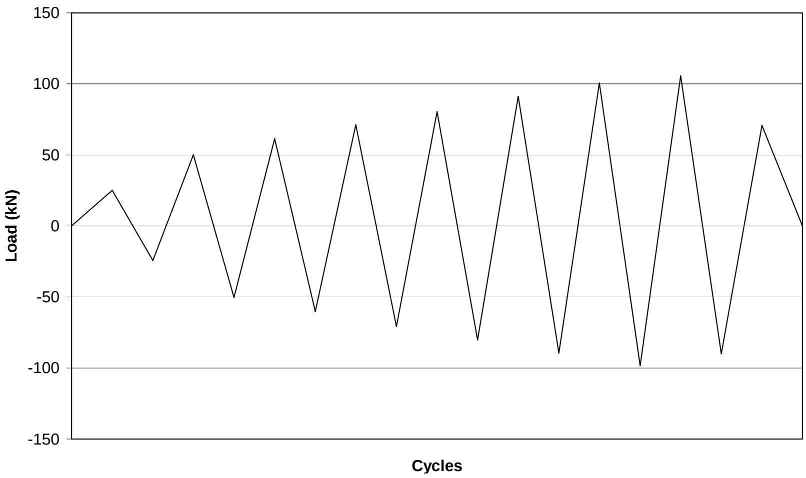


Figure 4.25 Loading History of SP-7

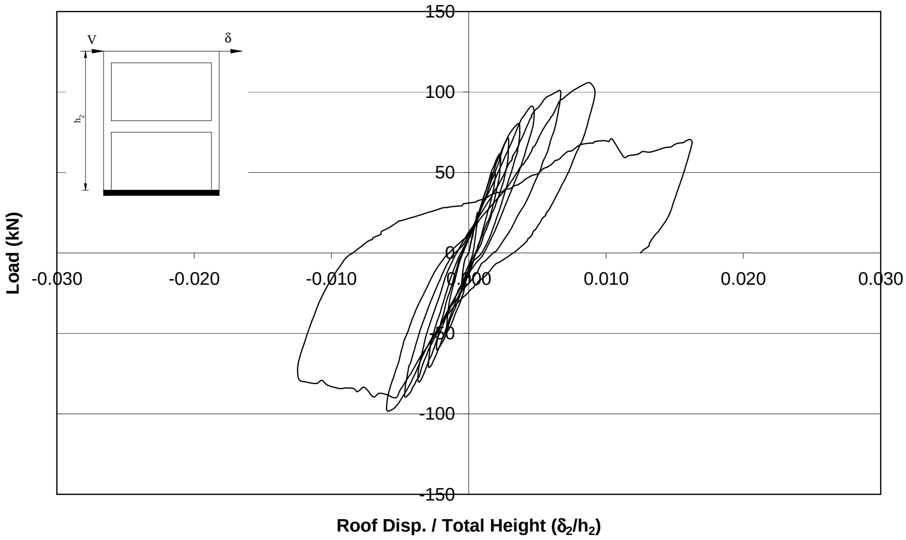


Figure 4.26 Lateral Load – Roof Drift Ratio Curves for SP-7

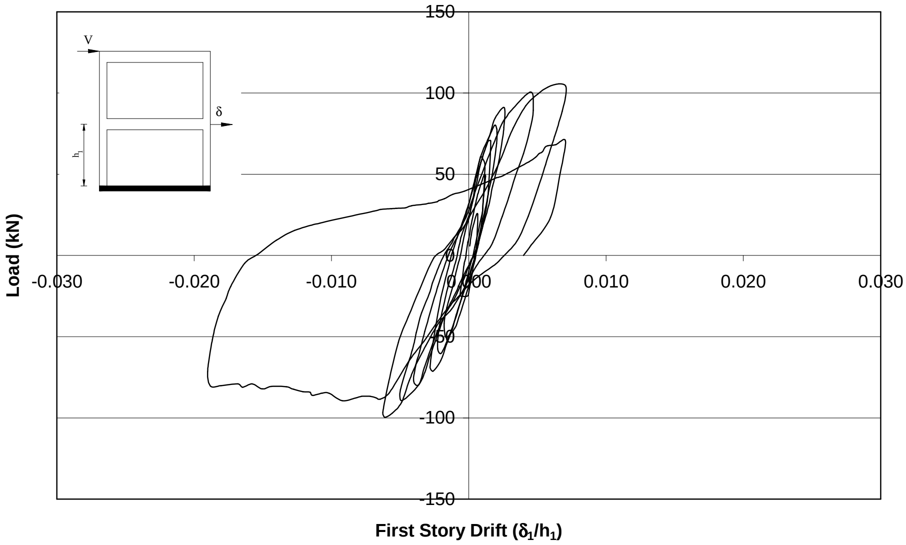


Figure 4.27 Lateral Load – First Storey Drift Ratio Curves for SP-7

5. EVALUATION OF TEST RESULTS

5.1 GENERAL

In this chapter, test results will be evaluated considering strength, stiffness, and energy dissipation.

5.2 STRENGTH

In evaluating the effectiveness of a strengthening technique or methodology, the strength increase attained is considered to be one of the most important parameters. The lateral load capacity of each specimen is given in the second column of Table 5.1. In addition, the response envelope curves are given in Figure 5.1 to enable comparison of behavior. Response envelope curves were developed by connecting the maximum values at each cycle.

Table 5.1 Summary of Test Results

Specimen	Maximum Lateral Load (kN)	Initial Stiffness (kN/m)	Total Dissipated Energy (kN-m)	Max. Interstory Drift Ratio (1st Story)
SP-1	55.8	29660	2.5	0.0170
SP-2	64.6	29520	6.1	0.0149
SP-3	65.4	21820	8.7	0.0114
SP-4	131.5	36430	11.1	0.0268
SP-5	118.8	39604	7.8	0.0175
SP-6	100.4	32624	4.2	0.0088
SP-7	105.7	24392	4.0	0.0086

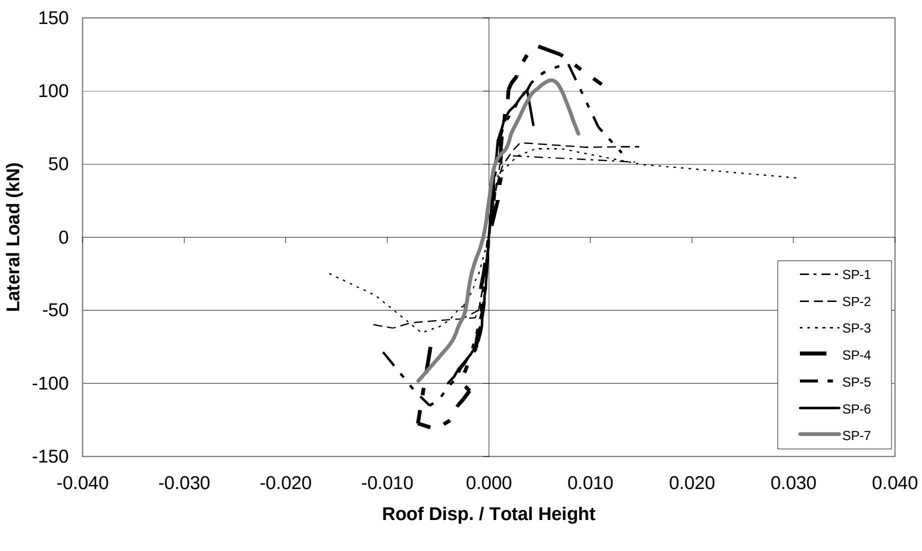


Figure 5.1 Response Envelope Curves

Table 5.1 and Figure 5.1 show that the CFRP applied to specimens SP-2 and SP-3 did not increase the strength of the structure significantly due to the debonding of CFRP at the early stages of these tests. As mentioned before, SP-4 was the optimum case for strengthening of the test specimen using CFRP. The strength of SP-4 was more than twice as compared to SP-1 in which no strengthening was carried out. It should be recalled that in SP-4 CFRP was applied to both faces of the infill. Also the CFRP was anchored to the infill and the frame members by special anchor dowels. In this specimen, since both faces are fully covered by two layers of CFRP, the economic feasibility of such a technique can be questioned.

In specimens SP-5, SP-6, SP-7, CFRP strips were placed like cross-bracing. Each strip consisted of one layer instead of two as in the case of SP-4. The strength increase in these three specimens was not as high as the one observed in SP-4. However, still the strength almost doubled as compared to the reference specimen SP-1. The amount of CFRP used in SP-5, SP-6 and SP-7 is much less than the one used in SP-4. The CFRP configuration used in these three specimens leads to an economical solution for strengthening the existing buildings with minimum disturbance to the occupants.

5.3 STIFFNESS

The stiffness of the specimen changes with structural and nonstructural damage. As mentioned before, the tests presented in this report were made under cyclic loading. Therefore the stiffness of the test specimens varied throughout the loading cycles. There are a number of methods that can be used to calculate the stiffness of a structure subjected to reversed cyclic loading. Of these methods, "peak to peak stiffness" was used in this study to calculate the relative stiffness of the test specimens. In this method, the slope of the line connecting the positive and the negative peaks of one cycle is assumed to be the stiffness of the structure for that specific cycle (Figure 5.2). To calculate the initial stiffness, the first cycles were used.

As can be seen from the third column of Table 5.1, the initial stiffnesses of the test specimens were very close to each other. Strength demand of a structure is directly related to the stiffness characteristics. Thus, it can be concluded that the proposed strengthening method increases the capacity of the structure without causing a significant increase in the capacity demand. This is an important advantage of strengthening by using CFRP.

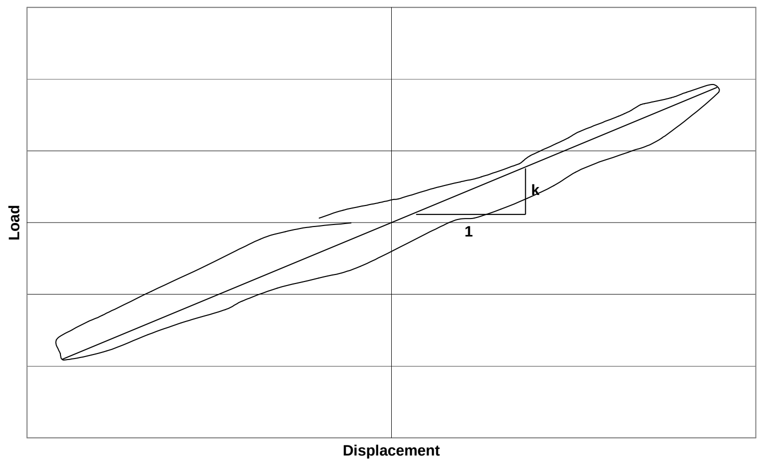


Figure 5.2 Peak to Peak Stiffness

Stiffnesses were also calculated for each cycle. Variation of stiffness with the total drift ratio is shown in Figure 5.3.

5.4 ENERGY DISSIPATION

Energy dissipation capacity of a structure is an indicator of ductility and is one of the important factors that determine the survival of the structure in a major earthquake.

The dissipated energy at each cycle was computed as the area under the load deformation curve. In Figure 5.4, values of cumulative energy dissipated by each specimen in terms of total drift ratio are presented. The total energy dissipated by each specimen is given in Table 5.1.

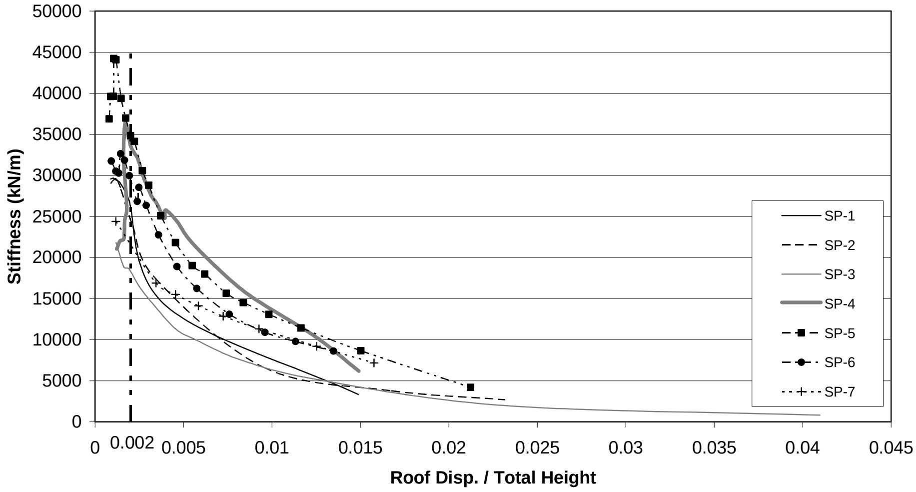


Figure 5.3 Stiffness Values for the Specimens

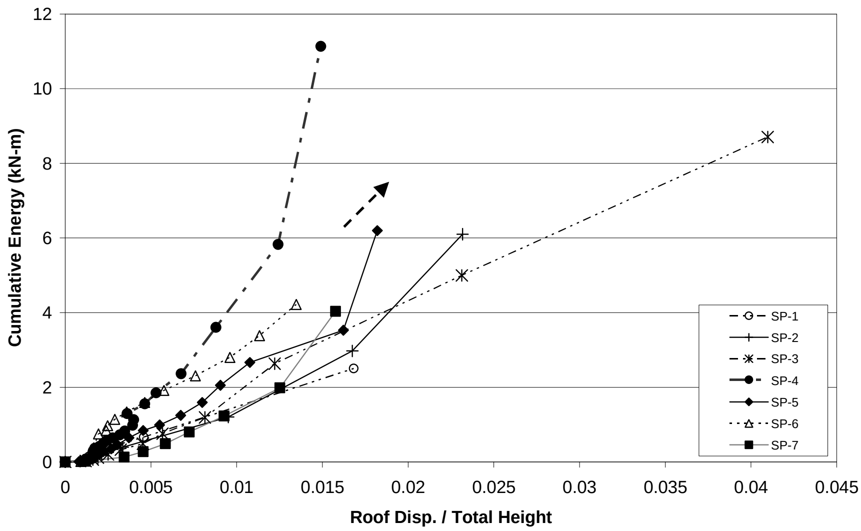


Figure 5.4 Cumulative Energy Dissipation for Test Specimens

From Figure 5.4 and Table 5.1, it can be seen that SP-3, SP-4, and SP-5 dissipated more energy than the remaining specimens. Nevertheless, the high energy dissipated in SP-3 is due to the significant damage in the second story columns. Thus, the behavior of SP-3 in terms of high energy dissipation cannot be considered satisfactory due to the unfavorable failure mechanism observed in this specimen.

The difference between the energy dissipation capacities of SP-5, SP-6, and SP-7 is mainly due to the failure mechanisms of these specimens. Failure of SP-5 was initiated by the formation of the plastic hinges at the base of the first story columns and completed by the crushing of concrete in these regions. In other words, the failure mechanism of SP-5 was relatively ductile. However, both SP-6 and SP-7 failed by a far more brittle mechanism. Since the bases of the first story columns were confined using CFRP, the failure zone moved to the next weak link which was the first story beam-column joints. Both specimens failed due to the sudden shear failure at these joints.

5.5 INTERSTORY DRIFTS

Interstory drift can simply be defined as the relative displacement between two consecutive floors divided by the floor height. Interstory drift is generally accepted as a measure of the nonstructural damage. Turkish Earthquake Code limits the interstory drift to 0.0035 based on elastic analysis [1].

In Table 5.1 the maximum interstory drift values for each specimen are given. In case of SP-3 and SP-4, infills sustained significant damage during the test. However, although interstory drift value for first floor of SP-5 in the last cycle increased up to 0.017, no significant damage was observed in the infill walls. This can be considered as a significant contribution of CFRP.

5.6 OVERALL COMPARISON

In this section, the results of all the seven tests carried out throughout this study are compared in terms of strength, stiffness and energy dissipation characteristics. Table 5.1 and Figure 5.1 summarize the results of these tests.

Table 5.1 and Figure 5.1 reveal that SP-4 and SP-5 were superior to the remaining four strengthened specimens in terms of lateral load capacity, stiffness and ductility. Although SP-4 behaved in a better manner than SP-5, the amount of CFRP reinforcement in this specimen was about four times more than that used in SP-5. Thus, CFRP detailing used in SP-5 seems to be the most efficient one among the other tested CFRP details as far as economy and behavior are concerned.

6. ANALYTICAL STUDIES

6.1 GENERAL

In this chapter, the analytical studies carried out to predict the observed behavior of the test specimens and the case study analyses performed on a real building are presented in detail. It must be noted that the specimens used in this study had CFRP reinforced hollow clay tile walls. In the literature there is no available material model to incorporate such members in the analysis of structural systems. Therefore in the first part of this section, attempts towards the derivations of realistic analytical models that describe the behavior of such materials under compression and tension will be presented. In the derivation of these models the ultimate goal was to simulate the experimentally observed response as accurately as possible. The second part involves the case study made on a real building, DBI building in Dinar [11]. In both parts Drain 2DX software was used [12]

6.2 MODELING AND PUSHOVER ANALYSES

6.2.1 Mathematical Model

Figure 6.1 shows the mathematical idealization of the one-bay-two-story infilled frame tested in the experimental phase of this study. In this figure, members 1 through 6 are reinforced concrete frame members. The research presented in this report and past research conducted in METU [4] and elsewhere revealed that brick infill walls contribute significantly to the stiffness and the load carrying capacity of the framed structures. Therefore in the structural analysis, the presence of infill walls should be taken into account by using realistic material models. This is especially important in the case of the reported research. For this reason, members 7 through 10 are introduced in the mathematical model. These members are diagonal struts modeling the presence of CFRP reinforced and/or unreinforced hollow clay tile walls.

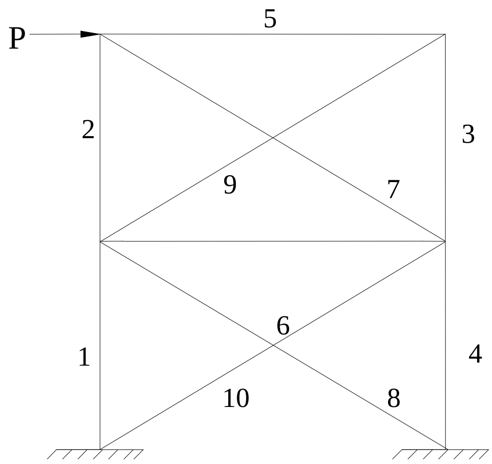


Figure 6.1 Modeling of Test Specimens

In the model the unreinforced infill panels are considered to act as compression struts, members 7 and 8 in Figure 6.2. The tensile strength of the unreinforced brick infill panels is neglected in the analyses. Thus in the analysis of frame with an unreinforced brick infill panel, the model consists of 8 elements, namely elements 1 through 8. In this analysis, the tension struts are not considered as effective members. On the other hand, the brick infill panels become very effective tension resisting elements when reinforced with CFRP. Therefore, in the analysis of those specimens where CFRP reinforcement is used, members 9 and 10 are introduced in the mathematical model.

The pushover analyses of the test specimens are performed by using a nonlinear structural analysis program, which is called Drain 2DX [12]. The pushover analyses require the definitions of nonlinear material models for each member used in the structural system.

6.2.2 Constitutive Model for the Plastered Brick Infill Panel

The first step in this procedure was the evaluation of the moment-curvature relationships of the frame members (members 1 to 6 in Figure 6.1). Knowing the geometry of the cross-sections, the properties of the materials used and the axial load imposed on each member, sectional analyses based on the internal force equilibrium were performed and the true nonlinear moment-curvature relationships of all frame members were obtained. These relationships were then idealized as bilinear relationships with one branch connecting the origin to the yield point of the section and the other being the fully plastic branch extending to infinity thereafter.

As mentioned before, the unreinforced infill panels are assumed to resist compressive forces only. It was decided to use a bilinear constitutive model to represent the axial response of the compressive struts (members 7 and 8 in Figure 6.1). The model requires definitions of the initial stiffness, the ultimate strength and the post-peak stiffness of the axial compression versus axial shortening relationship.

A trial-error procedure was utilized to determine the properties of the strut used for modeling the infills of the test specimens. This procedure requires initial estimates of the three parameters mentioned in the previous paragraph. With the initial estimate of the compressive strut model and the idealized bilinear moment-curvature relationships of the frame member a series of pushover analyses are initiated to predict the experimentally observed behavior of Specimen 1. It should be noted that Specimen 1 is the reference specimen where only plastered brick infill panels were used in the design. Iterations were made by assigning different values to the initial stiffness, ultimate strength and the post-peak stiffness of the compression strut each time. The iterations were repeated until good agreement between the analytical and the experimental response curves was obtained.

The uniaxial constitutive model for the compression struts that has been thus obtained and the results obtained using these models are presented in Figures 6.2 and 6.3, respectively.

6.2.3 The Constitutive Model for the Tension Strut

The CFRP fabric used in the experiments had fibers in one direction only. The CFRP reinforcement was applied as 200 mm wide strips extending along the main diagonals of the brick infill panels. Only one layer of CFRP fabric is applied in each diagonal. As explained in Section 2.4, the CFRP reinforcement is applied in 4 stages. Together with the chemicals used in the application (i.e. the substrate, putty, and the epoxy) the thickness of the composite CFRP layer was approximately 1 mm. Five 25 mm wide tensile coupon specimens were taken from Specimen 4 and tested under uniaxial tensile force to determine the tensile strength of the composite CFRP reinforcement. Results based on 5 coupon tests indicated that the average tensile strength of the composite CFRP reinforcement can be taken as 1000 MPa. It should be kept in mind that, the tension struts (members 9 and 10 in Figure 6.1) are basically formed by the combined action of (a) the brick infill panel, (b) 10 mm plaster applied on both faces of the brick infill and (c) the composite CFRP reinforcement. Although the average tensile strength of the composite CFRP layer obtained from tensile coupon tests was very high, test results indicated this value cannot be used as a representative value for the tensile strength of the tension strut. Observations made during the tests indicated that the maximum force that can be transmitted by the tension strut was limited by the behavior of the anchor dowels that connect the tension strut to the neighboring frame members. Therefore the authors of this report believe that the realistic analysis of the test specimens can only be made once the integral action of the tension strut and its connectors (i.e. anchor dowels) are taken into consideration.

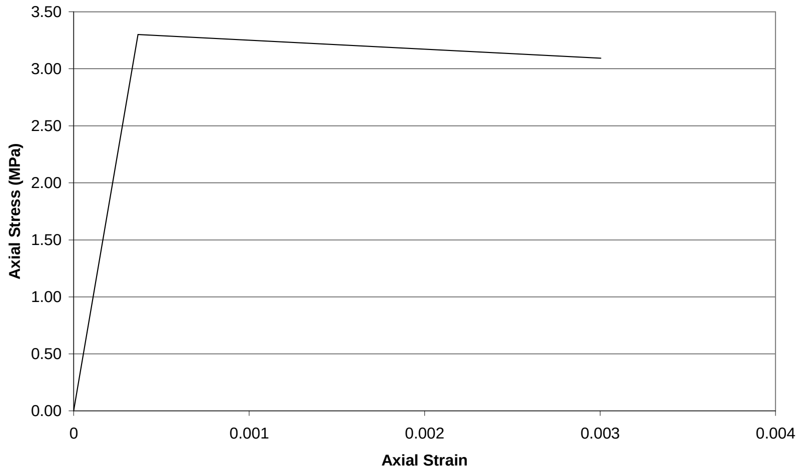


Figure 6.2 Strut Model of the Infill

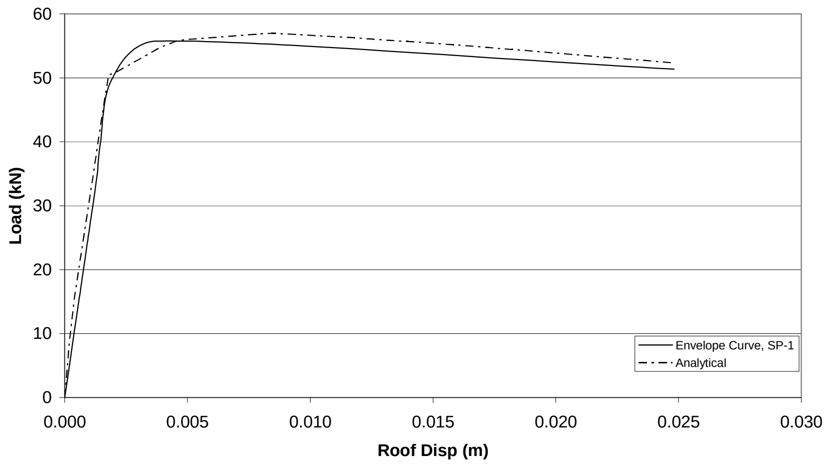


Figure 6.3 Comparison of Experimental and Analytical Results of SP-1

In order to model this behavior a trial and error approach similar to the one explained in Section 6.2.1 is employed. To represent the behavior of the tension strut, a tri-linear behavior model is found suitable, Figure 6.4. In this model point “A” is named as the first yield in the composite material. This is the point where the stiffness of the tension strut drops approximately by 50 percent. Point “B”, on the other hand, marks the first fracture in the anchor dowels. This point actually corresponds to the peak resistance displayed by the tension strut. Thus, the model requires the definitions of the following parameters (a) the initial stiffness, (b) the post-yield stiffness, (c) the post-peak stiffness values, (d) the first yield load and (e) the first fracture load of the tension strut.

A series of pushover analyses were performed to predict the experimentally observed behavior of Specimen 5. In these analyses, the models explained in Section 6.2.1 were used for the frame members and the compression struts. The iterations were made on the selected model parameters explained in the previous paragraph. These five parameters were adjusted until a reasonable agreement between the experimentally observed response and the analytically prediction was reached.

Figure 6.4 shows the uniaxial constitutive model for the tension strut that has been thus obtained. The pushover analysis results along with the experimentally obtained response curve of the Specimen 5 are presented in Figure 6.5.

It should be noted that this model includes the characteristics of CFRP, plaster, infill, and the anchor dowels into account since it is directly derived from the experimental results.

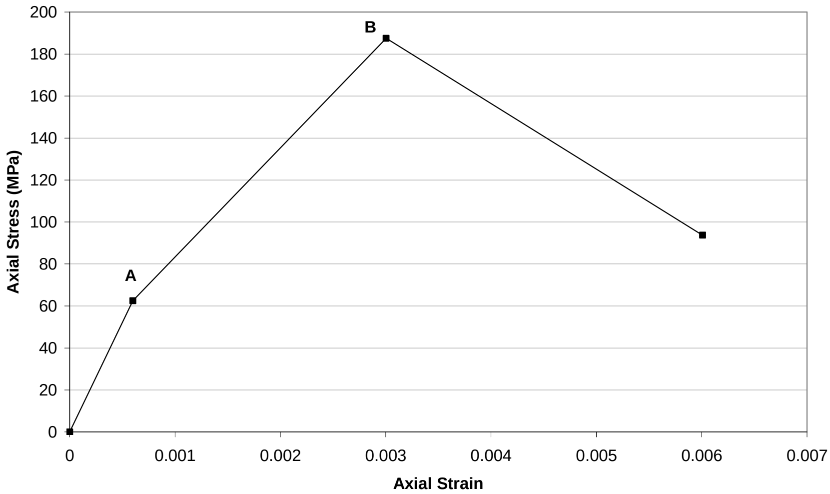


Figure 6.4 Strut Model Developed for the Composite Material

6.3 CASE STUDY

In this part of the study, it was intended to develop design criteria for the strengthening of reinforced concrete structures using CFRP reinforcement as discussed in the previous sections. In developing these criteria, the main objective was limited to the prevention of collapse.

The case study analyses are made on a building from real life. This building was moderately damaged after the 1995 Dinar earthquake. It was later rehabilitated by a team of experts from the Middle East Technical University, [11]. For this reason, the building was very well documented, and the information on its damage status after the earthquake, the structural layout, the dimensions and the amount of reinforcements of structural members and the material properties were all known.

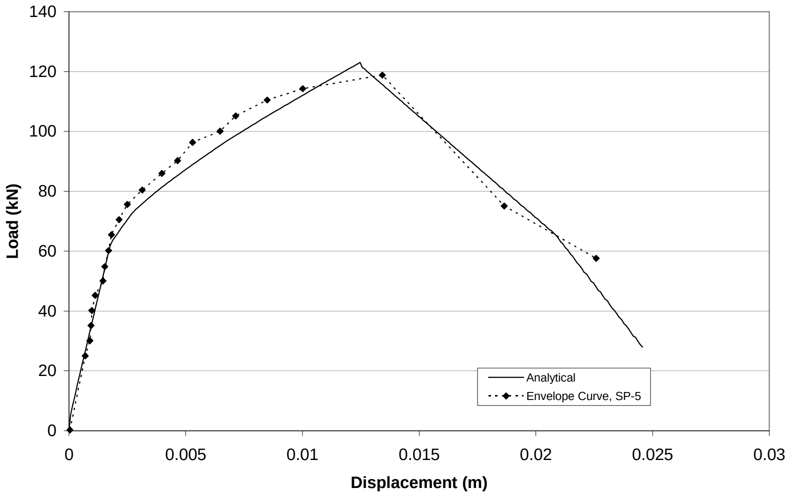


Figure 6.5 Comparison of Experimental and Analytical Results of SP-5

The building is known as the DBI building in Dinar, which is a four story framed structure. The floor area at the base of the building is approximately 310 m^2. The height of the first floor is 3.8 m and the height of the remaining three floors is 3.5 m. Moreover, the amount of brick infill walls at the first floor was much less as compared to the others resulting in a soft story problem. Observations made by the METU-EERC teams revealed that the structure had the typical deficiencies observed in Turkey like inadequate confinement, low lateral rigidity, beams stronger than columns and low concrete strength. The structure was originally rehabilitated by introducing reinforced concrete infill walls. The total area of reinforced concrete infill walls provided in each direction was 0.93% of the floor area at the base. The overview and the original strengthening scheme applied on the floor plan of the building are given in Figures 6.6 and 6.7.

Figure 6.6 Overview of DBI Building

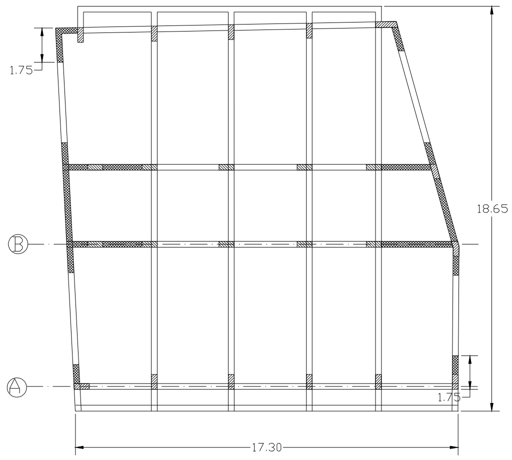
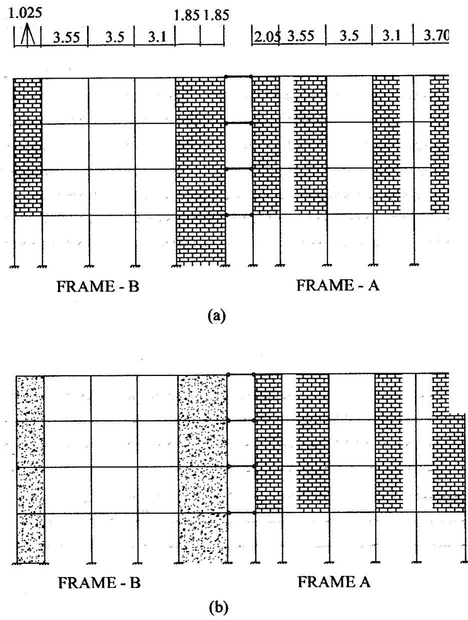


Figure 6.7 Applied Strengthening by using RC Infills on First Floor Plan

The pushover analyses of the building were made by using the DRAIN-2DX software. Some simplifying assumptions were made in the modeling. These are, namely:

- The constitutive model developed in Section 6.2.1 describes the behavior of in-situ response of the brick infill panels of the real structure.
- The constitutive model developed in Section 6.2.2 will be applicable in cases where CFRP reinforcement is used.
- The floor plan was assumed to be symmetrical. Hence, only Frame A and Frame B shown in Figure 6.7 were included in the mathematical model. These two frames were connected to each other by rigid links. The models of the DBI Building prior to and after the original strengthening by introducing RC infill walls are shown in Figure 6.8.



Amount of RC Infills:

0.93 % of the Floor

Area

Figure 6.8 Modeling of (a) Original (b) Strengthened Structure

In the case study analyses, four different CFRP strengthening schemes were considered. In the first one, only the infills which were replaced by RC infill walls in the original design were strengthened by CFRP reinforcement. Thus, the amount of brick infills strengthened was 0.93% of the floor area at the base, Figure 6.9.

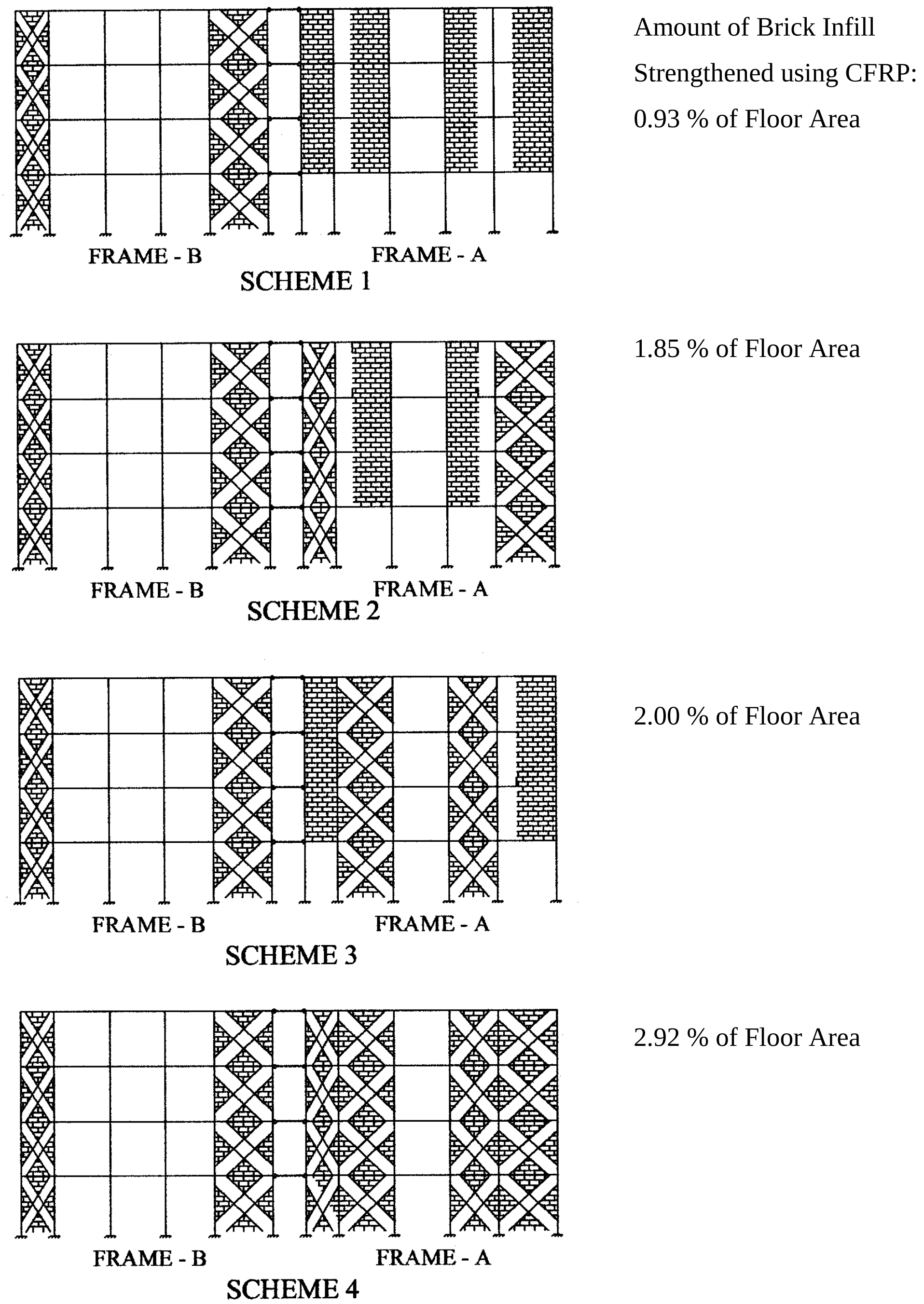


Figure 6.9 Strengthening Schemes using CFRP

In the second and third schemes, the amount of infill walls that were strengthened by using CFRP reinforcement was doubled. In Scheme 2, in addition to the strengthening applied in Scheme 1, the brick infills in the two outermost bays of Frame A were strengthened with

CFRP reinforcement. The amount of walls strengthened in this scheme was 1.85% of the floor area at the base. In Scheme 3, instead of strengthening the two outermost bays, the infills of the neighboring inner bays of Frame A were reinforced with CFRP strips. Brick infills constituting 2.00% of the floor area at the base were strengthened in Scheme 3. Scheme 4 demonstrates the ultimate strengthening scheme in which the infill walls in all bays of Frame A were strengthened (2.92% of the floor area at the base). Figure 6.9 shows all four schemes considered in the analyses.

In developing the strengthening schemes, architectural requirements were ignored. Moreover, the infill walls were assumed to be extending from one column to the other in each bay. That is, window or door openings in the infill walls were ignored in the analyses.

Pushover analyses were carried out for; (a) CFRP strengthened structure (4 patterns), (b) structure strengthened with RC infills and (c) structure with no strengthening (bare frame and brick infilled frame). Cracked moment of inertia values were used for the concrete members. In modeling the structure with reinforced concrete infills, the columns between which RC infill walls had been introduced were assumed to behave monolithically together with the infills.

It is clear that the computer program used in the pushover analyses cannot take the deficiencies, like inadequate confinement and inadequate splice length, into account. However, the DBI building is known to have such deficiencies. Past research in Turkey and elsewhere revealed that the capacity of a frame having these deficiencies may be as low as 70 percent of a structure having no deficiencies, [5, and 13]. Thus, the column capacities found from member analysis were reduced by 30 percent in order to take the negative effect of the system deficiencies into account in the analyses. Pushover curves for the four CFRP strengthening schemes are shown in Figure 6.10. It is clear from these curves that Scheme 4 increases the load carrying capacity of the structure more than the other schemes. However, this strengthening scheme seems practically impossible to apply since it requires strengthening of a large amount of brick infills (2.92% of floor area). Strengthening schemes 2 and 3 seems to be the best alternatives. In these two cases, a considerable amount of strength increase is reached with a reasonable amount of strengthening. The amount of brick infills to be strengthened in these two schemes was 1.85 and 2.0 percent, respectively.

In Figure 6.11 the pushover curves for schemes 2 and 3 were compared with the pushover curves for bare, hollow clay tile infilled, and RC infilled structures. Although the performance of frames strengthened using CFRP were not as favorable as the behavior of the RC infilled frame, the amount of increase in strength compared to bare and brick infilled frames seemed to be significant as far as prevention of collapse of the structure was concerned.

Pushover curves show that the load carrying capacity of the structure can be increased to satisfactory levels by CFRP strengthening. When hollow clay tile infills constituting approximately 2.0 percent of the floor area were strengthened using CFRP, the lateral load carrying capacity of the frame was increased by approximately 100 percent as compared to the bare frame and by 60 percent as compared to the hollow clay tile infilled frame. However, once the maximum load level was reached for CFRP strengthened cases, the capacity of the frame starts to decrease drastically with increasing displacements.

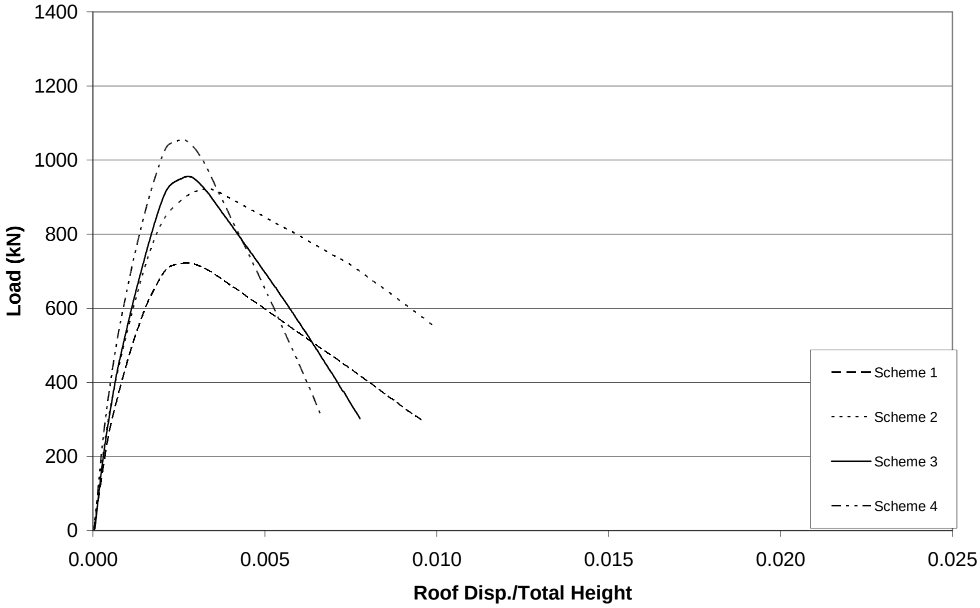


Figure 6.10 Pushover Curves for Different Strengthening Schemes

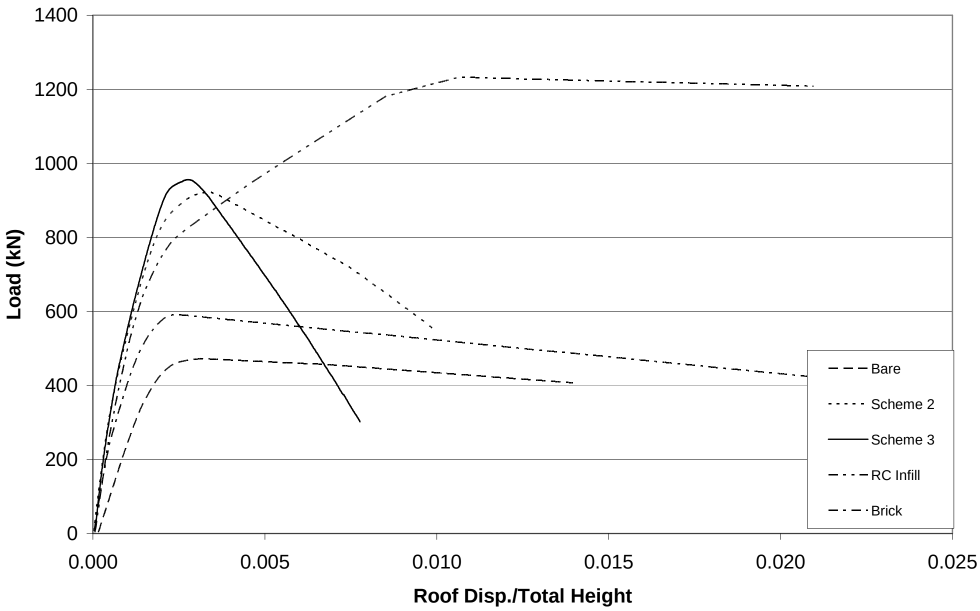


Figure 6.11 Pushover Curves

In addition to the pushover analyses, time history analyses were carried out for both unstrengthened and strengthened structures. The ground motion applied to the building was the 1999 Marmara Earthquake, Sakarya East-West component with a peak acceleration of 0.407g and duration of 65.7 seconds (Figure 6.12). As a result of these analyses, in addition to hysteretic base shear versus roof displacement relationships, the progressive plastic hinge formation patterns were also obtained.

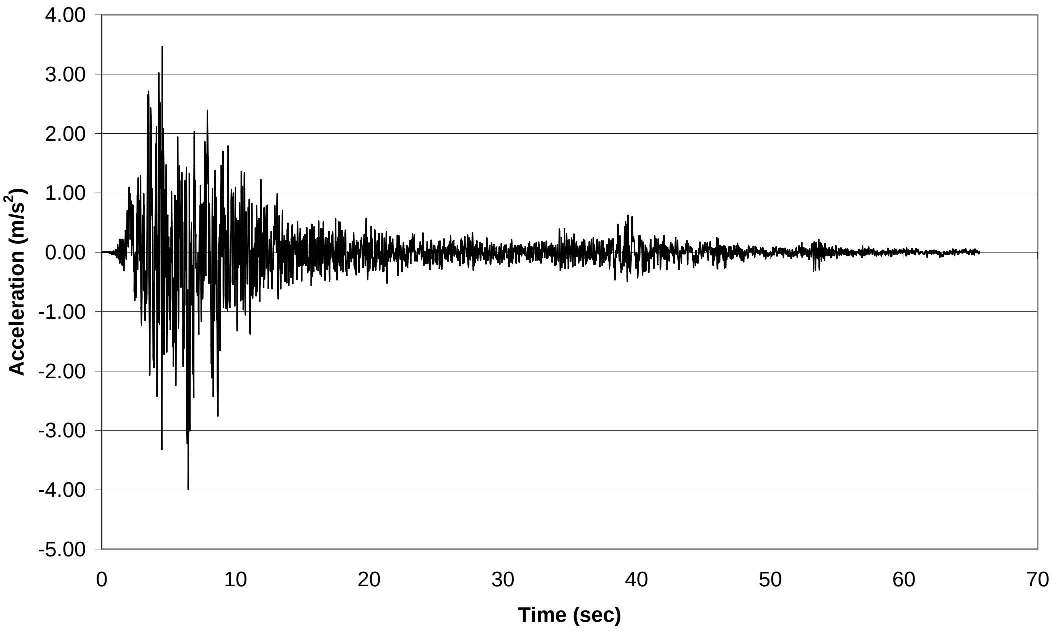


Figure 6.12 1999 Marmara EQ, Sakarya E-W Component Record

In Figures 6.13 to 6.19, the critical load stages obtained from the time history analyses are marked on the pushover analyses results.

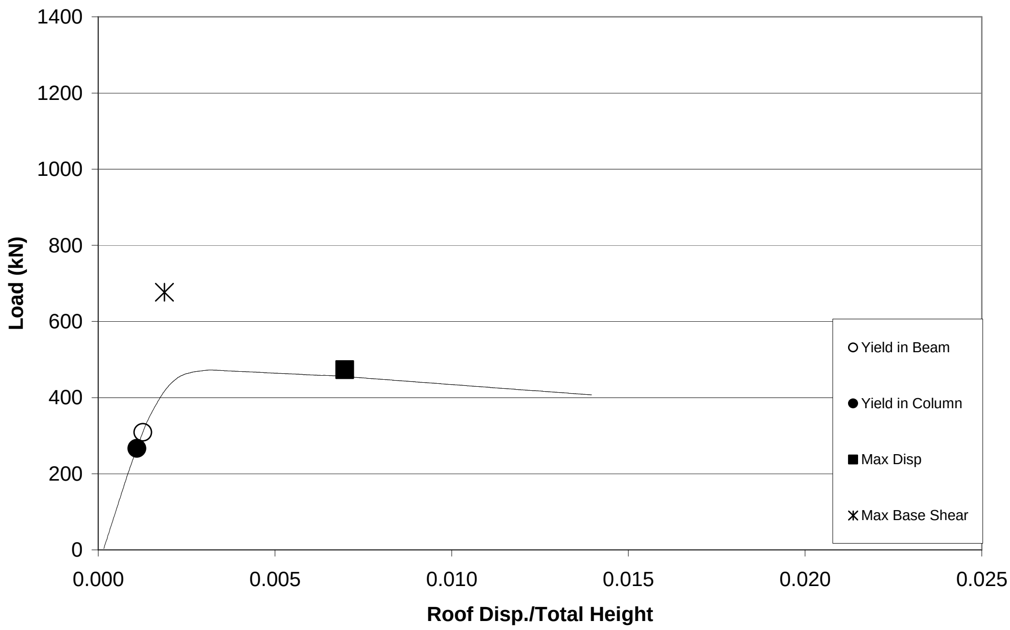


Figure 6.13 Bare Frame

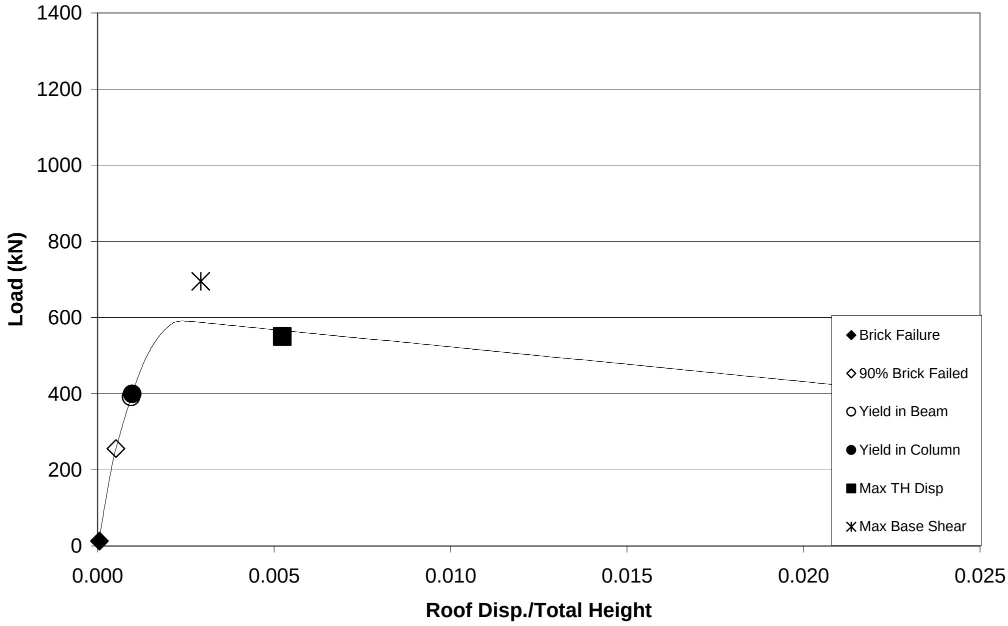


Figure 6.14 Brick Infilled Frame (no strengthening)

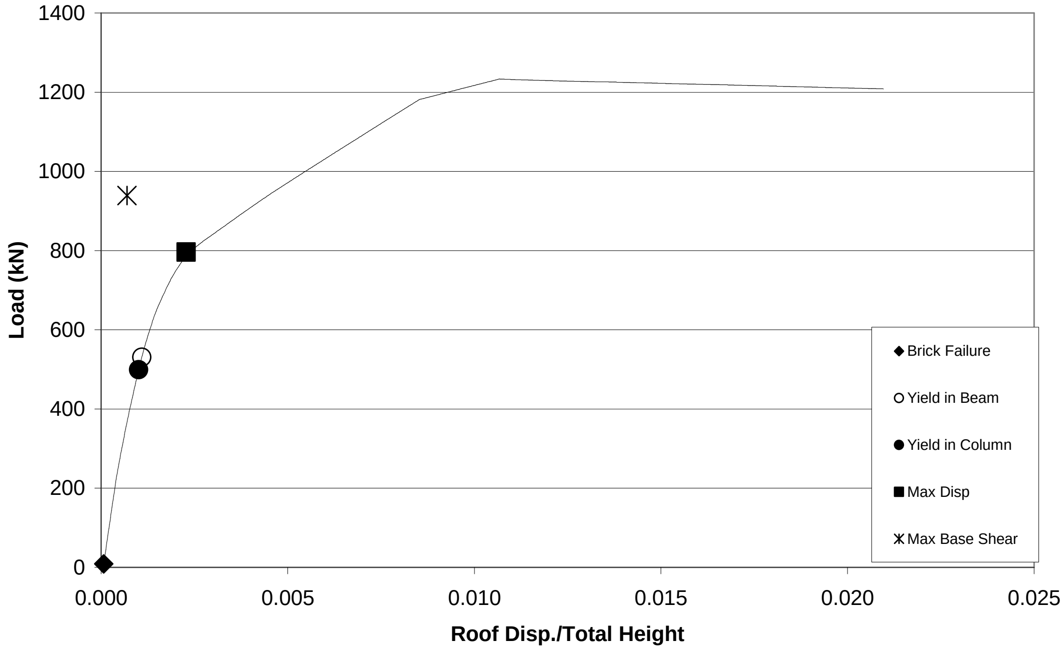


Figure 6.15 RC Infilled Frame

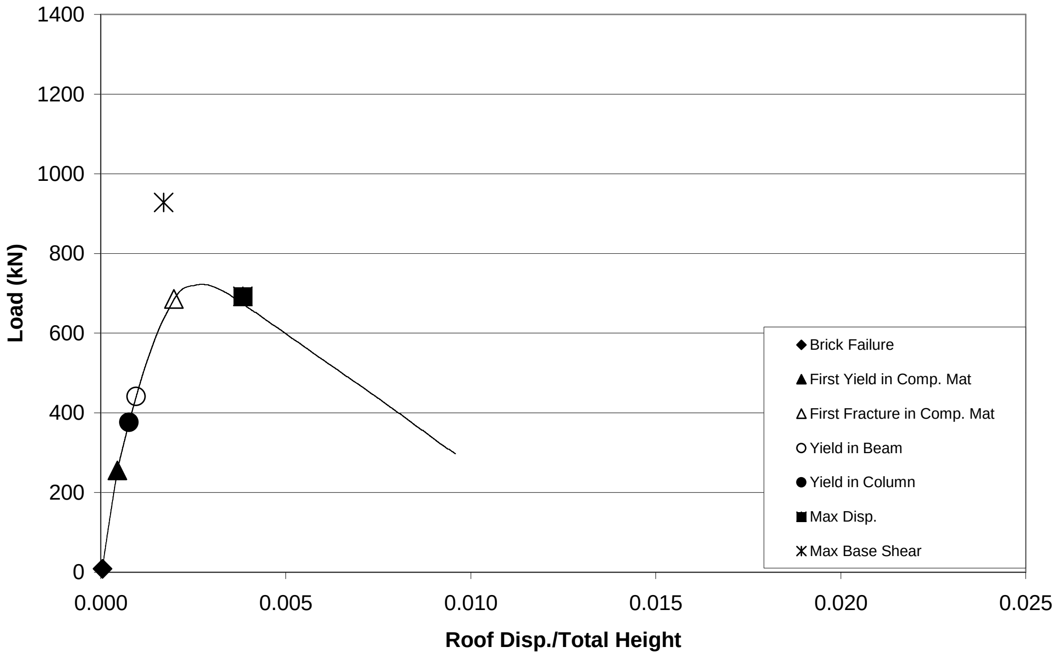


Figure 6.16 Frame Strengthened by CFRP using Scheme 1

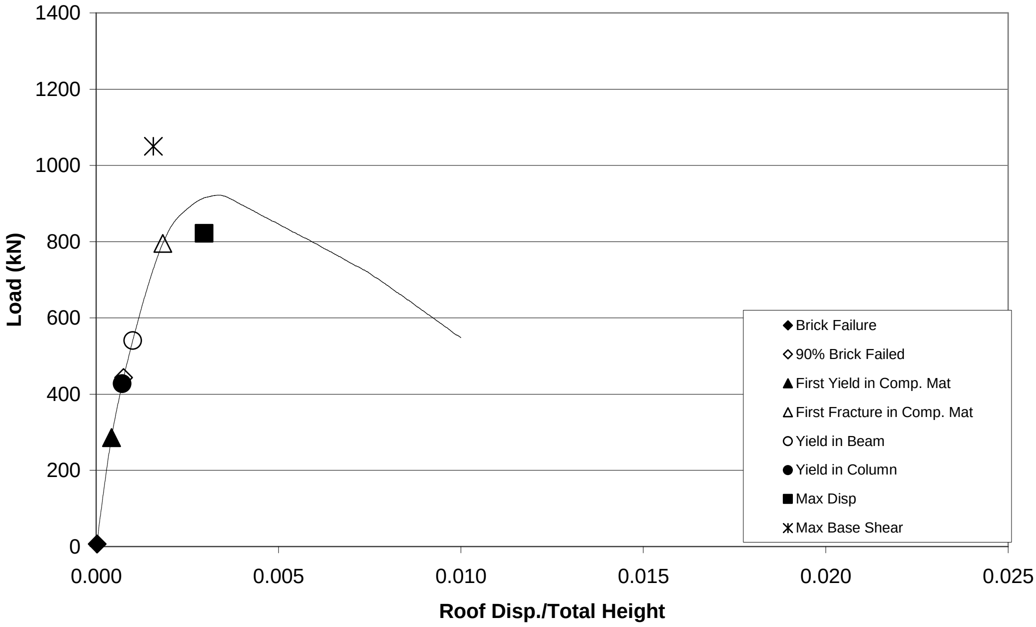


Figure 6.17 Frame Strengthened by CFRP using Scheme 2

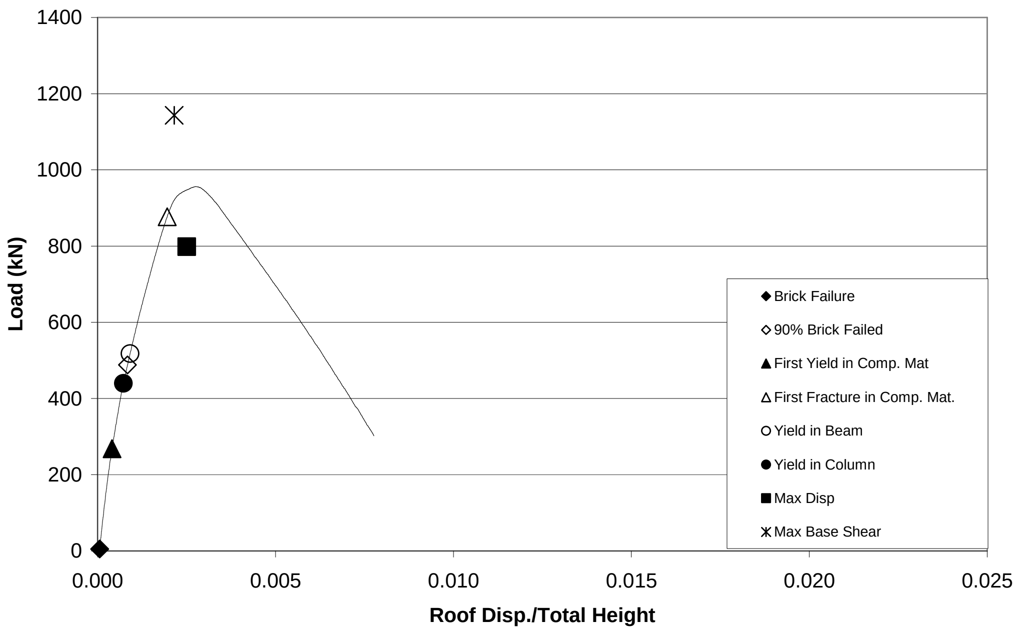


Figure 6.18 Frame Strengthened by CFRP using Scheme 3

It must be noted that the points marked as "First Yield in Composite Material" in Figures 6.16 to 6.19 correspond to the first stiffness change in the composite material (point A in Figure 6.4) and "First Fracture in Composite Material" correspond to the second stiffness change (point B in Figure 6.4)

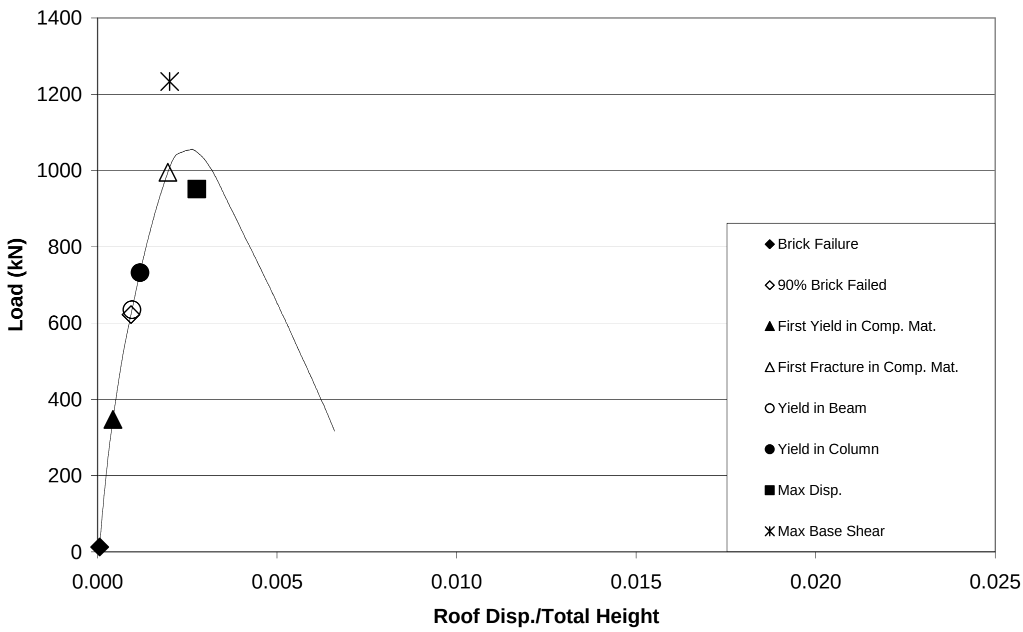


Figure 6.19 Frame Strengthened by CFRP using Scheme 4

The displacement demand under a sample major ground motion was greatly reduced when the infill walls of the structure were strengthened by using CFRP reinforcement. The amount of reduction depends directly on the amount of infills strengthened. In case of Scheme 1, in which the area of CFRP strengthening was nearly 1.0 percent of the floor area, the reduction in displacement demand was 27.4 percent as compared to the brick infilled frame. Although this can be considered as a significant reduction, both the strength and displacement demand of this scheme was found to be insufficient. When the strengthened area was increased to 1.85 percent (Scheme 2), the reduction in displacement demand became 43 percent. This strengthening scheme seems to be effective in terms of the strength gain and reduction in displacement demand. The total amount of brick infill walls that were strengthened in Schemes 2 and 3 were almost equal (1.85 percent in Scheme2 and 2.0 percent in Scheme 3). This led to similar decreases in displacement demand and gains in strength for these two cases. However, considering the economy and the ease in construction, Scheme 2 seems to offer a more favorable alternative for the solution of the problem. On the other hand, apparently the gain in load carrying capacity and the reduction in displacement demand provided by Scheme 4 were both very high. However, this regime requires an excessive amount of retrofitting intervention, i.e. the brick infill walls to be strengthened occupy nearly 3.0 percent of floor area, and it seems to be unfeasible from practical and economical viewpoints.

The maximum shear strength demand in Figures 6-15 to 6.19 seems to exceed the capacity of the strengthened infilled frames. In case of Scheme 2 this exceedance is about 30 percent. It must be noted that the pushover analysis is a static analysis and it provides a conservative approach for the evaluations of the earthquake response of structures. The resistance of the structure under dynamic loading is expected to be higher than the resistance under static loading. For this reason, under dynamic effects, 30 percent exceedance of the static capacity is not considered as an unfavorable feature of the proposed schemes.

In order to be evaluate the response of the strengthened structure under the given earthquake motion more precisely, the hysteretic base shear versus top story drift ratio values were calculated and plotted at each time step. These points are shown on the related pushover curves in Figures 6.20 to 6.24.

Although none of the proposed schemes could reach the favorable behavior of reinforced concrete infilled frames, Schemes 2 and 3 gave satisfactory results in terms of both lateral load capacity and displacements. In Figures 6.17 and 6.18 it can be seen that the maximum base shear due to the given ground motion exceeded the capacity of the structure. However, Figures 6.21 and 6.22 shows that the capacity of the structure was exceeded by a few times. It must be noted that the computer program used in the analyses is a 2-D program in which the response of reinforced concrete beams and columns were idealized by elasto-plastic moment-curvature relationships. Thus, some important features of the true non-linear response of the reinforced concrete structures cannot be incorporated in the analyses. Among these, stiffness softening due to flexural cracking, reserve strength beyond yielding and the redistribution of forces between the frame members were not taken into consideration. It is a known fact that each one of these features does contribute the ultimate strength of a RC structure. Thus, the response improvement obtained by using Scheme 1 and Scheme 2 can be considered as satisfactory as far as collapse prevention is concerned.

The results of these analyses are summarized in Table 6.1.

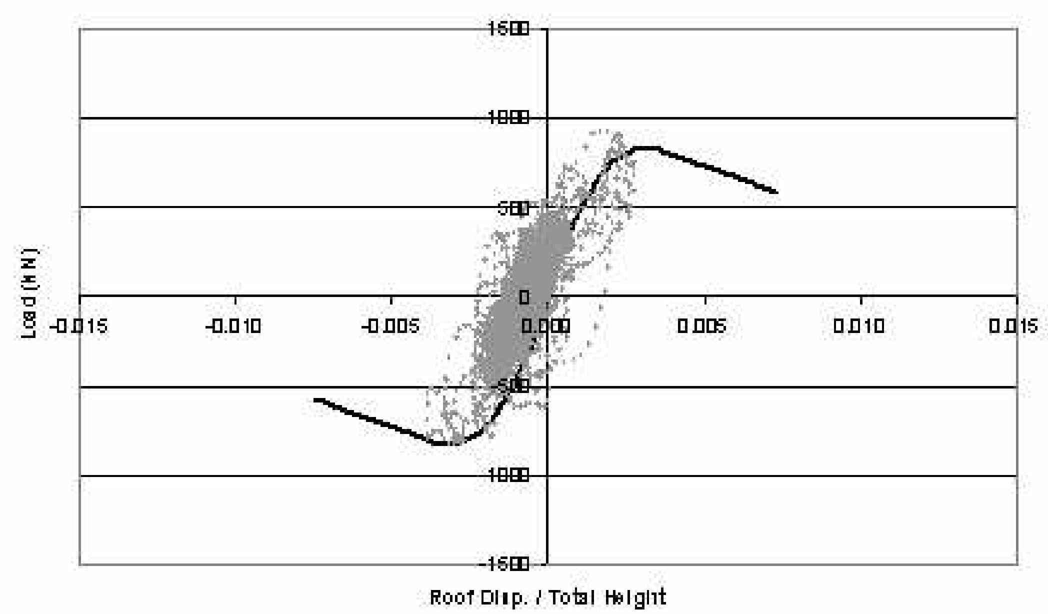


Figure 6.20 Time History Analysis and Pushover Curve for Scheme 1

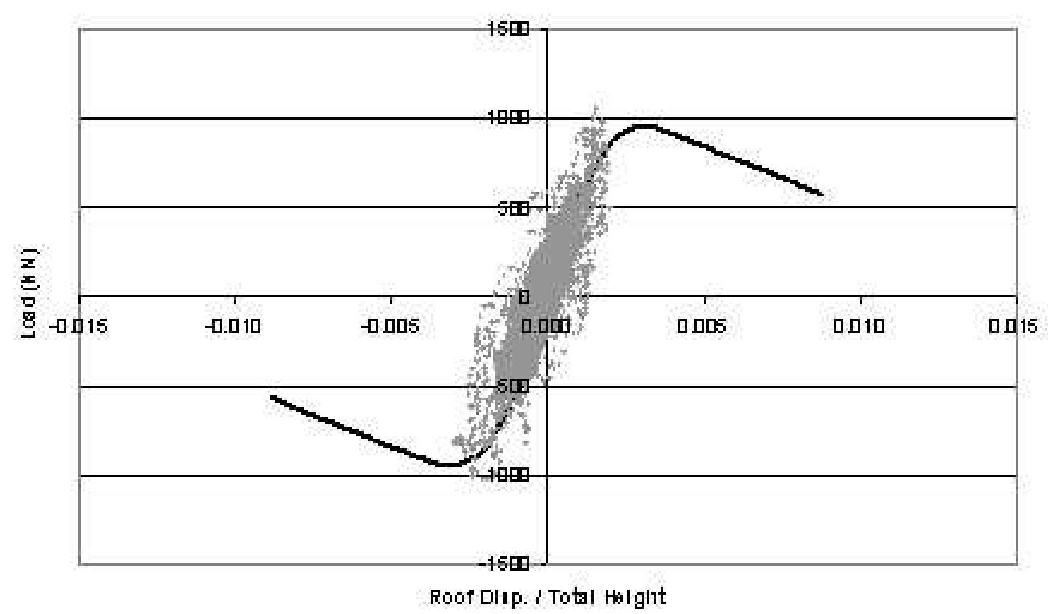


Figure 6.21 Time History Analysis and Pushover Curve for Scheme 2

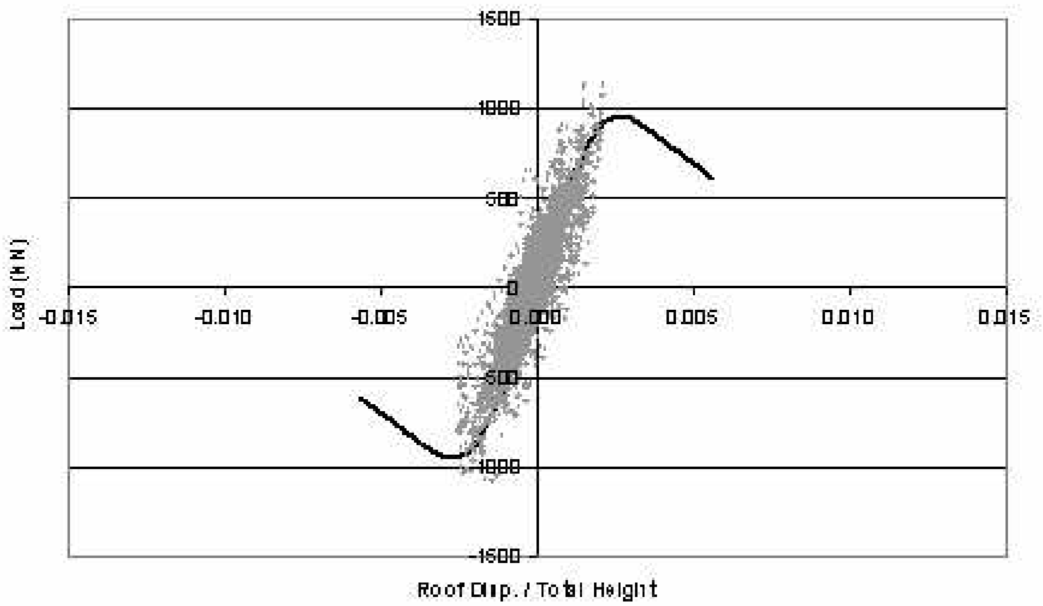


Figure 6.22 Time History Analysis and Pushover Curve for Scheme 3

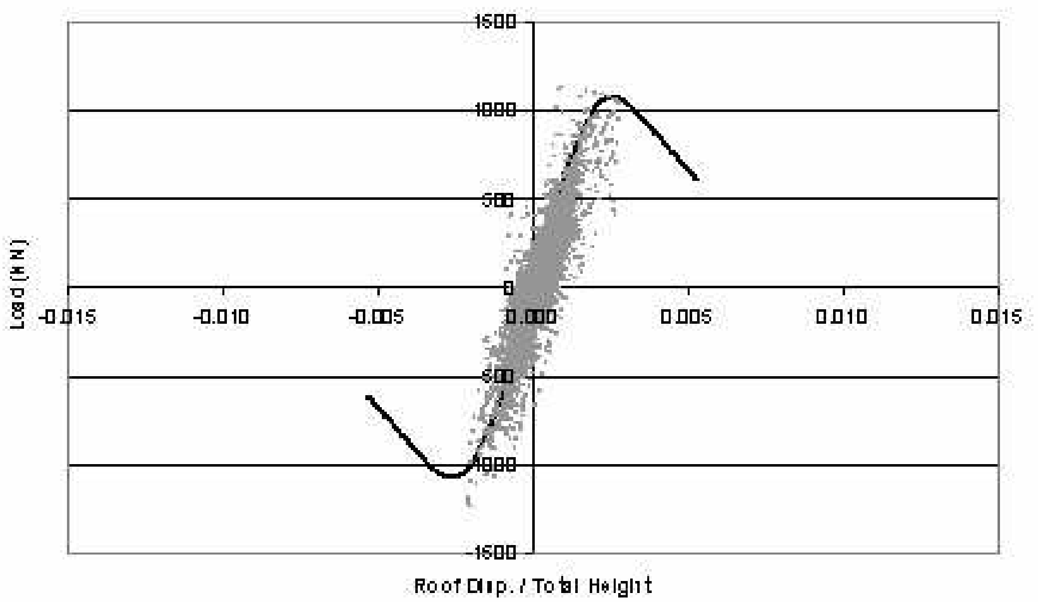


Figure 6.23 Time History Analysis and Pushover Curve for Scheme 4

Table 6.1 Results of Case Study

	Strengthened Area (% of Floor Area)	Ultimate Load (kN)	Stiffness (kN-m)	First Failure in Brick (kN)	90 % of Brick Infills Failed (kN)	First Yield in Comp. Mat. (kN)	First Yield in Beam (kN)	First Yield in Column (kN)	First Fracture in Comp. Mat. (kN)
Bare Frame	-	472.7	14657	-	-	-	307.4	263.4	-
Brick Infilled Frame	-	590.9	19508	5.96	254.3	-	390.9	399.4	-
RC Infilled Frame	0.93	1233.1	26171	7.12	-	-	527.8	495.5	-
Scheme 1	0.93	722.7	23913	8.90	-	255.6	441.3	376.2	685.8
Scheme 2	1.85	922.2	29362	1.58	443.6	284.2	539.2	427.0	795.0
Scheme 3	2.00	955.9	30461	3.58	486.4	265.2	515.9	439.6	876.7
Scheme 4	2.92	1055.7	34800	6.54	621.9	343.9	632.5	730.1	994.4

Table 6.2 Results of Case Study (Continued)

	Strengthened Area (% of Floor Area)	Max Base Shear		Max Displacement		Average Shear Stress in Columns (MPa)
		Force (kN)	Drift	Force (kN)	Drift	
Bare Frame	-	677.0	0.0019	473.3	0.0070	0.26
Brick Infilled Frame	-	695.1	0.0029	550.2	0.0052	0.33
RC Infilled Frame	0.93	939.2	0.0007	796.0	0.0029	0.42
Scheme 1	0.93	928.1	0.0017	692.3	0.0038	0.40
Scheme 2	1.85	1050.3	0.0016	821.9	0.0030	0.51
Scheme 3	2.00	1143.2	0.0022	798.5	0.0025	0.53
Scheme 4	2.92	1233.1	0.0020	951.5	0.0028	0.59

In order to evaluate the increase in strength demand due to the rehabilitation studies more precisely, the fundamental periods of the structures were calculated and plotted on the related design acceleration spectrum proposed by the Turkish Earthquake Code [1], Figure 6.24. In determining the design spectrum, the soil type used was Z3. As shown in Figure 6.24, there was no difference in the strength demands of the strengthened structures and the brick infilled structure. Moreover, this difference was significantly low between the bare frame and the strengthened frames. The strengthening method proposed increased the capacity of the structure by almost 100 percent, while the increase in the strength demand was significantly low.

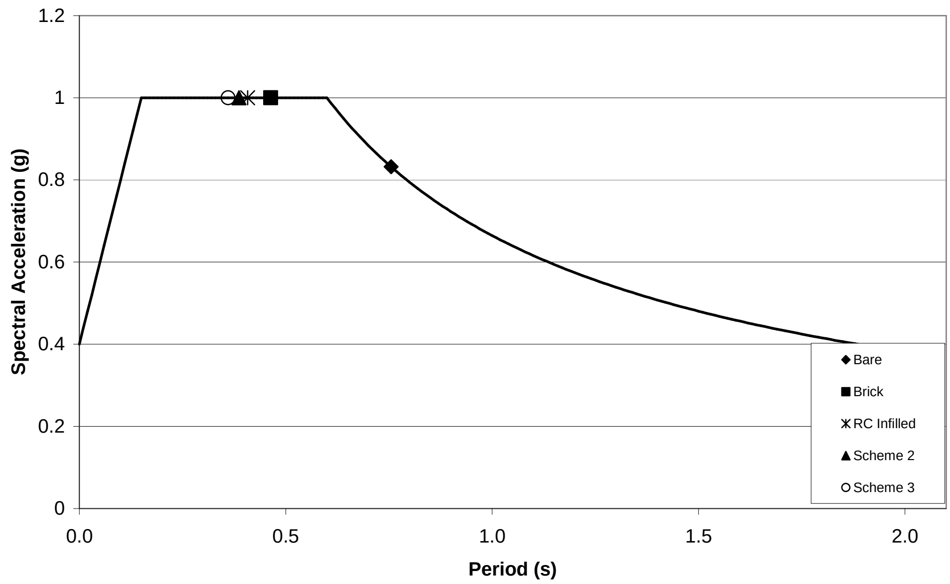


Figure 6.24 Design Response Spectrum for DBI Building

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 SUMMARY

In Turkey, most of the residential buildings have been designed and constructed with some deficiencies mainly due to lack of inspection. Moreover, due to unfair competition, most of the engineers designing and/or constructing these structures have insufficient knowledge regarding the seismic behavior of reinforced concrete structures. These two facts result in enormous number of structures which need to be strengthened in order to possess sufficient earthquake safety. Upgrading all these structures to such a level that they could be in use after a major earthquake does not seem to be economically feasible. Thus, a strengthening method that would prevent the collapse of the structure should be developed. Conventional strengthening techniques like the introduction of reinforced concrete infills seem to be inconvenient for strengthening such a great number of structures since they enforce the residents to leave the building during the strengthening period. In this study strengthening with carbon fiber reinforced polymers (CFRP) was proposed as a solution to this engineering problem due to favorable mechanical properties and ease of application. The technique developed can be applied with minimum disturbance to the occupants.

In this project, experimental and analytical studies were carried out to see the effectiveness and the feasibility of strengthening the buildings using CFRP. In the experimental program, seven one bay two story 1/3 scale undamaged brick infilled reinforced concrete frames strengthened using CFRP were tested under reversed cyclic loading. The reinforced concrete test frames used in this study possessed the most common deficiencies observed in practice.

The test results were evaluated considering strength, stiffness and energy dissipation characteristics of the test specimens.

In the analytical part of the study, CFRP reinforced infills were modeled using diagonal struts. The strut model was developed by carrying out a trial and error procedure using the test results. The strut model developed was applied to a sample building and the results of the analyses were used to develop design criteria for strengthening of reinforced concrete structures using CFRP.

7.2 OBSERVATIONS AND CONCLUSIONS

The conclusions given here are mainly based on the test results reported by the authors. Conclusions related to design criteria for the seismic strengthening of reinforced concrete buildings using CFRP are drawn from analytical studies made on a sample building. Parameters used in the analytical studies were derived from the experimental results reported in this report. Considering the type, configuration and size of the specimens tested, location of the lateral load and the load history, one should not generalize the conclusions without making careful judgments. Further experimental and analytical studies are needed to confirm some of the conclusions given below.

- The first two strengthened specimens (SP-2 and SP-3) showed a superior behavior as compared to the unstrengthened reference specimen SP-1. However the strength increase due to CFRP strengthening was not very significant, (about 15%). These two tests (SP-2, SP-3) clearly showed that CFRP strengthening would not be effective unless CFRP is properly anchored to the infill and to the frame members.
- Lapped splices in column longitudinal reinforcement at floor levels have a very adverse effect on the behavior of strengthened infilled frames. Confining the splice region by wrapping with CFRP, eliminated the local failure in this region and improved the behavior.

- Special anchor dowels made by rolling CFRP sheets and inserting them into holes in drilled in frame members and the infills were very effective in anchoring the CFRP sheets and strips. Strengthening made by using properly anchored CFRP significantly improved the strength and held the infill intact.
- Covering the whole surface of the infill on both faces with CFRP sheets (blanket type) and anchoring these to both the frame members and the infill increased the strength significantly. The lateral load capacity of SP-4 was more than twice that of the reference specimen SP-1 in which no strengthening was applied.
- Considering the cost of CFRP, in specimens SP-5, SP-6 and SP-7, instead of covering the whole surface by CFRP, strips were applied to the infill arranged like cross-bracing members. These CFRP strips were anchored to the infill and to the frame members using the special anchor dowels developed. The CFRP used in these three specimens was about ¼ of the amount used in specimen SP-4.
- Strengths of the test specimens with diagonally arranged CFRP strips were not as high as that of SP-4. However, the ratios of the lateral load capacity of these to that of SP-1 were; 2.1 for SP-5, 1.8 for SP-6 and 1.9 for SP-7.
- CFRP strengthening did not increase the lateral stiffness significantly. The maximum increase in initial lateral stiffness due to strengthening was about 34% in specimen SP-5.
- If the lateral stiffness of the structure, considering the contribution of the infills is adequate to control the drift, a small increase in stiffness due to CFRP strengthening can be considered as an advantage since the strength demand will not increase significantly.
- Test results showed that local strengthening made at the base of the columns could lead to inferior behavior. Local strengthening made in specimen SP-6 by confining the base of the column at the foundation level by wrapping with CFRP changed the failure mechanism and resulted in brittle shear failure. In the specimen where this local strengthening was not applied (specimen SP-5), the failure mode was flexure and therefore the behavior was more ductile.
- It was observed that the energy dissipation capacity was increased in specimens strengthened by CFRP.
- It was interesting to observe that even at high interstory drift ratios (about 0.01), no damage was observed on the infills of the specimens where the strengthening was made with cross strips (SP-5, SP-6 and SP-7). This was attributed to the confining effects of the CFRP strips.
- Analyses made on a sample building indicated that strengthening of some selected infills (about 2% of the floor area at the base) effectively increases the lateral load capacity of the structure to the life safety level.
- Although strengthening by CFRP strips applied on the infill was not as effective as the reinforced concrete infills, the fact that CFRP strengthening can be applied without disturbing the occupancy makes this technique very attractive.
- Test results and analytical studies made reveal that the failure of structures strengthened by applying CFRP strips to the infills, is brittle as compared to the one made by introducing RC infills. Therefore the ratio of infills to be strengthened by CFRP should be selected accordingly.

7.3 RECOMMENDATIONS

- Tests should be extended to multi-bay frames, in which different strengthening schemes could be applied to the selected bays.
- The experimental data obtained in this study could be used in further studies such as the Capacity Spectrum Method.
- Full scale tests should be carried out to evaluate the scale effect in structures strengthened by CFRP.

In addition to the studies carried out so far, shaking table tests would be useful to evaluate the performance of the proposed strengthening method.

ACKNOWLEDGEMENT

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