Seismic Performance Evaluation of Indigenous Brick Masonry Infill Panel Walls in Reinforced Concrete Structures

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Abstract

Infill walls are normally considered as nonstructural components in Reinforce Concrete (RC) frames, and are often neglected in the structural analysis and design because of their complex behavior. In Pakistan, which is the 6th most populous country of the world, RC frames with infill walls is popular form of construction. Therefore, the principal objective of this research is to investigate the effect of infill walls on RC frame structures under lateral loading designed to BCP-SP2007.

To achieve the desired objectives, this research was conducted in two phases. In the first phase, six full scale, single story and single bay RC frames were tested. The variables, in these frames were opening type, opening location and quantity of infilled material used. These frames were representative of typical construction in Pakistan. Before the construction of the frames, engineering properties of the constituent materials were determined according to the ASTM standards. Frames were tested in displacement-control under quasi-static loading arrangement and crack propagation, collapse hierarchy and damaged levels were studied in detail. Discussion includes crack and failure patterns, hysteresis curves, energy dissipation, back bone curve, stiffness degradation, strength, displacement ductility, ductility factor, over strength factor, response modification factor and performance levels obtained. It was concluded from this phase that infill wall increased the strength and stiffness of the reinforced concrete frame. By providing door opening at the center, not at the side of infilled wall strength increased but stiffness decreased. Strength and stiffness are related to quantity of infilled wall used. Energy dissipation and performance levels are affected by infilled wall, and also affected by opening type and opening location in the infilled wall. It was also concluded that response modification factor is more sensitive to material strength and geometric configuration (period of structure) as compared to the single value of 8.5 for concrete special moment resisting frame being adopted by Building Code of Pakistan Seismic Provisions (BCP SP-2007).

In the second phase of this research, a half scale model of two story and two bay prototype was tested on a 6DOF/60-ton capacity shake table. The prototype was designed for Zone IV as per BCP SP-2007. Similitude relations based on dimensional analysis were drawn before the construction of the model. It was the first large scaled model designed according to BCP SP-2007. It was tested on newly installed largest shake table in the history of Pakistan. Acquisition
of material was done according to the scale of the models. Before the construction of model on the shake table, it was validated with 18-ton and 48-ton service loads, because the shake table was operated for the first time and it was necessary for design of experimental testing of program. Infill walls were provided at various locations with different combinations of door and window openings.

Three different test runs were performed on the model, using shake table. Before and after performing each run of high intensity, ambient and free vibration tests were performed to compute the natural frequency and damping of the model. To capture full range of the performance of the model, it was subjected to a series of sinusoidal motion with increasing frequency starting from 0.4, 0.5, 0.75 to 7.5 Hz with the increment of 0.25 Hz. The duration of the sinusoidal motion was kept around 20 seconds, and the data was recorded for 25 seconds at a sampling frequency of 200 Hz without applying any anti-aliasing filter.

Acceleration response histories and displacement response histories from the respective accelerometer and displacement transducers installed at the top, mid and base/bottom of the model were interpreted from which hysteretic curves were drawn. Energy dissipation was calculated for the first story, second story and for whole structure by finding area enclosed by the hysteretic curves.

It was concluded from this phase that masonry infilled panel wall alters the global response of the structure by decreasing the natural time period of the structure. After the generation of the cracks and dislodging of portions of infill walls, the natural frequency of the structure decreases and it follows different patterns depending on the properties and geometry of the infilled panel. Infilled panel with door and window openings is more vulnerable to lateral loadings, and non-uniform distribution of the infilled wall produces torsion in the structure.

The RC frame design behaved well (except in fills) thus showing worthiness of BCP-SP 2007, however update to codes are needed based on this research and special attention to Non-structure components is needed.
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Dedication

I dedicate this research work to my beloved late father, Mian Gul Zada, and my family members whose prayers, love, devotion and hard work made it possible for me to pursue PhD degree in civil engineering.
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Chapter 1
Introduction

1.1 Background

Pakistan is located at the triple junction of Eurasian, Arabian and Indian plates [1]. The tectonic settings of Pakistan make it a seismically active region [2]. The first known destructive earthquake was the Quetta (Balochistan) earthquake of magnitude 7.7 $M_w$ in which between 30,000 and 60,000 people had died in 1935 [3]. In 1968 Shahbandar (Sindh, Pakistan), an earthquake of magnitude 7.5 $M_w$ occurred in which perished almost 50,000 people [4]. In Kashmir earthquake of magnitude 7.6 $M_w$, more than 80,000 people were killed; due to damage of 450,000 buildings, about 2.8 million people became homeless in 2005 [5]. In a regional setting, the earthquakes are still considered as moderate because estimates suggest an average slip rate of $\sim 18$ mm/year; therefore, great earthquake of magnitudes $> 8.0$ is possible in this region [6].

Pakistan is the 6$^{th}$ most populous country of the world [7]. Reinforced concrete (RC) frames with infill wall is a popular form of construction in Pakistan [8]. In RC structures masonry infilled walls are used to resist temperature, fire, moisture, and noise [9]. It is also aesthetically more pleasing to the people of Pakistan to have RC structures with infilled walls. Normally, in these RC frames, infill walls are considered as non-structural components and often neglected in structural analysis and design. But under seismic loads, these infill walls can develop strong interaction with the bounding frames [10]. Therefore, it’s most important to evaluate the infill wall performance and to observe its impact on the RC frame. The seismic performance of the structure is expected to improve with infill wall, because infill wall increases the strength and stiffness of the structure [11]. During an earthquake, the exact role of the masonry walls is complex and not yet clearly understood. In many cases, the strength and stiffness of an RC frame is increased by infill panels as demonstrated in Northridge earthquake of 1994 (EERI 1996). On the other hand, in some cases, interacting infill walls induced a brittle shear failure in the RC columns and, thereby, led to catastrophic failures of the structure, as demonstrated in Kocaeli earthquake of 1999 (EERI 2000). Due to large seismic loads, significant damaged in the infilled wall created falling of debris hazards as demonstrated in chi-chi earthquake of
1999 (EERI 2001) and the absence of infill wall in the vertical direction created soft stories as demonstrated in Kumamoto Japan Earthquake of 2016 (EERI 2016).

1.2 Problem statement

In Pakistan construction industry, constructing concrete frame structures with infill walls is a common practice. But a large number of variables that are involved in the contribution of infill walls are disregarded in conventional design practice. Pakistan is located in a high seismically active region. Therefore, it is necessary to probe into the effects of infill wall on RC frames and develop design recommendations for the local construction industries.

1.3 Objectives

Primary objectives of the study are:

1. To evaluate how the presence of unreinforced masonry infills modifies the stiffness, strength, ductility, response modification factor, failure pattern and energy dissipation characteristics of reinforced concrete frames by using indigenous materials.

2. To evaluate the relative merits and demerits of different opening positions and recommend the suitable opening position for doors and windows.

3. To evaluate the effect of bay multiplicity on structural behavior of RC frame.

4. To propose guidelines for designers to incorporate infill walls of indigenous materials in the design of RC building.

1.4 Scope of the Work

In this research, the effects of infill panel wall on reinforced concrete frame structure under lateral loading were evaluated. In the first phase, six full scaled, single story and single bay RC frames with different arrangement of infilled walls were tested in displacement control, quasi-static loading at the Structural Engineering Laboratory of University of Engineering and Technology (UET) Peshawar. The variables, in these frames were opening type, opening location and quantity of infilled material. In the second phase, half scaled, two story and two bay RC model was constructed and tested using 6 DOF shake table installed at the Earthquake Engineering Centre (EEC) of UET Peshawar. All the six frames tested in first phase were
incorporated in the model to study their response, in the global response of the model to lateral loading.

1.5 Research Methodology

The following methodology was implemented to achieve the objectives of the research:

1. Firstly, a reconnaissance survey was done to explore the types of construction practices in Pakistan and the types of materials used in these construction practices.

2. The most commonly used materials were selected after testing according to the standards of the American Society for Testing and Materials (ASTM).

3. To evaluate the effect of infilled panel wall on reinforced concrete frame structures under lateral loading, an experimental program was devised as shown in Figure 1-1, details of which are given below.

   - In the first phase of this research, six full scale, single story and single bay RC frames were tested under quasi-static loading arrangement, because in this type of test, loads or displacements are applied slowly, due to which the performance of the structures such as rate of propagation of cracks, hierarchy of collapse and related levels of damages were easily investigated. The variable parameters in these frames were opening type, opening location and quantity of infill material. These frames were representative of typical construction in Pakistan. They were tested in displacement-control. To simulate the load of slab, a constant vertical load was applied on all the frames.

   - In the second phase of this research, a half scale model of two story and two bay prototype was tested on a shake table to investigate the dynamic behavior of the model. The prototype was designed for zone IV as per Building Code of Pakistan Seismic Provisions (BCP SP-2007). Infill walls were provided at various locations with different combinations of door and window openings.
Experimental Program

Quasi-Static Reverse Cyclic Test

Dynamic Shake Table Test

Frame-1

Frame-2

Frame-3

Frame-4

Frame-5

Frame-6

Figure 1-1: Experimental Program outline
1.6 Thesis organization

This thesis titled “Seismic Performance Evaluation of Indigenous Brick Masonry Infill Panel Walls in Reinforced Concrete Structures” is organized in five chapters.

1. **Introduction:** This chapter is an introduction to the research. It states the problem statement of the research, research objectives, scope of the research and methodology adopted for the achievement of the objectives.

2. **Literature review:** This chapter addresses previous experimental and computational research work carried out on infilled RC frames. Previous experimental and computational research works are highlighted in term of their objectives, methodologies and conclusions. At the end of this chapter, various conclusions drawn from the available literature are also discussed.

3. **Full Scaled Reinforced Concrete Frames:** This chapter focuses on the methodology adopted for the construction of full scale RC frames, testing of these frames under quasi-static loading, results of these frames. Towards the end of this chapter laboratory capabilities are discussed in details.

4. **3D Reinforced Concrete Model:** In this chapter, methodology adopted for the construction of 3D RC model, testing by using a shake table and results are discussed in details.

5. **Summary, Conclusions, recommendations and future work:** In this chapter, summary, conclusion, recommendation based on this research and recommendations for future research are provided.
Chapter 2

Literature Review

2.1 Introduction

In order to ascertain the damage patterns and behavior of a structural component or structural system, experimental testing is the most reliable method. To understand the performance of infilled RC frame during ground motion, researchers conducted several experimental tests over the last decades. In addition, researchers have developed analytical models of these structural systems for the design process. Similarly, for the parametric studies of these structural systems, numerous computational models have been developed. In this chapter, a review of experimental and computational investigations, relevant to this research, has been presented. At the end of this chapter, various conclusions drawn from literature are also discussed.

2.2 Experimental investigation of frames structures

Polyakov was the first to propose the concept of the equivalent strut in the year 1956. He conducted experimental tests on three story-three bay steel frame infilled with masonry [11]. It was observed from the crack patterns of the system that it behaves like a diagonally braced frame and has only compression struts. This was due to the fact that at the end of compression diagonals, deformations were concentrated while near the ends of tension diagonals deformations were diminished. Equations were developed for the prediction of lateral load resistance of the panel.

Benjamin and Williams, in the year of 1957, tested three types of specimens under in-plane loading. They were: (i) reinforced concrete frames infilled with brick masonry, (ii) steel frames infilled with brick masonry and (iii) brick masonry without enclosing frames. All these types of specimens were single story and single bay frames [12]. They were scaled from 0.34 to 1.0. The walls used in these specimens had aspect ratio from 0.9 to 3.0. After testing, it was concluded that stiffness and ultimate strength were greatly affected by the aspect ratio. It was reported that when plain brick masonry is properly confined by frame, they have significant strength. Within limited test, brick size had no significance and also was the case for the frames until it was strong enough to fail the infill first. It was observed that there was little effect of
the variation of reinforcement steel area and column concrete area. About the scaling it was concluded that there was no significant misrepresentation of behavior by scaled down models. In the end, to predict the behavior of infill wall, some approximate relationships were developed.

Sachanski, in the year 1960, tested prototype and model infilled frames under monotonic loading [13]. His purpose was to compute force distribution between infill material and the RC frame and also to calculate the combined stiffness of the infill material and RC frame. An analytical model was proposed based on test results in which contact forces between the infill and the frame were analyzed by replacing their mutual bond by thirty redundant reactions. Equations for the compatibility of displacements of the infill and the frame were solved for the determination of forces, after which stress function for the stress analysis was proposed. There was some assumption in his method which might not be realistic like infill is isotropic, elastic and no separation between infill wall and the RC frame exists. Conclusions drawn on the basis of experimental testing were: (i) When loading the disc with a horizontal force, one of the diagonals is subjected to compression and the other to tension. As a result of the tension, the first fissure of the disc in the direction of the compression diagonal appeared. The increase in loading led to an increase in the fissures or to the appearance of other fissures parallel to the first one. (ii) The deformation of the masonry was small until the appearance of the diagonal fissure, on account of which the reinforced concrete frame took up a smaller part of the horizontal force (5 - 20%). (iii) The openings of the masonry decreased substantially its carrying capacity (2-5 times or more). (iv) The specific carrying capacity (for a unit cross section) of the full size masonry was lower than the specific carrying capacity of the masonry obtained through model investigations as shown in Table 2-1 and Table 2-2. Therefore, the model investigation could not be used for full size elements without taking into consideration the scale modulus.

<table>
<thead>
<tr>
<th>No</th>
<th>Dimension of masonry</th>
<th>Dimension of opening</th>
<th>Cross section of the frame</th>
<th>Mortar Compression (kg/cm²)</th>
<th>Mortar Tension (kg/cm²)</th>
<th>Test First fissure tons</th>
<th>Test Collapse tons</th>
<th>Calculated Capacity tons</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>L (cm) 160</td>
<td>H (cm) 108</td>
<td>A (cm) -</td>
<td>B (cm) 12</td>
<td>B1 (cm) 12</td>
<td>H1 (cm) -</td>
<td>30</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>L (cm) 160</td>
<td>H (cm) 108</td>
<td>A (cm) 12</td>
<td>B (cm) 12</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 2-1: Geometric dimension and characteristics of the tested model brick disc (source: Sachanski 1960)
Table 2-2: Geometric dimension and characteristics of the tested full-size disc  
(source: Sachanski 1960)

<table>
<thead>
<tr>
<th>No</th>
<th>Dimension of masonry (cm)</th>
<th>Dimension of opening (cm)</th>
<th>Cross section of the frame (cm)</th>
<th>Mortar Compression (kg/cm²)</th>
<th>Mortar Tension (kg/cm²)</th>
<th>Test First fissure tons</th>
<th>Test Collapse tons</th>
<th>Calculated Capacity tons</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>350</td>
<td>280</td>
<td>12 15</td>
<td>0.65</td>
<td>23.5</td>
<td>2.5</td>
<td>16</td>
<td>15.4</td>
</tr>
<tr>
<td>2</td>
<td>350</td>
<td>280</td>
<td>12 15</td>
<td>0.65</td>
<td>21.4</td>
<td>2.3</td>
<td>16</td>
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</tr>
<tr>
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<td>280</td>
<td>12 15</td>
<td>0.65</td>
<td>7.0</td>
<td>8.6</td>
<td>16</td>
<td>7.6</td>
</tr>
<tr>
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<td>265</td>
<td>25 15</td>
<td>1.30</td>
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<td>41</td>
<td>16</td>
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</tr>
<tr>
<td>5</td>
<td>395</td>
<td>255</td>
<td>25 25</td>
<td>1.47</td>
<td>31</td>
<td>32</td>
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<tr>
<td>6</td>
<td>385</td>
<td>245</td>
<td>25 40</td>
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<tr>
<td>7</td>
<td>395</td>
<td>255</td>
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<td>1.12</td>
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<td>27</td>
<td>16</td>
<td>22.9</td>
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<tr>
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<td>25 25</td>
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<tr>
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<td>191</td>
<td>20 20</td>
<td>1.47</td>
<td>25</td>
<td>26.8</td>
<td>16</td>
<td>22.4</td>
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<tr>
<td>10</td>
<td>200</td>
<td>127</td>
<td>13 13</td>
<td>1.53</td>
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<tr>
<td>12</td>
<td>174</td>
<td>255</td>
<td>25 25</td>
<td>1.65</td>
<td>17.2</td>
<td>18.8</td>
<td>16</td>
<td>22.0</td>
</tr>
<tr>
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<td>255</td>
<td>25 25</td>
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<tr>
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<td>380</td>
<td>240</td>
<td>140 30</td>
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<tr>
<td>15</td>
<td>380</td>
<td>240</td>
<td>140 30</td>
<td>1.4</td>
<td>12</td>
<td>13.2</td>
<td>16</td>
<td>13.8</td>
</tr>
</tbody>
</table>
Holmes, in the year 1961, tested thirteen full scale and small scale brick masonry infilled steel frames under rocking and shear loading [14]. He followed Polyakov’s idea that “the effect of infilled wall can be considered as diagonal bracing in each panel”. To study the effect of infill material on stiffness and strength of the steel frame, he applied loads both in horizontal and vertical direction. Because of the failure of infilled frame, the frame and infill wall was only contacted at the corner, as shown in Figure 2-1. He proposed that infill panel wall could be replaced by equivalent strut, having thickness equal to the thickness of the panel and width equal to one-third of the diagonal length of the panel. This one third rule as suggested is applicable to all infilled frame irrespective of the relative stiffness of the infill panel and frame. To determine the ultimate load capacity, he derived an equation as given in Equation 2.1.

\[
H = \frac{24EI \epsilon_c d}{h^3 \left(1 + \frac{I}{I_o} \cot \theta \right) \cos \theta} + Af_c \cos \theta \tag{2.1}
\]

In this equation, “\(H\)” represents horizontal load at failure acting on the infilled frame, “\(I\)” represent column moment of inertia of the steel frame, moment of inertia of the beam is represented by “\(I_o\)”, height of infill wall is represented by “\(h\)”, angle of inclination of the diagonal strut with the horizontal is represented by “\(\theta\)”, modulus of elasticity of the steel

Figure 2-1: At failure contact of the infilled wall and Frame (source: Holmes, 1961)
members is represented by “E”, diagonal length of the infill is represented by “d”, uniaxial compressive strain at failure of the infill material is represented by $\dot{e}_c$ and equivalent diagonal strut compressive strength by “$f_c$” while its area is represented by “$A$”.

Yorulmaz and Sozen in 1968 tested ten one-eighth scaled reinforced concrete models in which three were bare—they were without infilled walls. The remaining seven were infilled with brick masonry [15]. Dimensions of beam and column were the same i.e. 3in x 3in. Different percentage of steel reinforcement were used in it to obtain different strengths of frames. The main objective of this study was to develop methods for computing energy absorption capacity of the brick masonry (infill) RC frames and to define the possible failure mechanism of the RC frames. The bare frames failed in flexure, and plastic hinges got developed in the columns by simple four-hinge mechanism. For the infilled frames the first crack developed in the infill wall approximately at the same load in all the frames. Different types of mechanisms were obtained after the wall cracking according to the component strength of the frame as after the wall cracking load was transferred from wall to frame. Extension hinge developed in the beam of those frames had low percentage of steel reinforcement; therefore, failure occurred in these hinges as shown in Figure 2-2. Pure shear failure occurred in those frames having high percentage of steel reinforcement. For the bare frames, conventional methods of analysis were used and there was sound agreement between the calculated values and test results. Discrete element model analysis was used for the infilled frame before cracking of brick masonry. On the other hand, conventional method of analysis was used after the formation of cracks in the infilled wall. For the infilled frames load-deflection curves were defined and also lower-bound, for their energy absorption capacities were determined.

![Figure 2-2: (a) Extension Hinge in the Beam (b) Failure Mechanism for that frame (source: Yorulmaz and Sozen 1968)](image-url)
Fiorato, Sozen and Gamble in 1970 tested twenty-seven one-eighth scaled reinforced concrete models in which eight were single bay-single story, twelve were single bay-five story, six were three bay-two story infilled with masonry and one was single bay-five story [16]. In addition to number of bays and number of stories, the other controlled variables were (i) walls openings (ii) quality, arrangement and amount of steel reinforcement in frame and (iii) vertical loads on the columns. The purpose was to study, under lateral static load, the response of RC frames infilled with masonry. After the test results they concluded that with the addition of infilled masonry the stiffness and strength increased while ductility decreased of the RC frame. In the response to the infilled frame, the critical stage was the development of a shearing crack, separating a wall into two parts when formed along a single joint. Different idealized failure modes for the basic infilled frames were defined as shown in Figure 2-3.

Figure 2-3: Idealized failure modes of the infilled frame (source: Fiorato et al. 1970)
Proportion and quality of the masonry wall affected the load at which shearing crack developed. A hypothetical model was developed for the infilled frame (termed as Knee braced frame) as shown in Figure 2-4.

(a) Characteristic crack pattern for one-story specimens

(b) Hypothetical model for describing the response of the frame wall system subsequent to cracking of the wall

Figure 2-4: Knee braced concept of the frame (source: Fiorato et al. 1970)
Leuchars and Scrivener in 1973 tested, under in-plan cyclic loading, three single story-single bay RC frames in which one was bare frame, one was filled with grouted hollow bricks but unreinforced and the last frame was filled with reinforced grouted hollow bricks [17]. For the initial portions of the tests, unreinforced masonry infilled frame and reinforced masonry infilled frame behaved similarly. For the uncracked infilled frame, the beam analogy presented by Fiorato et al. (1970) and diagonal strut method presented by Carter and Smith (1960) provided good prediction of stiffness. However, for the prediction of strength at failure, these methods were less satisfactory. Maximum ductilities, which were defined as the maximum deflection to initial deflection at maximum load, calculated were 20 for bare frame and 17 for a reinforced infill panel frame. However, damage at these ductilities was severe. The behavior during deflection up to 1”, which meant a ductility factor of 5, gave an indication of good performance. At these deflections, unreinforced masonry infilled frame sustained 50 kips or 71.5% of maximum load and reinforced masonry infilled frame sustained 60 kips or 81% of maximum load. Repetition of these deflections gave a deterioration of load capacity.

Klingner and Bertero, in 1976, performed a series of tests on one-third scale, one and a half bay, three and a half story RC frames as shown in Figure 2-5(a). These frames were the sub-assemblages of the bottom three and half stories of prototype which was eleven story high, ten bays long and three bays wide having infills in outer two bays as shown in Figure 2-5(b) [18]. RC frames considered for the tests were four types, one bare frame and three were infilled frames. Grouted hollow core blocks were used as an infilled material but they were reinforced to allow multiple cracks. These specimens were tested under axial loads plus quasi static cycles to simulate the gravity plus earthquake loads on the prototype building. For preventing shear failure phenomena, frames were designed for ductile behavior. When weak beam-strong column mechanism developed in bare frame during the test, it was filled with clay block panel and then tested again. Multiple cracks developed in the infilled walls of all the infilled frames. Infill panel performed like diagonal struts when frame infill panel separation occurred. After the crushing of infill, strength degradation and development of soft story occurred. In concrete block infilled frame soft story developed at the second story. Main conclusions of the research were that the infill panel significantly improved the seismic performance of the RC frames. The stiffness of the infilled frames was six times and strength was two times more than the bare frame. Infill panel also prevented the deterioration of beam-column joint and also cracks were
distributed in the infill panel due to which the energy dissipation capacity of the infilled frames was better than the bare frame.

Angel and Abrams in 1994 tested eight single story, half scale and single bay masonry infill panels confined within a RC frame [19]. Masonry units used were concrete blocks or reclaimed bricks obtained from a demolished building. Other varying parameters included height to thickness ratio of the infill panel and mortar type. The uniqueness of this investigation was the evaluation of loss of out-of-plane strength as a result of in-plane damages in the panel. The specimens were tested under in-plane cyclic loading until the lateral drift became twice of the initial cracking, then these specimens were subjected to an out-of-plane pressure which was increased monotonically. Air bags were used to apply out-of-plane pressure. Size of the air bag and relative location with respect to infill is shown in Figure 2-5. From the in-plane tests it was concluded that equivalent strut theories provide good prediction of lateral stiffness up to cracking. Decrease in lateral stiffness occurred once the infill cracked. The lateral stiffness was directly proportional to the compressive strength of the masonry. Within the elastic region, the stiffness was not affected by the repetitive loading. From the out-of-plane tests, it was concluded that out-of-plane failure depends on slenderness ratio and masonry compressive strength. It does not depend on masonry tensile strength. Out-of-plane strength got reduced by
a factor, as high as two, by the in-plane cracking in slender panels. An analytical model was developed to predict out-of-plane behavior and strength of infill panel on the bases of arching action theory.

Sarah Haider, in 1995, tested four single story-single bay full scale RC frames, with and without infills, with clay brick masonry under in-plane cyclic loading [20]. In the first phase, the bare frames were subjected to a drift of ±1.0 %, and in the second phase (when these frames were infilled with unreinforced masonry) they subjected to a drift level up to ±4.0 %. Parametric variables were relative stiffness of the infilled wall and bounding frame, and aspect ratio (height to length) of the infill panel wall. Details of frames are shown in Table 2-3. Variables were studied in the form of strength, stiffness, failure modes and energy dissipation.

It was concluded that infill unreinforced masonry panel walls improved the stiffness, strength and energy dissipation of the frames but after separation from the bounding frame it is prone to out-of-plane failure. Infills with high stiffness bounding frame had greater strength, better energy dissipation capacity and smaller stiffness degradation as compared to less stiff or flexible bounding frame. Energy dissipation and strength decreased with decrease in aspect ratio but it had no substantial effect on the stiffness. Also when this ratio became smaller than 1.0, it had less pronounced effect on equivalent strut action of the specimen. Increasing mortar strength had no substantial effect on the response of infilled frame. Equivalent diagonal
compression struts models were developed on the bases of test result to represent the response of masonry infilled RC frame as shown in Figure 2-7. For the linear elastic analysis of masonry infilled RC frame, a simplified method was developed. For non-linear analysis of the infilled wall, load deformation plots, strength deterioration, stiffness degradation parameters and hysteretic loops pinching were identified.

Table 2-3: Frame Properties (source: Sarah Haider)

<table>
<thead>
<tr>
<th>Properties</th>
<th>Frame A</th>
<th>Frame B</th>
<th>Frame C</th>
<th>Frame D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength $f_c'$ (psi)</td>
<td>4975</td>
<td>5105</td>
<td>5440</td>
<td>5540</td>
</tr>
<tr>
<td>Column area $A_g$ (in$^2$)</td>
<td>100</td>
<td>144</td>
<td>100</td>
<td>144</td>
</tr>
<tr>
<td>Column moment of inertia $I_g$ (in$^4$)</td>
<td>833.3</td>
<td>1728</td>
<td>833.3</td>
<td>1728</td>
</tr>
<tr>
<td>Column steel ratio $\rho_s = A_s/A_g$ (%)</td>
<td>1.76</td>
<td>1.83</td>
<td>1.76</td>
<td>1.83</td>
</tr>
<tr>
<td>Column ties (M_u)$_{col}$ (k-in)</td>
<td>#3@5&quot;</td>
<td>#3@5&quot;</td>
<td>#3@5&quot;</td>
<td>#3@5&quot;</td>
</tr>
<tr>
<td>$V_u$ column (k)</td>
<td>26.2</td>
<td>35.1</td>
<td>26.7</td>
<td>35.8</td>
</tr>
<tr>
<td>Beam area $A_g$ (in$^2$)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>Beam moment of inertia $I_g$ (in$^4$)</td>
<td>833.3</td>
<td>1728</td>
<td>833.3</td>
<td>1000</td>
</tr>
<tr>
<td>Beam top/bottom steel ratio $\rho_{bs} = A_{bs}/A_d$ (%)</td>
<td>1.03/1.03</td>
<td>1.03/1.03</td>
<td>1.03/1.03</td>
<td>0.86/0.86</td>
</tr>
<tr>
<td>Shear stirrups (M_u)$_{beam}$ top/bottom (k-in)</td>
<td>#3@5&quot;</td>
<td>#3@5&quot;</td>
<td>#3@5&quot;</td>
<td>#3@5&quot;</td>
</tr>
<tr>
<td>$V_u$ beam (k)</td>
<td>26.2</td>
<td>26.4</td>
<td>26.7</td>
<td>29.0</td>
</tr>
<tr>
<td>Column/Beam flexural capacity $(M_u)<em>{col}/(M_u)</em>{beam}$</td>
<td>1.00</td>
<td>1.85</td>
<td>1.00</td>
<td>1.83</td>
</tr>
<tr>
<td>Height of opening (in)</td>
<td>81.5</td>
<td>81.5</td>
<td>81.5</td>
<td>81.5</td>
</tr>
<tr>
<td>Width of opening (in)</td>
<td>79</td>
<td>75</td>
<td>61</td>
<td>57</td>
</tr>
<tr>
<td>Aspect ratio</td>
<td>0.97</td>
<td>0.92</td>
<td>0.75</td>
<td>0.70</td>
</tr>
</tbody>
</table>
Mehrabi et al. in 1996, conducted a series of tests on fourteen RC frames in which one was bare frame and thirteen were infilled with two types of masonry, strong (solid blocks) and weak (hollow concrete blocks) masonry [21]. These frames were extracted from first story of a three bay and six story prototype structure. These frames were half scale, single-story and single-bay, excluding the two infilled frames which were two bay, with height/length ratio for each bay was 0.67. The design was according to the specification of Uniform Building Code (UBC 91). The live load taken was 50 psf while dead load estimated was 130 psf. Two types of frame were designed for the purpose of parametric study, one was a weak frame, designed for 26 psf lateral wind pressure with a wind speed of 100 mph and the other was a strong frame, designed for a set of equivalent static forces stipulated for Seismic Zone 4 in the UBC. The weak frame represented the existing RC frame not meeting the detailing requirements of the seismic design provisions of ACI 318-89. These frames were subjected to cyclic and monotonic lateral loads. The investigated parameters included relative stiffness and strength of the infill panel with the bounding frame, aspect ratio of the panel, lateral load history, influence of the adjacent bay and distribution and magnitude of vertical loads. It was concluded that infill improved energy dissipation capability, stiffness and strength. The frames showed quicker strength degradation and lower resistance when subjected to cyclic loadings than those exposed to monotonic loadings. The study also identified a number of failure mechanisms caused by the infill panel and frame interaction which are broadly summarized in Figure 2-8. For capturing failure mechanism, plastic method of analysis was proposed.
Mosalam, White and Gergely in 1997 conducted quasi-static tests on reduced scale, single story, gravity load designed steel frame with semi-rigid connection and infilled with unreinforced masonry wall panels [22]. Total five specimens were tested in which one was single bay and the remaining four were two bay. Openings were provided in only two of these frames. The investigated parameters were; type of infill openings, number of bay and relative strength of mortar joints and concrete blocks. It was concluded that the failure mode of infill panel depends on the compressive strength of concrete block. Stronger blocks lead to mortar cracking while weak blocks lead to corner crushing. Ultimate load of frames with failure in mortar cracking mode is 10% higher than frames with failure in corner crushing mode. The ultimate load of two bay specimen were double of the ultimate load of single bay specimen but
its initial stiffness was only 1.7 times higher than single bay specimen. Stiffness of the solid infill reduced to 40% for loads below the cracking level by providing opening. Solid infill led brittle behavior of infilled frames while perforated infill led to ductile behavior. Along the diagonals the compressive stresses were predominated. The relation between applied stresses and the strain along the diagonals was almost linear up to the cracking load indicating the equivalent strut analogy validity for this stage of loading. From the investigation, a hysteric model for infilled frames was formulated as shown in Figure 2-9. To describe the loop, five physical quantities: $K_u$ (Unloading curve maximum slope), $K_r$ (Reloading curve maximum slope), $K_o$ (Slope at zero displacement), $\rho_o$ (Residual story shear force) and $A$ (Loop area) were identified.

![Generic Hysteresis Loop and its Physical Parameters](source: Mosalam, White and Gergely 1997)

Kakaletsis and Karayannis, in 2006, conducted tests on seven one third scale, single bay and single RC frames, among which one was bare frame and six were infilled with masonry [23]. Variable parameters were (i) infill compressive strength and (ii) the shape of opening. The frames were designed according to the provision of modern codes and were tested up to 40% drift level under cyclic horizontal loading, while infill wall was designed in such a way that its cracking load was less than shear resistance of the column. The shear strength of the two types of masonry having different compressive strength was kept almost the same. Results were presented in the form of cracking mode, energy dissipation, stiffness, strength and cycling degradation. It was concluded that the presence, failure and behavior of the infills even in the
cases with openings can improve the performance of RC frames in terms of the observed load resistance, ductility, stiffness and energy dissipation capacity. The use of infills with window and door openings cannot cause a brittle frame failure as long as the infilled frames are designed in a way that the available shear resistance of the column is greater than the solid infill cracking resistance. Infilled frame with strong infills showed a better performance than those with weak infills as shown in Table 2-4. As compared to weak infills, a better distribution of cracking occurred in strong infills, thus showing a more effective mechanism for energy dissipation. The energy dissipation of the infilled frames with openings is higher in comparison with the bare frame for low lateral displacement, but the energy dissipation is reduced for infilled frame with opening and remains constant for bare frame for high lateral displacement.

Table 2-4: Comparison of test results of test specimens(source: Kakaletsis and Karayannis 2006)

<table>
<thead>
<tr>
<th>Spec.</th>
<th>Specimen description</th>
<th>v</th>
<th>γ_s (%)</th>
<th>γ_u (%)</th>
<th>k</th>
<th>v_m</th>
<th>ρ</th>
<th>β_rex</th>
<th>V_2 / V_1 (m. v.)</th>
<th>W_2 / W_1 (m. v.)</th>
<th>ΣW/ΣW_b</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Bare frame</td>
<td>1.00</td>
<td>3.44</td>
<td>15.50</td>
<td>1.00</td>
<td>0.54</td>
<td>3.97</td>
<td>1.00</td>
<td>0.90</td>
<td>0.70</td>
<td>1.00</td>
</tr>
<tr>
<td>S</td>
<td>Weak solid infill</td>
<td>1.84</td>
<td>2.82</td>
<td>9.23</td>
<td>2.48</td>
<td>0.65</td>
<td>4.24</td>
<td>1.34</td>
<td>0.87</td>
<td>0.85</td>
<td>1.57</td>
</tr>
<tr>
<td>IS</td>
<td>Strong solid infill</td>
<td>1.65</td>
<td>3.10</td>
<td>13.69</td>
<td>2.62</td>
<td>0.84</td>
<td>6.31</td>
<td>1.68</td>
<td>0.87</td>
<td>0.70</td>
<td>1.42</td>
</tr>
<tr>
<td>WO2</td>
<td>Weak infill Window l/d = 0.25</td>
<td>1.50</td>
<td>3.87</td>
<td>11.11</td>
<td>1.74</td>
<td>0.76</td>
<td>3.89</td>
<td>1.20</td>
<td>0.85</td>
<td>0.72</td>
<td>1.43</td>
</tr>
<tr>
<td>IWO2</td>
<td>Strong infill Window l/d = 0.25</td>
<td>1.54</td>
<td>2.54</td>
<td>20.17</td>
<td>2.50</td>
<td>0.70</td>
<td>6.42</td>
<td>1.26</td>
<td>0.88</td>
<td>0.75</td>
<td>1.41</td>
</tr>
<tr>
<td>DO2</td>
<td>Weak infill Door l/d = 0.25</td>
<td>1.39</td>
<td>2.76</td>
<td>12.02</td>
<td>1.57</td>
<td>0.53</td>
<td>3.20</td>
<td>1.06</td>
<td>0.87</td>
<td>0.69</td>
<td>1.02</td>
</tr>
<tr>
<td>IDO2</td>
<td>Strong infill Door l/d = 0.25</td>
<td>1.33</td>
<td>3.24</td>
<td>13.2</td>
<td>1.73</td>
<td>0.71</td>
<td>6.77</td>
<td>1.27</td>
<td>0.86</td>
<td>0.70</td>
<td>1.28</td>
</tr>
</tbody>
</table>

v: Lateral resistance, β_rex: Residual resistance, γ_s: Serviceability limit, γ_u: Ultimate limit, k: In. stiffness, v_m = V_1/V_2; ρ: Ductility factor, V_2/V_1: mean value for all cycle amplitudes of the ratios of the maximum recorded force during the second cycle to the maximum recorded force during the first cycle, W_2/W_1: mean value for all cycle amplitudes of the ratios of the energy dissipation during the second cycle to the energy dissipation during the first cycle, ΣW/ΣW_b: ratio of the cumulative energy dissipation by each infilled frame to the cumulative energy dissipation by the bare frame.

Based on the observed cracking pattern and experimental results, failure mechanism was defined for single story and single bay infilled frames with opening as shown in Figure 2-10.
Figure 2-10: (a) Major damage modes, (b), (c), (d), (e) failure mechanisms (1) Weak solid infill frame (2) Strong solid infill frame (3) Weak infill & window opening frame (4) Strong infill & window opening frame (5) Weak infill & door opening frame (6) Strong infill & door opening frame (source: D.J. Kakaletsis and C.G. Karayannis 2006)
Huanjun et al. in 2014, tested seven full scale; single story and single bay RC frames under low cycle reversed loading [24]. Main purpose of this study was to study the flexible connection between infill wall and RC frame, because in Wenchuan Earthquake (2008) most of structural damages were due to masonry infill walls which were rigidly connected to those structures. Therefore, in these seven frames, five were infilled with masonry and they were flexibly connected as shown in Figure 2-11, one frame was infilled with masonry but it was rigidly connected and one was bare frame. Variable parameters for the frames with flexible connection were: (i) vertical slit set in infilled wall, (ii) quantity and type of constructional column and (iii) the quantity of tie steel bar. Results generated were damage pattern, stiffness degradation, lateral strength, dissipated energy and displacement ductility. It was concluded that the infill wall which was rigidly connected to the frame had increased stiffness, strength and energy dissipation but decreased displacement ductility as compared to bare frame. In the case of flexible connection between infill wall and frame, the stiffness, strength and energy dissipation dropped significantly, while ductility rose up as compared to rigidly connected infilled frame.

![Diagram of infill wall connections](image)

**Figure 2-11:** (a) Column infilled wall connection (b) Beam infilled wall connection  
(source: Huanjan et al. 2014)
Qunxian et al. in 2015, tested five single bay, single story, half scale RC frames under reversed cyclic loading in which one was bare frame and the remaining four were infilled with masonry materials including solid clay bricks (traditional masonry material) and two new lightweight wall materials i.e. aerated concrete block and hollow concrete blocks [25]. The studied variable parameters included materials of the infill masonry and infill panel aspect ratio (width/height) which are shown in Table 2-5. Frame specimens were designed according to provisions of GB50011-2001 “Chinese seismic code” and were subjected to combination of vertical and horizontal lateral loading. Vertical loads of 328 kN were applied at the top of each column through hydraulic actuators. The objective of this research was to evaluate the performance of strong frame infilled with weak panel and the intricate interaction between infill panel and the surrounding frame at different stages. It was found that infilled frames showed greater performance as compared to bare frame in terms of stiffness, strength, ductility and energy dissipation capacity. Hollow concrete blocks were severely damaged as compared to solid concrete blocks and aerated concrete blocks as shown in Figure 2-12, which led to the instability of the infill wall in in-plane and out-plane directions. Greater internal forces were produced in the surrounding frame as compared to bare frame, particularly at the end of columns which had affected the failure pattern of the surrounding frame. Therefore, it was recommended to enlarge the shear and moment design values of the column to consider the local effect of infill wall on the surrounding frame. Failure mode of the frames was significantly affected by the shear strength ratio of the bounding frame to infill wall; therefore, it was recommended that the ratio of shear strength of the column to shear strength of the masonry infill panel should be greater than 1.3 in order to prevent the brittle shear failure of the columns. It was suggested that strong frame-weak infill principle should be applied for the design of infilled frame which develops two-line system resistance against lateral forces on the basis of which stiffness and strength can be calculated.
Table 2-5: Details of specimen and types of infill unit (source: Qunxian et al. 2015)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of infill unit</th>
<th>Aspect ratio of infill wall</th>
<th>Width of infill wall (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>IF-1</td>
<td>SCB (solid clay brick)</td>
<td>1.5</td>
<td>120</td>
</tr>
<tr>
<td>IF-2</td>
<td>HCB (hollow concrete block)</td>
<td>1.5</td>
<td>180</td>
</tr>
<tr>
<td>IF-3</td>
<td>HCB (hollow concrete block)</td>
<td>2.0</td>
<td>180</td>
</tr>
<tr>
<td>IF-4</td>
<td>ACB (aerated concrete block)</td>
<td>1.5</td>
<td>120</td>
</tr>
</tbody>
</table>

Figure 2-12: (a) Solid infill masonry panel (b) Hollow infill masonry panel (source: Qunxian et al. 2015)

Teguh, in 2016, tested three full scale, single bay and single story RC specimens under fully reversed quasi-static cyclic loads [26]. These specimens were confined brick masonry, confined concrete block masonry and bare frame. The main objective of this research was the evaluation of structural behavior of RC frames that are constructed by traditional methods in non-engineered buildings. They were tested under displacement-controlled scenario and the test was stopped when the ultimate load dropped to twenty percent. To draw the failure pattern, at the end of each cycle, crack patterns were recorded. Results were in the form hysteretic curves, strength contribution, stiffness and failure mode. Configurations of the tested specimen and the cracking pattern at failure are shown in Figure 2-13. It was observed that infill walls increased collapse resistance capacity of the RC frame by providing alternate paths for load transfer, but it reduced the ductility and could change the failure mode of the RC frame. It was summarized that infill wall increased strength and stiffness, changed distribution of strain in tie-column and tie-beam members, reduced ductility of the frame, change failure mode and crack development of the frame.
Figure 2-13: Configuration of the all specimen and the cracking pattern (source: Teguh 2016)
Brodsky and Yankelevsky, in 2016, tested seven half scaled single story and single bay masonry infilled RC frames without a supporting column [27]. During earthquake, impact load or blast load, a supporting column of the frame may be damaged which lead to partial or fully progressive collapse of the building. The possible contribution of infill masonry in preventing the progressive collapse of the frame or building when the supporting column would be missing or severely damaged was studied. The variable parameters were beam-column relative stiffness, block masonry type, reinforcement details of the frames and method of construction of the infill walls. The instrumentation and test setup is shown in Figure 2-14. The base of left side column was hinge supported to represent next span horizontal element and column of lower floor. While to represent missing column, there was no vertical support under the right side column. At the top of right side column monotonic vertical load was applied with the help of the hydraulic actuator and the test was terminated when it opened around 130 mm. After the test of first five specimens, weaknesses of the RC frame were noted, and reinforcement detailing was improved before the fabrication of the last two specimens. In these two specimens, shear capacity of the RC frame was improved by increasing the hoop density near the corners to prevent the formation of weak cross sections at the reinforcement overlapping zone. Longer lap splices were provided and to prevent early pull out damage the reinforcement anchor length was increased. Details of specimens with regular reinforcement and flexible columns is shown in Figure 2-15 while details of specimen with improved reinforcement and stiff columns is shown in Figure 2-16. After tests it was concluded that frame resistance to vertical load was increased 500% with infill masonry wall. The deflection caused by the loss of supporting column is controlled by the shear resistance of infill masonry wall. The performance of the frame was affected by the reinforcement detailing; resistance was increased 100% by the proposed reinforcement detailing. Shear connectors of the column and masonry block type had key influence on the failure mode of the frame.
Figure 2-14: Instrumentation and Test arrangement (source: Brodsky, Yankelevsky 2016)

Figure 2-15: Details of specimen with regular reinforcement and flexible columns (source: Brodsky, Yankelevsky 2016)
Arton et al. in 2017, conducted a series of tests on eight single story, single bay, 2/3 scaled reinforced concrete frames designed according to codes followed in Kosovo [28]. In these frames, one was bare frame, six were designed in such a way that they had different lateral strengths and were infilled with hollow block unit masonry, and the last one was infilled with solid clay bricks masonry. Specimen were tested under reversed quasi-static cyclic loading in force-controlled scenario with increment of 10 kN; but when yielding occurred, they were only subjected to loading in one direction, one specimen was exceptional from this type of loading which was tested under monotonic loading. At the top of each column, a constant vertical load of 20 kN was applied to simulate the upper story’s gravitational loads. Results were obtained in the form of cracking load, maximum load resisted, stiffness, ductility, crack pattern and force displacement curve as shown in Figure 2-17. On the basis of experimental results, it was concluded that strength of RC frame was greatly influenced by infill type (block or brick), shear strength of masonry, shapes and dimension of the masonry units; similarly, failure mechanism was depending on relative strength of infill and RC frame. It was suggested by the authors that design lateral strength ratio of reinforced concrete frame to hollow clay block masonry infilled should be greater than 1.2 to minimize the adverse effect of infilled
masonry to RC frame and to controlled the failure mechanism. Thus, for the design of infilled reinforced concrete frames, strong frame with weak infill was proposed.

![Figure 2-17: (a) beam column joint detail (b) cracking pattern (c) force deformation curve of the Frame 2](source: Arton et al. 2017)

Aristomenis et al. in 2017, conducted five tests on single bay, single story and one third scaled RC frames [29]. The research was aimed at evaluating the response of isolated infilled masonry wall from the RC frame using cellular material during initial stage of deformation. In the first phase, three frames were constructed and were infilled with hollow clay brick with different interface conditions. In one infilled frame 1.5 mm thick cellular material strip as shown in Figure 2-18 (a) was applied at full parameter between infilled masonry and RC frame. In the other, it was only applied at vertical interface between infilled masonry and RC column as shown in Figure 2-18 (b). While the third served as control specimen, in which no cellular material was provided and infilled masonry was in full contact with the frame. These frames were tested in in-plane direction under reverse cyclic loading in displacement control scenario. In the second phase the infilled wall was removed from the first two tested frames and were repaired using CFRP and they were infilled partially with masonry. In these retrofitted frames, 3mm thick strips of foamed polyethylene were provided between column and infilled masonry in one frame as shown in Figure 2-18 (c), the second was constructed without isolation. During the test, infilled frame without isolation was severely damaged as compared to the isolated one. Similarly, isolated partially infilled minimized the adverse influence of frame infill interaction. It was concluded that proposed isolation system increased lateral stiffness and shear strength at higher deformation of the infilled RC frames. At the end, a simple analytical model was developed by combining nonlinear springs for foamed polyethylene and single strut element.
for infilled masonry. The model was implemented in Open Sees (Open System for Earthquake Engineering Simulation) software which showed a good match with the experimental results. A parametric study was performed on three RC buildings which were three bays and three story to demonstrate the proposed isolation system.

Figure 2-18: (a) Strips of Polyethylene (b) frame with strip b/w infill and column (c) retrofitted frame with partially isolated infill wall (source: Aristomenis et al. 2017)

Quanmin et al. [30] in 2017, conducted quasi-static test on five, single bay-single story RC frames; one was bare frame and four were infilled with fly ash hollow block masonry; details are shown in Table 2-6. Two types of connections were applied, one was rigid in which mortar was provided between frame and infill wall and other was flexible connection in which 30 mm thick strip of polystyrene was provided between infill and frame, as shown in Figure 2-19. The other variable was type of infill walls, in one type RC core columns were provided at the ends of infill wall by inserting vertical steel rebar at the holes of fly ash hollow block and then pouring concrete and in the other type no RC core columns were provided. After the test, it was concluded that infill wall panel increased stiffness and strength of the bare frame greatly at small displacement stages, but at large displacement stages this increase became small due to development of the cracks in the infill panels. Similarly, infilled frames with rigid connection had shown greater stiffness and strength than the infilled frames with flexible connection. Core column construction at the end of hollow masonry further improved the performance of the infilled frame. Infill wall improved the energy dissipation capacity of the bare frame which further improved with the core column in the infill wall. However, connection between infill and the bounding frame did show regular effect on dissipation energy. During the stage of large
displacement, flexibly connected infilled frame dissipated greater energy than rigidly connected infilled frame; but during the stage of small displacement, it dissipated less energy due to the large influence of core columns on energy dissipation. In extreme large displacement, the flexible connection became superior. The finite element model was developed in ABAQUS by combining the available concrete damage plasticity model for a masonry infill panel and fiber model for a RC frame which simulated the pinching effect on hysteresis curves very well. For the low stiffness specimen such as RWF1 and PF, the FE model overestimated the stiffness and strength during early stage, but for the high stiffness specimens such as RWF2, GWF1 and GWF2 it underestimated them in large displacement stage. The accurate simulation of the complex cyclic behavior was difficult as it involved numerous material parameters and several material models. It was concluded by considering the uncertainties related to scale effect, number of specimens, variability of masonry’s mechanical properties and other factors, to conduct further experimental and analytical studies.

Table 2-6: Characteristics of specimen. (source: Quanmin et al. 2017)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Frame type</th>
<th>Masonry infill type</th>
<th>Connection between frame and infill</th>
</tr>
</thead>
<tbody>
<tr>
<td>PF</td>
<td>Bare frame</td>
<td>None</td>
<td>----</td>
</tr>
<tr>
<td>GWF1</td>
<td>Infilled frame</td>
<td>fly ash hollow block masonry without RC core columns</td>
<td>rigid</td>
</tr>
<tr>
<td>RWF1</td>
<td>Infilled frame</td>
<td>fly ash hollow block masonry without RC core columns</td>
<td>flexible</td>
</tr>
<tr>
<td>GWF2</td>
<td>Infilled frame</td>
<td>fly ash hollow block masonry with RC core columns</td>
<td>rigid</td>
</tr>
<tr>
<td>RWF2</td>
<td>Infilled frame</td>
<td>fly ash hollow block masonry with RC core columns</td>
<td>flexible</td>
</tr>
</tbody>
</table>
2.3 Modes of failure of infilled frames

According to Crisafulli, it is normally difficult to predict the type of failure of infilled frame, because it depends on several factors such as dimension of the structure, the strength of their components and the relative stiffness of the infill panel and the frame. The collapse of the structure commonly comprises one or more simple type of failure which occurs in the masonry infill as well as in the boundary frame. He summarized the different modes of failure which may occur in masonry infills which are shown in Figure 2-20 and in the boundary frame which are shown in Figure 2-21.
Figure 2-20: Modes of failure observed in masonry infills (source: Crisafulli 1997 [31])
The most prevailing tool for the analysis of complex structural problems is the finite element method [32]. In recent decades, the method has undergone huge developments related to the mathematical representation of the complex experimental behavior. This method is used by several researchers for the analysis of the behavior of frames infilled with masonry.

The first finite element method to analyze infilled frames was suggested by Mallick and Severn in 1967 [33]. Linear elastic rectangular finite elements were used for the simulation of infill panels in which each corner node was two degrees of freedom as shown in the Figure 2-22. Contact length was calculated for the modeled interface between infill and frame. In the contact region, link element was used to take into account the slip between frame and infill by considering frictional shear forces in them. This element has two displacements $u$ and $v$ at each...

**Figure 2-21: Modes of failure observed in reinforced concrete boundary frames**

*(source: Crisafulli 1997)*

### 2.4 Computational investigation of frames structures

The most prevailing tool for the analysis of complex structural problems is the finite element method [32]. In recent decades, the method has undergone huge developments related to the mathematical representation of the complex experimental behavior. This method is used by several researchers for the analysis of the behavior of frames infilled with masonry.

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node. The element was incapable of resisting tensile forces, but was able to transfer bond and compressive forces. A stiff matrix for an element of the infill in a state of plane-stress was obtained on the basis of assumed stress distributions, together with consistent load matrices for the kind of loads experienced by an infilled frame. The method deals with literally loaded rectangular as well as square infilled frames. It was seen that the stiffness values obtained conformed closely to experimental values and were an improvement to previous theoretical work. For multi-story frames, the finite element method is possible though it is time-consuming and attention is paid to a much simpler approach based upon the concept of a ‘shear-structure’. The finite element method and experiment were used to show that this approach is certainly satisfactory if the height to span ratio is not greater than two.

Axely and Bertero, in 1979, proposed two finite element approaches to find the stiffness of infilled frames named constraint scheme and exact scheme as shown in Figure 2-23 [34]. In constraint approach, infilled frame was modeled by separate assemblage of finite elements for infill and frame. The separate stiffness matrices were formulated for both infill and frame, and using condensation the stiffness of the infill alone was reduced to boundary degree of freedom. A constraint relation was assumed between the infill boundary degree of freedom and 12 degree of freedom of frame. Exact scheme with refined assemblage of beam and plane stress elements
was adopted to validate the result of constraint scheme. This scheme considers coupled system. Escuela De Niñeras, a nursery school with infilled frame structural system, was selected for the analytical study because in Guatemalan earthquake 1976 it suffered extensive damage but not complete collapse. To study the effect of both the floor slabs and the infill masonry panels on the dynamic characteristics of the system, a series of four, three dimensional models of the Escuela De Niñeras was investigated. It was concluded that the infill panel wall provides economy to the construction by mean of increasing strength and stiffness of the structural frames but unfortunately this potential is often compromised by the brittle strength characteristics of commonly used infill construction as well as the brittle column behavior induced by frame infill interaction.

![Model Scheme](source: Axely and Bertero 1979)
Liauw and Kwan in 1984 proposed a plastic theory of non-integral infilled frames [35]. In the theory, for the reserve strength, friction was neglected and the stress redistribution towards collapse was taken into account. Three different modes were identified related to panel proportion and the relative strengths of the infill, the columns and the beams. Mode 1 was termed as corner crushing mode with failure in columns in which failure occurs in the columns with subsequent crushing of the infill at the loaded corners as shown in Figure 2-24 (a). This type of failure occurs in infilled frame with relatively weak columns and strong infill. Mode 2 was termed as corner crushing mode with failure in beams in which failure occurs in the beams with subsequent crushing of the infill at the loaded corners as shown in Figure 2-24 (b). This type of failure occurs in infilled frame with relatively weak beams and strong infill. Mode 3 was termed as diagonal crushing mode in which failure occurs in the infill with subsequent failure in the joints of the frame as shown in Figure 2-24 (c). This type of failure occurs in infilled frame with relatively weak infill and strong frame.

Figure 2-24: Failure modes of single story non-integral infilled frames (a) mode 1 (b) mode 2 (c) mode 3 (source: Liauw and Kwan 1984)
Rivero and Walker, in 1984, developed a nonlinear dynamic model to study the response of masonry infilled frames to earthquake motions [36]. The nonlinearities of the model include the inelastic behavior of the frame, the interaction between the frame and wall, the bracing effect that the wall has on the frame, cracking and failure of the wall, and the discontinuities between the wall and frame. In the model, triangular elements were used to model the infill panel which were characterized by isotropic, homogeneous and linear elastic constitutive relations. Each triangular element represented several bricks and mortar joints. To model the interface between infill and frame, a joint and gap element was introduced as shown in Figure 2-25. The joint element was used to represent contact condition between frame and infill while gap element was used for no-contact condition between the two members. After the analysis it was concluded that diagonal bracing of the frame by the wall depends upon the wall coming into contact with beam at opposite diagonal corners, the fundamental elastic frequency of the open frame is not an adequate measure of the frequency or behavior of the infilled frame system. The three most important variables in the infilled frame system are the strength of the infill wall, the gap size and the time of the maximum response of the open frame.

Figure 2-25: Schematic representation of the wall model (source: Rivero and Walker 1984)
Dhanasekar and Page in 1986 studied the effect of brick masonry infill properties on the performance of infilled frames [37]. They simulated the behavior of infilled frames subjected to racking loads, using a finite element model. The wall was modeled homogenously and the wall to frame joint was modeled with 1D joint element to model shear failure and separation. The nonlinear material properties of the infill panel were defined on the bases of results from the biaxial tests of 186 half scaled square panel. The incremental finite element was capable to simulate progressive failure and non-linear behavior caused by material non-linearity. The model was verified by comparison with the results of raking tests on brick masonry infilled steel frames. The finite element model was used to perform a parametric study to understand the effect of brick masonry infill properties on the performance of infilled frames subjected to racking load. They concluded that modulus of elasticity of infill masonry have a greater effect on load-deflection characteristic and have a lesser effect on ultimate strength of infilled frame. The effect of variations in Poisson ratio is negligible. The effect of the inelastic deformation is negligible. Compressive strength variation of masonry has no effect on the racking capacity of infilled frames when failure occurs by shearing down the panel diagonal but it affects the ultimate strength when the failure occurs by corner crushing. The shear and tensile bond strengths of the masonry critically effects the ultimate load, the load-deflection behavior and in extreme cases, the mode of failure of the infilled frame as shown in Table 2-7.

### Table 2-7: Effect of bond strength on the failure of brick masonry infill
(Source: Dhanasekar and Page 1986)

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Tensile bond strength, MPa</th>
<th>Shear bond strength, MPa</th>
<th>Ultimate load, kN</th>
<th>Mode of failure, MPa*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1†</td>
<td>0.40</td>
<td>0.30</td>
<td>43</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>0.00</td>
<td>0.30</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>0.80</td>
<td>0.30</td>
<td>63</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>0.40</td>
<td>0.15</td>
<td>16</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>0.40</td>
<td>0.60</td>
<td>75</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>0.80</td>
<td>0.60</td>
<td>75</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>0.20</td>
<td>0.15</td>
<td>12</td>
<td>1</td>
</tr>
</tbody>
</table>

*1. Diagonal cracking; 2. Corner crushing.
† Original material model.

Asteris in 2003 developed a new finite element (FE) technique to model the in-plane anisotropic behavior of infilled frame under lateral loads [38]. A parametric study was performed using the parameters, the percentage and the position of opening in the masonry
infill panel for the case of single-bay and single-story infilled frame. The study was then extended to the case of multistory partially or fully infilled frames. It was concluded from this study that the lateral stiffness decreases with the increase in opening percentage. For a bare frame (frame with 100% opening) this decrease can reach 87%. The stiffness factor λ remains practically constant for opening exceeding 50% as shown in Figure 2-26(a). The opening position when moved towards the compression diagonal adversely affects the overall action between the infill and the frame as shown in Figure 2-26(b). In multistory building, the infill walls have considerable contribution to the lateral resistance and stiffness of frame. In the case of three stories frame, the infill wall contributes up to 77% decrease in lateral displacement. Shear forces on the columns of frame decreased with the presence of infill walls, but in the case of soft ground story in the infilled frame, the shear forces acting on columns are significantly higher than those attained from the analysis of the bare frame.

Figure 2-26(a) λ of infilled frame in relation to opening percentage (Case B: opening upon the compressed diagonal) (b) λ of infilled frame in relation to opening percentage for different opening positions. (source: Asteris 2003)

Stavridis in 2009 performed experiments on RC frames in two phases in his Ph.D. research work [39]. In the first phase five single-bay, single-story RC frames were tested under cyclic quasi-static tests in which one was small-scale and the remaining four were large-scale. In the second phase, two-bay, three-story and large-scaled infilled frame was tested using a shake table. The testing sequence was used to develop and validate a finite element modeling methodology. In the model, he combined the smeared crack approach for masonry units and discrete crack approach for mortar joint to compensate the stress locking problem associated
with smeared crack model, while modeling the shear behavior in masonry mortar joints and RC frame. To evaluate the effect of a number of parameters to the structural performance of RC frame, a parametric study was performed, including design parameters, i.e. the geometry of the frame, the amount of transverse and longitudinal reinforcement, the size and location of openings in the infill panel, the number of bays, and the vertical load. A number of conclusions were drawn from experimental and numerical testing. The presence of openings in masonry walls can affect their response in terms of the strength, stiffness, post peak behavior and failure mechanism to lateral loads. The inability to scale the mortar joint and size of brick lead to a different failure mechanism despite of the same masonry specifications and frame design of the small scale specimen. In single bay configuration, two struts tend to develop at angle close to 45\(^\circ\) as shown in Figure 2-27, instead of compressive strut along the diagonal of the infill panel as commonly perceived. The behavior of the single story, single bay frames was similar to that of two bay, three story frame tested on the shake table as they showed similar lateral strength and cracking patterns. For the non-ductile frame, the ratio of longitudinal and transverse reinforcement between 1 to 4 % did not affect the failure mechanism and overall performance significantly, but the amount of shear reinforcement affected the structural behavior since increasing the cross-sectional area or reducing the spacing of the stirrups have prevented the loss of strength caused by shear cracks. Aspect ratio of the infill is the most influential parameter of the geometric configuration which has affected the shear force and axial forces capacity of the column. The interaction between infilled panel and frame can lead to a variety of dissimilar failure mechanism because it depends on a number of parameter including the geometry and design of the frame, the material properties of the masonry panel and RC frame, and the existence, location and geometry of opening.

![Figure 2-27: Development and geometry of compressive struts with solid infills](source: Stavridis 2009)
Yuen and Kuang [40] in 2015 simulated the nonlinear response history behavior of five typical types of infilled RC frames (i) a completely infilled frame (ii) an infilled frame with two-thirds story height infills (iii) an infill frame with a soft story (iv) an infilled frame with window openings and (v) an infilled frame with door openings, under four realistic earthquakes (a) El Centro 1979 (b) Superstition Hills 1987 (c) Kobe 1995 and (d) Chi-Chi 1999, using discrete finite element analysis with damage based constative relations. After the analysis, they concluded that full height of infill panel enhances the energy dissipation and overall stability of frame in “full infills”, “door”, “widow” structures as long as out-of-plane failure of infills does not occur under seismic excitations. Due to the infilled wall effect, the central short columns experience more severe damage as compared to edge short column in two-thirds height infilled frame because edge columns are restrained on one side while central columns are restrained on both sides by infills, thus leading to approximately 1.7-2.6 fold higher lateral seismic forces. Serious stress and damage localization cause irregular arrangement of infill walls. Vertical load carrying capacity of the infilled frame with soft first story reduces due to abrupt changes in the vertical stiffness and lead to concentrated damage of the unbraced soft story columns as shown in Figure 2-28. The axial load carrying capacity of the columns can be significantly degraded after flexural and/or shear damage as reflected by modelling. Due to the propagation of sliding cracks originated from the stress concentration regions around the corners of the openings, infilled frames with door or window openings show ratcheting or shakedown phenomena respectively in the hysteresis behavior. Due to the effect of infill on the bare frame, the capacity design concept of “strong column weak beam” may not always be applicable to infill frames because columns of infilled frames suffer much greater damage than the adjacent connected beam members.
Figure 2-28: Damage pattern of frames under Chi-Chi 1999 earthquake (a) Crack strain at peak response in frame (b) plastic response in reinforcement (c) ultimate compressive damage in frame. (source: Yuen and Kuang 2015)
Penava et al. in 2018 developed a computational model based on the non-linear FE method of analysis [41]. A series of experimental tests were carried out for the validation of computational model. A parametric study was performed to investigate the effect of door and widow size and location on the shear resistance of infilled frame. The internal shear forces in the RC frame columns and masonry infill wall of the infilled frames at various damage levels were computed and were compared with the corresponding shear forces of the bare frame. After the analysis, it was concluded that the type, size and location of opening influenced the shear resistance of masonry infill wall components. When the ratio of the area of the opening to the area of masonry infill was increased, the shear resistance of the structural components of the masonry infill decreased. The shear resistance of the columns of the RC frame was not related to opening size, type and location. The RC frame carried more shear resistance than masonry infill wall having centrically located window and eccentrically located window openings (when loads are applied in negative direction) of the infilled RC frame. However, it carried less shear resistance when loads were applied in positive direction in the case of eccentrically located window openings. The shear resistance of the bare frame was less than the frame in infilled frames containing door openings. The shear resistance of the infilled frame containing eccentrically located door openings was higher than infilled frame containing centrically located door openings. The deformation capacity of the masonry infill walls of infilled RC frame containing window opening centrically was higher than the deformation of the RC frame and lower than the fully infilled frame and the bare frame. However, for eccentrically located window opening infilled RC frame, the deformation capacity of the masonry infill frame was higher than that of the RC frame and lower than that of fully infilled frame and bare frame. The type, size and location of opening effect the design characteristics of the infilled RC frame. As compared to shear resistance of the columns of RC frame, the shear resistance of columns of infilled frame containing door opening was higher; therefore, the shear capacity of the column of the infilled RC frame containing door was underestimated in the design which might lead to inaccurate predictions of the response of the system. However, in the case of infilled frame containing window opening the shear resistance of columns was lower than the shear resistance of the column of the RC frame, therefore it was overestimated in the design.
2.5 Conclusion based on literature review

It was noted after reviewing a large amount of experimental research that various researchers arrived at different conclusion after their experimental results. A lot of parameters can affect the response of infilled structure and in the research of each researcher more than one variables were changed. Moreover, each researcher used different dimension, interface condition, infilled materials and construction techniques in their test specimens. Therefore, it is difficult to assess and understand the effect of each separate parameter on the structural response of the structure.

The structural response of the RC frames is strongly affected by the masonry panels and if the effects of these panels are neglected it can cause adverse consequences. The probable effects of these panels are:

(a) Soft story formation due to non-uniform distribution of infilled panel in vertical direction of the structure

(b) Modification of the torsional response due to non-uniform distribution of infilled panel in horizontal direction of the structure

(c) Alteration of the global response caused by the decrease of the natural time period of the structure

(d) Formation of short column effect due to provision of partial provision of infilled panel in a bay of the structure

(e) Unanticipated mode of failure of the boundary RC frame

The structural response of masonry infilled RC frames is significantly nonlinear when subjected to lateral loading. It can be distinguished into three different stages. At the start, the masonry infilled RC frame behaves like a cantilever beam until separation between infilled panel and the frame occurs, its performance can be explicated by braced frame mechanism at this stage. Further increase in lateral loading yields cracks in the infilled masonry panel succeeding different patterns. At final stage, the stiffness and strength considerably degrade resulting in collapses of the structure. Hysteretic curves show severe pinching when masonry infilled RC frames subjected to dynamic or cyclic loading.
Chapter 3

Full Scaled Reinforced Concrete Frames

3.1 Introduction

Keeping in view the availability of equipments and capacity of equipments, in this research six full scaled RC frames were tested in first phase and one ½ scaled two story 3D RC model was tested in second phase. In this chapter the methodology which was adopted and experimental layout of full scaled RC frames are explained in detail.

3.2 Selection of Materials

Main purpose of this research was to evaluate the effect of infilled walls on reinforced concrete frame during seismic load. For the infilled walls, there are two types of materials available in Pakistan i.e. bricks and concrete blocks. In these two materials bricks are commonly available and frequently used in construction industry. Therefore, in this research baked bricks were used as infilled material in reinforced concrete frames. Details of materials are given in the following sub sections.

3.2.1 Cement

Type-I Cement according to ASTM C 150 was purchased from Kohat cement factory for this Research endeavor [42]. To characterize the cement, various tests were performed. ASTM C191 was used to find initial and final setting time of the cement [43]. The density of the cement was calculated according to ASTM C188 [44]. ASTM C117 was used to calculate fineness of the cement [45]. The compressive strength and tensile strength of cement were found according to ASTM C109 [46]. Results of these tests are given in Table 3-1.

<table>
<thead>
<tr>
<th>Description</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Type-I</td>
<td></td>
</tr>
<tr>
<td></td>
<td>hydraulic</td>
<td></td>
</tr>
<tr>
<td>Initial setting time</td>
<td>99</td>
<td>minutes</td>
</tr>
<tr>
<td>Final setting time</td>
<td>208</td>
<td>minutes</td>
</tr>
<tr>
<td>Finessness</td>
<td>90.3</td>
<td>%</td>
</tr>
<tr>
<td>Density</td>
<td>188.5</td>
<td>lb/ft³</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>381</td>
<td>psi</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>2,595</td>
<td>psi</td>
</tr>
</tbody>
</table>
3.2.2 Coarse Aggregate

Basai quarry was selected for a source of coarse aggregates. Sample of aggregate was collected from this quarry according to ASTM D-75 [47]. ASTM C136 was used for the sieve analysis [48]. Results are shown in Table 3-2 and the plotted curve is shown in Figure 3-1. The bulk density and void ratio were calculated according to ASTM C-29 [49]. ASTM C127 was used for specific gravity and water absorption [50]. The results are given in Table 3-3.

Table 3-2: Sieve analysis of coarse aggregate

<table>
<thead>
<tr>
<th>ASTM Sieve #</th>
<th>Weight Retained (gm)</th>
<th>% Weight Retained</th>
<th>Cumulative % Passed</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (\frac{3}{4})&quot;</td>
<td>0</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>1&quot;</td>
<td>48</td>
<td>0.96</td>
<td>99.04</td>
</tr>
<tr>
<td>3(\frac{1}{4})&quot;</td>
<td>461</td>
<td>9.22</td>
<td>89.82</td>
</tr>
<tr>
<td>1(\frac{1}{4})&quot;</td>
<td>1845.4</td>
<td>36.91</td>
<td>52.91</td>
</tr>
<tr>
<td>3(\frac{3}{8})&quot;</td>
<td>1160</td>
<td>23.20</td>
<td>29.71</td>
</tr>
<tr>
<td>4&quot;</td>
<td>1222.2</td>
<td>24.44</td>
<td>5.27</td>
</tr>
<tr>
<td>Pan</td>
<td>263.4</td>
<td>5.27</td>
<td>0</td>
</tr>
</tbody>
</table>

Total weight of the specimen = 5000 gm.

Figure 3-1: Sieve analysis of coarse aggregate
### Table 3-3: Properties of coarse aggregates

<table>
<thead>
<tr>
<th>Description</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density</td>
<td>87.9</td>
<td>lb/ft³</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.45</td>
<td></td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.64</td>
<td></td>
</tr>
<tr>
<td>Water absorption</td>
<td>0.63</td>
<td>%</td>
</tr>
<tr>
<td>Moisture content</td>
<td>0.11</td>
<td>%</td>
</tr>
</tbody>
</table>

#### 3.2.3 Fine Aggregate

Fine aggregates were selected from Nizampur quarry for this research work. ASTM C136 was used for the sieve analysis. Results are shown in Table 3-4 and the plotted curve is shown in Figure 3-2 from which the fineness modulus was also calculated. The bulk density and void ratio were found according to ASTM C-29. For specific gravity and water absorption ASTM C128 was used [51]. The results are given in Table 3-5.

### Table 3-4: Sieve analysis of fine aggregate

<table>
<thead>
<tr>
<th>ASTM Sieve #</th>
<th>Weight Retained (gm)</th>
<th>% Weight Retained</th>
<th>Cumulative % Retained</th>
<th>Cumulative % Passed</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.45</td>
<td>0.09</td>
<td>0.09</td>
<td>99.91</td>
</tr>
<tr>
<td>8</td>
<td>1.70</td>
<td>0.34</td>
<td>0.43</td>
<td>99.57</td>
</tr>
<tr>
<td>16</td>
<td>35.75</td>
<td>7.15</td>
<td>7.58</td>
<td>92.42</td>
</tr>
<tr>
<td>30</td>
<td>157.55</td>
<td>31.51</td>
<td>39.09</td>
<td>60.91</td>
</tr>
<tr>
<td>50</td>
<td>221.45</td>
<td>44.29</td>
<td>83.38</td>
<td>16.62</td>
</tr>
<tr>
<td>100</td>
<td>69.95</td>
<td>13.99</td>
<td>97.37</td>
<td>2.63</td>
</tr>
<tr>
<td>Pan</td>
<td>13.15</td>
<td>2.63</td>
<td>100</td>
<td>0</td>
</tr>
</tbody>
</table>

Total weight of the specimen = 500gm and the sum of cumulative % retained = 227.94
Table 3-5: Properties of fine aggregates

<table>
<thead>
<tr>
<th>Description</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density</td>
<td>152.25</td>
<td>lb/ft³</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.017</td>
<td></td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.57</td>
<td></td>
</tr>
<tr>
<td>Water absorption</td>
<td>3.40</td>
<td>%</td>
</tr>
<tr>
<td>Moisture content</td>
<td>4.55</td>
<td>%</td>
</tr>
</tbody>
</table>

3.2.4 Steel Reinforcement

Steel reinforcements were selected from various vendors in Peshawar city for full scaled RC frames according to ASTM A-615 specification [52]. For longitudinal reinforcement # 5 bars of grade 40 were used, while for confinement # 3 bars of grade 60 were used. The stress strain curve of # 3 and # 5 bars are shown in Figure 3-3 and Figure 3-4 respectively.
3.2.5 Bricks

First class brick was selected after performing different types of tests from a local vendor. In Section 3.3 brick sample test results have been presented.
3.3 Assessment of Materials characteristics

After collecting all the pre-requisite materials for the construction of RC frame members, the second phase of experimental program was the testing of constituent materials to determine the various material properties. The test procedures carried out are explained as follows:

3.3.1 Compressive strength of brick

Compressive strength of masonry unit was determined according to ASTM C-67 specification [53]. Before subjecting to uniaxial compressive loading, all the bricks units were capped with gypsum. Six frames were constructed in three phases. For each phase 5 bricks were tested in Universal Testing Machine as shown in Figure 3-5. The calculated average result of compressive strength of brick units is shown in Table 3-7.

![Compressive strength test of brick under UTM](image)

Figure 3-5: Compressive strength test of brick under UTM

3.3.2 Initial Rate of Absorption (IRA)

Section 10 of ASTM C-67 test specification was used to calculate the initial rate of absorption of brick units. For each phase, five brick were tested; so a total of 15 bricks were tested. IRA was calculated as:

\[
\text{Initial Rate of Absorption (gm/min/30in}^2\text{)} = \frac{30(W_w - W_d)}{A_c}
\]

Where

\(W_w\) = weight in grams of wet brick, as per ASTM – 67 placed for one minute duration in a tray
of water

\[ W_d = \text{weight in grams of oven dry brick} \]

\[ A_c = \text{Brick surface area in contact with water} \]

The calculated average results of IRA test of brick units are shown in Table 3-7.

![Figure 3-6: Soaking of brick unit in 1/8" water](image)

### 3.3.3 Water absorption test

In this test sample of bricks were dried in an oven and its weight was determined and then they were kept for a period of 24 hours in clean water as shown in Figure 3-7. Quantity of water absorbed by the bricks sample after this soaking period was calculated by using the following formula;

\[
\text{Absorption (\%)} = \frac{W_w - W_d}{W_d} \times 100
\]

Where;

\[ W_w = \text{brick wet weight (gm) after soaking for 24 hours in water} \]

\[ W_d = \text{brick dry weight (gm)} \]

After conducting this test, the average value of water absorption is given in Table 3-7.
3.3.4 Flexural tensile strength, $f_{bt}$

Specification mentioned in Section 6 of ASTM C-67 was used to conduct flexural tensile strength test. During the test of specimens, simply-supported end conditions were assured with a span of 7.92 in. At the middle of the brick unit a vertical concentrated force was gradually increased till the failure as shown in Figure 3-8. For each phase 5 bricks were tested. The following formula was used to determine the flexural tensile strength of the brick unit.

$$f_{bt} = \frac{M_{mid}}{S_b}$$

Where;

$M_{mid} = \text{Bending moment at brick mid span}$

$S_b = \text{Section modulus of brick} = \frac{B_b T_b^2}{6}$

where, $B_b = \text{Brick width}$ and $T_b = \text{Bick thickness}$

The calculated average results of brick flexural tensile strength test are provided in Table 3-7.
3.3.5 Compressive strength of mortar, $f_{m}$

The compressive strength of mortar used in brick masonry infilled wall was executed according to ASTM C-109 specification [54]. Mortar used in this research was prepared with the ratio of 1:6 one-part cement and six parts of sand as commonly used ratio in construction industry of Pakistan. From the said mix molds of 2”x2”x2” were filled and then removed from the mold after 24 hours. Uniaxial compressive load was applied to the cube till failure in a Universal Testing Machine (UTM) as shown in Figure 3-9. For each phase six mortar cubes were tested. The calculated average results of mortar compressive strength of 28 days are presented in Table 3-7.
3.3.6 Compressive strength of brick prism, $f_{m'}$

To calculate compressive strength of masonry, prisms of dimension 18” (406mm) long x 16” (406mm) high x 9” (229mm) wide were made according to ASTM C-1314 specification [55]. English bonds which are commonly used in Pakistan for the construction masonry infilled RC frames were also used in the fabrication of prisms. The brick prisms were subjected to 28 days water curing. The brick prism was tested under uniaxial loading in UTM till failure as shown in Figure 3-10. For each phase three brick prisms were constructed. As the aspect ratio (height/width) of the prism is 1.78, therefore, a correction of factor 0.94 was applied to the data. The correction factor was calculated by interpolation because it was between the two values of the correction factors as given in Table 3-6. The brick prism compressive strength $f_{m'}$ was calculated by dividing the peak load by the cross sectional area of the loaded surface. The calculated average result of brick prism compressive strength is shown in Table 3-7.

![Figure 3-10: Brick prism compressive strength test under UTM Loading](image)

Table 3-6: Correction factor for masonry prism compressive strength [55]

<table>
<thead>
<tr>
<th>$h_p/t_p^*$</th>
<th>1.3</th>
<th>1.5</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>4.0</th>
<th>5.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Correction factor</td>
<td>0.75</td>
<td>0.86</td>
<td>1.00</td>
<td>1.04</td>
<td>1.07</td>
<td>1.15</td>
<td>1.22</td>
</tr>
</tbody>
</table>

*$h_p/t_p$ – Ratio of Prism height to least lateral dimension of prism
3.3.7 Modulus of Elasticity of brick prism, $E_m$

By using ASTM C-1314 specification, the compressive strength test of brick was performed from which modulus of elasticity was computed. Two dial gauges were installed on the brick prism one was installed at the front face whereas the other was installed at the back face of the prism. Data acquisition system was used to record the deformation in the brick prism. The following equation was used for calculating the Modulus of elasticity of masonry.

$$\text{Modulus of elasticity} = \frac{\text{Longitudinal stress difference}}{\text{Longitudinal strain difference}}$$

Where,

$$\text{Longitudinal stress difference} = \frac{\text{Compressive strength}}{3} - \frac{\text{compressive strength}}{20} = \frac{f_{m'}}{3} - \frac{f_{m'}}{20}$$

$$\text{Longitudinal strain difference} = \text{Difference in strain corresponding to} \frac{f_{m'}}{3} \text{ and } \frac{f_{m'}}{20} \text{ respectively.}$$

The calculated results of modulus of elasticity are given in Table 3-7. The brick masonry prism, stress-strain curve is shown in Figure 3-11.

![Figure 3-11: Stress strain curve of masonry prism](image-url)
3.3.8 Diagonal tensile strength $f_{tu}$ and Modulus of Rigidity $G_m$ of masonry prism

The ASTM E 519-02 specifications [56] were used to calculate the diagonal tensile strength of brick masonry prisms from diagonal compression test. The specimen prepared were of dimension 27” (686mm) high x 27” (686mm) long x 9” (229mm) wide and were subjected to testing as displayed in Figure 3-12. For each phase three specimens were prepared and tested.

![Figure 3-12: Brick masonry prism diagonal compression test](image)

The developed shear stresses in the masonry prism during the application of load were calculated by using the following equation as per ASTM E 519-02 specifications,

$$S_s = \frac{0.707 P_D}{A_n}$$

Where

- $S_s =$ Average shear stress along diagonal created under the application of load
- $P_D =$ Applied diagonal load and
- $A_n =$ Net area of masonry prism, according to ASTM E 519 – 02 it is calculated by multiplying the average length of two sides and thickness of the prism

To compute the principal tensile strength of masonry from prism diagonal compression test, ASTM have not provided information. Therefore RILEM specification [57] was followed to calculate masonry prism principle tensile strength,

$$f_{tu} = \frac{0.5 P_u}{A_n}$$

Where,
\( f_{tu} = \) Masonry prism principle tensile strength
\( P_u = \) Ultimate load applied during the test on masonry prism

Modulus of rigidity \( G_m \), was calculated from Figure 3-13 by plotting shear stress against shear strain between two points which were corresponding to 1/20\(^{th}\) and 1/3\(^{rd}\) of the peak shear strength as the secant modulus. The calculated results are shown in Table 3-7.

![Figure 3-13: Shear stress verses shear strain curve of masonry prism](image)

### 3.3.9 Cohesion c and coefficient of friction \( \mu \)

To calculate cohesion c and coefficient of friction \( \mu \) from masonry triplet test, there is no specification in ASTM standard. Therefore EN-1052-3 specifications [58] were followed to conduct triplet tests to determine these parameters for the shear strength of masonry as shown in Figure 3-14. For each phase five specimens were prepared and tested.

![Figure 3-14: Triplet test of brick masonry prism](image)

The following formula was used to calculate the triplet shear strength \( \tau_{tu} \)
\[ \tau_u = \frac{V_{us}}{2A_{bm}} \]

Where,

\( V_{us} = \) Shear force required to produce shear sliding in the triplet prism
\( A_{bm} = \) area of brick motor joint between the two bricks.

In the Table 3-7 the calculated average value of coefficient of friction \( \mu \) and cohesion \( c \) are given.

### 3.3.10 Specific weight of masonry assemblage

In order to find the total weight of masonry infilled frame structure, specific weight of masonry assemblage was calculated by using the following formula:

\[
Specific\ weight\ (\gamma_m) = \frac{Weight\ of\ the\ masonry\ specimens}{Volume\ of\ the\ Specimen}
\]

In the Table 3-7 the computed average results of specific weight of three masonry assemblage is given.

### 3.3.11 Compressive strength of concrete \( fc' \)

To achieve the required strength of concrete, mix design was performed. In accordance with mix design, the ratio of 1:1.85: 3.65 (1-part cement, 1.85 parts sand and 3.65 parts coarse aggregate) with a water cement ratio of 0.49 was used to prepare concrete. To calculate compressive strength of concrete cylinder specimen, ASTM C39 specifications [59] were used. Before subjecting to uniaxial compressive load, the loading surface of the cylinders was properly capped with gypsum in UTM as shown in the Figure 3-15. The calculated results are shown in the Table 3-7.
**Figure 3-15: Concrete cylinder test under UTM**

**Table 3-7: Material Properties**

<table>
<thead>
<tr>
<th>Description</th>
<th>Symbols</th>
<th>Values</th>
<th>Units</th>
<th>C.O.V (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength of bricks,</td>
<td>$f_b$</td>
<td>2021</td>
<td>psi</td>
<td>14.4</td>
</tr>
<tr>
<td>Initial rate of absorption,</td>
<td>--</td>
<td>66</td>
<td>g/min/30 in$^2$</td>
<td>19.2</td>
</tr>
<tr>
<td>Water absorption of bricks,</td>
<td>--</td>
<td>18</td>
<td>%</td>
<td>13.8</td>
</tr>
<tr>
<td>Flexural tensile strength of bricks</td>
<td>$f_{bt}$</td>
<td>195.2</td>
<td>psi</td>
<td>20.2</td>
</tr>
<tr>
<td>Compressive strength of mortar,</td>
<td>$f_{mo}'$</td>
<td>751</td>
<td>psi</td>
<td>2.9</td>
</tr>
<tr>
<td>Compressive strength of masonry,</td>
<td>$f_m'$</td>
<td>701.2</td>
<td>psi</td>
<td>17.5</td>
</tr>
<tr>
<td>Elastic modulus of masonry,</td>
<td>$E_m$</td>
<td>198.6</td>
<td>ksi</td>
<td>21.8</td>
</tr>
<tr>
<td>Diagonal tensile strength of masonry,</td>
<td>$f_{nu}$</td>
<td>34.8</td>
<td>psi</td>
<td>29.6</td>
</tr>
<tr>
<td>Modulus of rigidity of masonry</td>
<td>$G_m$</td>
<td>4.49</td>
<td>ksi</td>
<td>23.1</td>
</tr>
<tr>
<td>Cohesion</td>
<td>$C$</td>
<td>3.22</td>
<td>psi</td>
<td>0.92 (COD)</td>
</tr>
<tr>
<td>Coefficient of friction</td>
<td>$\mu$</td>
<td>0.21</td>
<td></td>
<td>0.92 (COD)</td>
</tr>
<tr>
<td>Specific weight of masonry material,</td>
<td>$\gamma_m$</td>
<td>95.2</td>
<td>pcf</td>
<td>9.6</td>
</tr>
<tr>
<td>Compressive strength of concrete,</td>
<td>$f_{c'}$</td>
<td>3023</td>
<td>pcf</td>
<td>11.5</td>
</tr>
</tbody>
</table>
3.4 Selection of RC frame

An RC frame was selected from actually constructed building; details are discussed in section 3.5. To evaluate the effect of infilled wall on frame, six different configurations for the same frame were selected; details of these frames are given in sub sections to follow.

3.4.1 Frame-1

Frame-1 was a bare frame without infilled wall as shown in Figure 3-16.

![Figure 3-16: Detailed diagram of Frame-1]

This frame serves as bench mark for the other frames. According to the SMRF, requirement fc’ in this frame was 3000 psi. Column dimension was 12 x 12 inch with longitudinal reinforcement as 8, #5 bars and shear reinforcement was # 3 @ 6” c/c. Similarly, beam was 12x12 inch having 3, #5 bars as main reinforcement, 3, #5 bars as negative reinforcement and shear reinforcement was #3 @ 6” c/c. Cross section of foundation pad was 18 x 12 inch, in which 8, #5 was used as longitudinal bars and #3 @ 6” c/c was used as shear reinforcement. Details are shown in Figure 3-16.

3.4.2 Frame-2

Frame -2 was completely infilled frame. Structural details of Frame-2 were just like Frame-1 but this Frame was completely infilled with brick masonry walls as shown in Figure 3-17.
3.4.3 Frame-3

Structural details of Frame-3 were like Frame-1. It was also infilled with brick masonry wall but a door opening was provided at the side of infilled wall as shown in Figure 3-18.
3.4.4 Frame-4

Structural details of Frame-4 were like Frame-1. It was also infilled with brick masonry wall but in this frame door opening was provided at the center of infilled wall as shown in Figure 3-19. From the literature, it was construed that with infilled wall lateral strength and stiffness of RC frame increased; but with openings this increase in strength and stiffness decreased. Opening also changes the load resisting mechanism and failure pattern of the frame. Main purpose of Frame-3 and Frame-4 was to evaluate the effect of door opening location on infilled wall.

![Figure 3-19: Schematic diagram of Frame-4](image)

3.4.5 Frame-5

Structural details of Frame-5 were like Frame-1 and it was also infilled with brick masonry. In this frame both door and window openings were provided as shown in Figure 3-20. Purpose of the Frame-5 and Frame-6 was to target the quantity of infilled walls and also to evaluate the load resisting mechanism and failure pattern of such type of infilled frames. From the literature, it was construed that with the increase of quantity of infilled wall, the lateral load resistance increases.
3.4.6 Frame-6

Structural details of Frame-6 were like Frame-1. It is infilled with brick masonry but in this frame window opening was provided at the center of infilled wall as shown in Figure 3-21.
3.5 Selection of loading condition

Quasi-static loading procedure was selected for the test of these six full scaled RC frames because in such type of loading arrangement loads are applied at a slow rate due to which crack propagation, collapse hierarchy and damaged levels can be studied in details. As the frame was selected from actual constructed building, plan of the building is given in Figure 3-22. The Building has been designed for seismic zone 3 as per Building Code of Pakistan Seismic Provision 2007 (BCP SP-07). The portion of the slab which was supported by the frame was calculated through tributary area in AutoCAD and for the said portion its self-weight and live loads were calculated as given below.

Live load = 150 lb/ft²  (Table 5.1, page no 5-43, BCP SP-07 [60])

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Calculation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab self-weight</td>
<td>( 49.95 \text{ ft}^2 \times 0.5 \text{ ft} \times 150 \text{ lb/ft}^3 )</td>
<td>3746.5 lb</td>
</tr>
<tr>
<td>Live load</td>
<td>( 49.95 \text{ ft}^2 \times 150 \text{ lb/ft}^2 )</td>
<td>7492.5 lb</td>
</tr>
<tr>
<td>Finishing load</td>
<td>( 49.95 \text{ ft}^2 \times 40 \text{ lb/ft}^2 )</td>
<td>1998 lb</td>
</tr>
<tr>
<td><strong>Sum</strong></td>
<td></td>
<td>13236.75 lb</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 tons</td>
</tr>
</tbody>
</table>
Figure 3-22: Plan of the building
3.6 Test setup

Test setup for the full scaled frames is shown in Figure 2-23. The calculated vertical load as discussed in section 3.5 was applied through vertical actuator. Point load from the vertical actuator was uniformly distributed on the frame through a girder placed on the top of the beam of the frame. One actuator was set in horizontal direction to apply lateral forces in displacement controlled scenario on the frame structure.

Figure 3-23: Schematic diagram for the loading arrangement
3.7 Construction of frames

A total of six full scaled, single bay and single story two dimensional reinforced concrete frames were constructed. Details of frames have been explained in section 3.4. There is a capacity of construction of only two frames at a time in the Structural lab of University of Engineering and Technology Peshawar (UET). Therefore, these six frames were constructed in three phases. In the first phase Frame-1 and Frame-2 were constructed. In the second phase Frame-3 and Frame-4 were constructed. In the third phase Frame-5 and Frame-6 were constructed. In Figure 3-24 some of the construction phases are given.
3.8 Instrumentation Plan

The lateral load analysis of all the six frames were done with the help of actuators. In order to measure the vertical and horizontal loads applied on the RC frames Load cells of capacity 250 kN and 500 kN were incorporated in the instrumentation process. Details of instrumentation plan of each frame are discussed in the following sections.

3.8.1 Instrumentation Plan of Frame-1

Twelve Linear variable displacement transducers (LVDTs), one String pot and one dial gauge were installed at Frame-1 at different locations as shown in Figure 3-26. Details are given in Table 3-8.
Table 3-8: Details of instrumentation layout of Frame-1

<table>
<thead>
<tr>
<th>S. No</th>
<th>Channel</th>
<th>Instrument (Capacity)</th>
<th>Location</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>000</td>
<td>Load cell (50 ton)</td>
<td>Horizontally</td>
<td>To measure Horizontal load</td>
</tr>
<tr>
<td>2.</td>
<td>001</td>
<td>String pot (500 mm)</td>
<td>Beam center main gauge</td>
<td>To measure Horizontal displacement</td>
</tr>
<tr>
<td>3.</td>
<td>002</td>
<td>LVDT (50 mm)</td>
<td>CRRB</td>
<td>To measure displacement at bottom, right side of the right column</td>
</tr>
<tr>
<td>4.</td>
<td>003</td>
<td>LVDT (50 mm)</td>
<td>CLLB</td>
<td>To measure displacement at bottom, left side of the left column</td>
</tr>
<tr>
<td>5.</td>
<td>004</td>
<td>LVDT (50 mm)</td>
<td>CRLB</td>
<td>To measure displacement at bottom, right side of the left column</td>
</tr>
<tr>
<td>6.</td>
<td>005</td>
<td>LVDT (50 mm)</td>
<td>CLRBB</td>
<td>To measure displacement at bottom, left side of the right column</td>
</tr>
<tr>
<td>7.</td>
<td>006</td>
<td>LVDT (50 mm)</td>
<td>CRRT</td>
<td>To measure displacement at top, right side of the right column</td>
</tr>
<tr>
<td>8.</td>
<td>010</td>
<td>LVDT (50 mm)</td>
<td>CLLT</td>
<td>To measure displacement at top, left side of the left column</td>
</tr>
<tr>
<td>9.</td>
<td>011</td>
<td>LVDT (50 mm)</td>
<td>CRLT</td>
<td>To measure displacement at top, right side of the left column</td>
</tr>
<tr>
<td>10.</td>
<td>012</td>
<td>LVDT (50 mm)</td>
<td>CLRT</td>
<td>To measure displacement at top, left side of the right column</td>
</tr>
<tr>
<td>11.</td>
<td>013</td>
<td>LVDT (50 mm)</td>
<td>BRT</td>
<td>To measure displacement at top, right side of beam</td>
</tr>
<tr>
<td>12.</td>
<td>014</td>
<td>LVDT (50 mm)</td>
<td>BLT</td>
<td>To measure displacement at top, left side of beam</td>
</tr>
<tr>
<td>13.</td>
<td>015</td>
<td>LVDT (50 mm)</td>
<td>BRB</td>
<td>To measure displacement at bottom, right side of beam</td>
</tr>
<tr>
<td>14.</td>
<td>016</td>
<td>LVDT (50 mm)</td>
<td>BLB</td>
<td>To measure displacement at bottom, left side of beam</td>
</tr>
<tr>
<td>15.</td>
<td>017</td>
<td>Dial gauge (20 mm)</td>
<td>FLC</td>
<td>To measure base slip</td>
</tr>
<tr>
<td>16.</td>
<td>018</td>
<td>Load Cell (25 ton)</td>
<td>Vertical</td>
<td>To measure Vertical load</td>
</tr>
</tbody>
</table>

The nomenclature/designations used for the various locations of LVDTs, string pot and dial gauges are shown in Figure 3-25.
Figure 3-25: Schematic diagram of instrumentation plan of Frame-1

Figure 3-26: Final Layout of instruments at Frame-1
3.8.2 Instrumentation Plan of Frame-2

Thirteen LVDTs, one string pot and two dial gauges were installed at different locations as shown in Figure 3-28: details of which are given in Table 3-9.

<table>
<thead>
<tr>
<th>S. No</th>
<th>Channel</th>
<th>Instrument (Capacity)</th>
<th>Location</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>000</td>
<td>Load cell (50 ton)</td>
<td>Horizontally</td>
<td>To measure Horizontal load</td>
</tr>
<tr>
<td>2.</td>
<td>001</td>
<td>String pot (500 mm)</td>
<td>Beam center main gauge</td>
<td>To measure Horizontal displacement</td>
</tr>
<tr>
<td>3.</td>
<td>002</td>
<td>LVDT (50 mm)</td>
<td>R-D</td>
<td>To measure displacement at right, diagonal of the infilled wall</td>
</tr>
<tr>
<td>4.</td>
<td>003</td>
<td>LVDT (50 mm)</td>
<td>L-D</td>
<td>To measure displacement at left, diagonal of the infilled wall</td>
</tr>
<tr>
<td>5.</td>
<td>004</td>
<td>LVDT (50 mm)</td>
<td>CRRB</td>
<td>To measure displacement at bottom, right side of the right column</td>
</tr>
<tr>
<td>6.</td>
<td>005</td>
<td>LVDT (50 mm)</td>
<td>CLLB</td>
<td>To measure displacement at bottom, left side of the left column</td>
</tr>
<tr>
<td>7.</td>
<td>006</td>
<td>LVDT (50 mm)</td>
<td>CRLB</td>
<td>To measure displacement at bottom, right side of the left column</td>
</tr>
<tr>
<td>8.</td>
<td>010</td>
<td>LVDT (50 mm)</td>
<td>CLRB</td>
<td>To measure displacement at bottom, left side of the right column</td>
</tr>
<tr>
<td>9.</td>
<td>011</td>
<td>LVDT (50 mm)</td>
<td>CRRT</td>
<td>To measure displacement at top, right side of the right column</td>
</tr>
<tr>
<td>10.</td>
<td>012</td>
<td>LVDT (50 mm)</td>
<td>CLLT</td>
<td>To measure displacement at top, left side of the left column</td>
</tr>
<tr>
<td>11.</td>
<td>013</td>
<td>LVDT (50 mm)</td>
<td>CRLT</td>
<td>To measure displacement at top, right side of the left column</td>
</tr>
<tr>
<td>12.</td>
<td>014</td>
<td>LVDT (50 mm)</td>
<td>CLRT</td>
<td>To measure displacement at top, left side of the right column</td>
</tr>
<tr>
<td>13.</td>
<td>015</td>
<td>Dial gauge (20 mm)</td>
<td>BRT</td>
<td>To measure displacement at top, right side of beam</td>
</tr>
<tr>
<td>14.</td>
<td>016</td>
<td>LVDT (50 mm)</td>
<td>BLT</td>
<td>To measure displacement at top, left side of beam</td>
</tr>
<tr>
<td>15.</td>
<td>017</td>
<td>LVDT (50 mm)</td>
<td>BRB</td>
<td>To measure displacement at bottom, right side of beam</td>
</tr>
<tr>
<td>16.</td>
<td>018</td>
<td>LVDT (50 mm)</td>
<td>BLB</td>
<td>To measure displacement at bottom, left side of beam</td>
</tr>
<tr>
<td>17.</td>
<td>019</td>
<td>Dial gauge (20 mm)</td>
<td>FLC</td>
<td>To measure base slip</td>
</tr>
<tr>
<td>18.</td>
<td>020</td>
<td>Load Cell (25 ton)</td>
<td>Vertical</td>
<td>To measure Vertical load</td>
</tr>
</tbody>
</table>

The nomenclature/designations used for the various locations of LVDTs, string pot and dial gauges are shown in Figure 3-27.
Figure 3-27: Schematic diagram of instrumentation plan of Frame-2

Figure 3-28: Final layout of instrument of Frame-2
3.8.3 Instrumentation plan of Frame-3

Thirteen LVDTs, one string pot and four dial gauges were installed at different locations as shown in Figure 3-30; details of which are given in Table 3-10.

<table>
<thead>
<tr>
<th>S. No</th>
<th>Channel</th>
<th>Instrument (Capacity)</th>
<th>Location</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>000</td>
<td>Load cell (50 ton)</td>
<td>Horizontally</td>
<td>To measure Horizontal load</td>
</tr>
<tr>
<td>2.</td>
<td>001</td>
<td>String pot (500 mm)</td>
<td>Beam center main gauge</td>
<td>To measure Horizontal displacement</td>
</tr>
<tr>
<td>3.</td>
<td>002</td>
<td>LVDT (50 mm)</td>
<td>R-D</td>
<td>To measure displacement at right, diagonal of the infilled wall</td>
</tr>
<tr>
<td>4.</td>
<td>003</td>
<td>LVDT (50 mm)</td>
<td>L-D</td>
<td>To measure displacement at left, diagonal of the infilled wall</td>
</tr>
<tr>
<td>5.</td>
<td>004</td>
<td>LVDT (50 mm)</td>
<td>CRRB</td>
<td>To measure displacement at bottom, right side of the right column</td>
</tr>
<tr>
<td>6.</td>
<td>005</td>
<td>LVDT (50 mm)</td>
<td>CLLB</td>
<td>To measure displacement at bottom, left side of the left column</td>
</tr>
<tr>
<td>7.</td>
<td>006</td>
<td>LVDT (50 mm)</td>
<td>CRLB</td>
<td>To measure displacement at bottom, right side of the left column</td>
</tr>
<tr>
<td>8.</td>
<td>010</td>
<td>LVDT (50 mm)</td>
<td>CLRB</td>
<td>To measure displacement at bottom, left side of the right column</td>
</tr>
<tr>
<td>9.</td>
<td>011</td>
<td>LVDT (50 mm)</td>
<td>CRRT</td>
<td>To measure displacement at top, right side of the right column</td>
</tr>
<tr>
<td>10.</td>
<td>012</td>
<td>LVDT (50 mm)</td>
<td>CLLT</td>
<td>To measure displacement at top, left side of the left column</td>
</tr>
<tr>
<td>11.</td>
<td>013</td>
<td>LVDT (50 mm)</td>
<td>CRLT</td>
<td>To measure displacement at top, right side of the left column</td>
</tr>
<tr>
<td>12.</td>
<td>014</td>
<td>LVDT (50 mm)</td>
<td>CLRT</td>
<td>To measure displacement at top, left side of the right column</td>
</tr>
<tr>
<td>13.</td>
<td>015</td>
<td>LVDT (50 mm)</td>
<td>BRT</td>
<td>To measure displacement at top, right side of beam</td>
</tr>
<tr>
<td>14.</td>
<td>016</td>
<td>LVDT (50 mm)</td>
<td>BLT</td>
<td>To measure displacement at top, left side of beam</td>
</tr>
<tr>
<td>15.</td>
<td>017</td>
<td>Dial gauge (20 mm)</td>
<td>BRB</td>
<td>To measure displacement at bottom, right side of beam</td>
</tr>
<tr>
<td>16.</td>
<td>018</td>
<td>LVDT (50 mm)</td>
<td>BLB</td>
<td>To measure displacement at bottom, left side of beam</td>
</tr>
<tr>
<td>17.</td>
<td>019</td>
<td>Dial gauge (20 mm)</td>
<td>CRT</td>
<td>To measure twisting of the frame</td>
</tr>
<tr>
<td>18.</td>
<td>020</td>
<td>Dial gauge (20 mm)</td>
<td>CLT</td>
<td>To measure twisting of the frame</td>
</tr>
<tr>
<td>19.</td>
<td>021</td>
<td>Dial gauge (20 mm)</td>
<td>FLC</td>
<td>To measure base slip</td>
</tr>
<tr>
<td>20.</td>
<td>022</td>
<td>Load Cell (25 ton)</td>
<td>Vertical</td>
<td>To measure Vertical load</td>
</tr>
</tbody>
</table>
The nomenclature/designations used for the various locations of LVDTs, string pot and dial gauges are shown in Figure 3-29.

Figure 3-29: Schematic diagram of instrumentation plan of Frame-3

Figure 3-30: Final layout of instrument of Frame-3
3.8.4 Instrumentation plan of Frame-4

Thirteen LVDTs, one string pot and four dial gauges were installed at different locations as shown in Figure 3-32; details of which are given in Table 3-11.

<table>
<thead>
<tr>
<th>S. No</th>
<th>Channel</th>
<th>Instrument (Capacity)</th>
<th>Location</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>000</td>
<td>Load cell (50 ton)</td>
<td>Horizontally</td>
<td>To measure Horizontal load</td>
</tr>
<tr>
<td>2.</td>
<td>001</td>
<td>String pot (500 mm)</td>
<td>Beam center main gauge</td>
<td>To measure Horizontal displacement</td>
</tr>
<tr>
<td>3.</td>
<td>002</td>
<td>LVDT (50 mm)</td>
<td>R-D</td>
<td>To measure displacement at right, diagonal of the infilled wall</td>
</tr>
<tr>
<td>4.</td>
<td>003</td>
<td>LVDT (50 mm)</td>
<td>L-D</td>
<td>To measure displacement at left, diagonal of the infilled wall</td>
</tr>
<tr>
<td>5.</td>
<td>004</td>
<td>LVDT (50 mm)</td>
<td>CRRB</td>
<td>To measure displacement at bottom, right side of the right column</td>
</tr>
<tr>
<td>6.</td>
<td>005</td>
<td>LVDT (50 mm)</td>
<td>CLLB</td>
<td>To measure displacement at bottom, left side of the left column</td>
</tr>
<tr>
<td>7.</td>
<td>006</td>
<td>LVDT (50 mm)</td>
<td>CRLB</td>
<td>To measure displacement at bottom, right side of the left column</td>
</tr>
<tr>
<td>8.</td>
<td>010</td>
<td>LVDT (50 mm)</td>
<td>CLRBB</td>
<td>To measure displacement at bottom, left side of the right column</td>
</tr>
<tr>
<td>9.</td>
<td>011</td>
<td>LVDT (50 mm)</td>
<td>CRRT</td>
<td>To measure displacement at top, right side of the right column</td>
</tr>
<tr>
<td>10.</td>
<td>012</td>
<td>LVDT (50 mm)</td>
<td>CLLLT</td>
<td>To measure displacement at top, left side of the left column</td>
</tr>
<tr>
<td>11.</td>
<td>013</td>
<td>LVDT (50 mm)</td>
<td>CRLT</td>
<td>To measure displacement at top, right side of the left column</td>
</tr>
<tr>
<td>12.</td>
<td>014</td>
<td>LVDT (50 mm)</td>
<td>CLRT</td>
<td>To measure displacement at top, left side of the right column</td>
</tr>
<tr>
<td>13.</td>
<td>015</td>
<td>LVDT (50 mm)</td>
<td>BRT</td>
<td>To measure displacement at top, right side of beam</td>
</tr>
<tr>
<td>14.</td>
<td>016</td>
<td>LVDT (50 mm)</td>
<td>BLT</td>
<td>To measure displacement at top, left side of beam</td>
</tr>
<tr>
<td>15.</td>
<td>017</td>
<td>Dial gauge (20 mm)</td>
<td>BRB</td>
<td>To measure displacement at bottom, right side of beam</td>
</tr>
<tr>
<td>16.</td>
<td>018</td>
<td>LVDT (50 mm)</td>
<td>BLB</td>
<td>To measure displacement at bottom, left side of beam</td>
</tr>
<tr>
<td>17.</td>
<td>019</td>
<td>Dial gauge (20 mm)</td>
<td>CRT</td>
<td>To measure twisting of the frame</td>
</tr>
<tr>
<td>18.</td>
<td>020</td>
<td>Dial gauge (20 mm)</td>
<td>CLT</td>
<td>To measure twisting of the frame</td>
</tr>
<tr>
<td>19.</td>
<td>021</td>
<td>Dial gauge (20 mm)</td>
<td>FLC</td>
<td>To measure base slip</td>
</tr>
<tr>
<td>20.</td>
<td>022</td>
<td>Load Cell (25 ton)</td>
<td>Vertical</td>
<td>To measure Vertical load</td>
</tr>
</tbody>
</table>
The nomenclature/designations used for the various locations of LVDTs, string pot and dial gauges are shown in Figure 3-31.

Figure 3-31: Schematic diagram of instrumentation plan of Frame-4

Figure 3-32: Final layout of instrument of Frame-4
3.8.5 Instrumentation plan of Frame-5

Fourteen LVDTs, one string pot and three dial gauges were installed at different locations as shown in Figure 3-33 and Figure 3-34.
3.8.6 Instrumentation plan of Frame-6

Fourteen LVDTs, one string pot and three dial gauges were installed at different locations as shown in Figure 3-35 and Figure 3-36.

Figure 3-35: Schematic diagram of instrumentation plan of Frame-5

Figure 3-36: Final layout of instrument of Frame-6
3.9 Loading pattern

To evaluate the behavior of infilled RC frames, different researchers have used different loading protocols such as those introduced in FEMA 461, IOS16670 and ATC-40 [61-63]. These loading patterns are rather similar because they are expected to produce similar performance and their energy dissipation demands are not very different. In this research, the loading pattern developed by FEMA 461 was selected. A displacement-controlled lateral loading was applied starting from 0.25 mm and reaching to a maximum of 100 mm. The loading procedure adopted for all types of frames was kept the same. It primarily comprised of an increasing amount of in-plane quasi-static loading cycles of reverse nature. Each of the displacement cycle comprised of loading the structure to a desired amount of displacement after which the load was removed and the unloading phase initiated. After unloading, the reloading phase started with a negative movement but to the range of the same amount of specified displacement. The typical lateral displacement loading history is shown in Figure 3-37.

![Figure 3-37: Typical lateral displacement loading history applied on frames](image-url)
3.10 Results and Discussions

Results and Discussion have been divided into the following sections.

3.10.1 Crack pattern

Before the application of load, the frames were white washed and grids of 4”x4” were drawn on each frame to trace the location of crack. Crack pattern of each frame is discussed in the following sub-section.

3.10.1.1 Crack pattern of Frame-1

The application of loading was started by initial displacement of 0.25 mm and the sequence followed afterwards which has been presented in section 3.9. The first visible crack appeared at a displacement of 8mm. This crack appeared at E1 box (at distance of 17” from the top of frame). At 10mm displacement diagonal crack appeared at right beam column joint and also vertical crack at left side of the beam at a location A4 to B4. The same crack extended to C3 at 12mm displacement. At 14 mm displacement a crack appeared in the bottom of left column at location V1. At 16mm displacement cracks appeared in the right column at location E28 and F30. At 20 mm the existing cracks widened and new cracks appeared at the bottom of both columns. The sequence mentioned in section 3.9 were followed and the frame was displaced to the end of sequence i.e. 100mm. Schematic diagram of the final crack pattern of the frame is shown in Figure 3-38 while some of the actual pictures are shown in Figure 3-39.
Figure 3-38: Schematic diagram of crack pattern of Frame-1
3.10.1.2 Crack pattern of Frame-2

After testing Frame-1, it was removed from the lab to keep Frame-2 visible from the front. Frame-2 was subjected to the sequence of loading as Frame-1. The first visible crack appeared at displacement of 2.5 mm. This crack appeared at the brick masonry and top beam of the frame joint at D5. At 3 mm displacement a crack appeared at the joint of left column and masonry at location from D4 to E4. The same crack extended to W4 and then it became horizontal and ended at W6 at 4 mm displacement. At the same displacement, vertical joint of right column and masonry cracked from top to bottom. At 6mm displacement the horizontal joint between top beam and masonry completely cracked from left to right. At the same
displacement, the crack which originated at W4 at 4 mm extended to W18 and then went diagonally to X19. At 12 mm lateral displacement, cracks began at both left and right beam column joint and they extended further at 14 mm displacement. At the same displacement, horizontal crack begun at H10 extended to H17, then the same crack became diagonal and ended at J19. After 14 mm horizontal cum diagonal, cracks appeared in the masonry. Schematic diagram of the final crack pattern of the frame is shown in Figure 3-40 while some of the actual pictures are shown in Figure 3-41.

Figure 3-40: Schematic diagram of crack pattern of Frame-2
3.10.1.3 Crack pattern of Frame-3

In this Frame the first visible crack appeared at 2 mm displacement at left column at masonry joint from F4 to M4 at then extended to P4. At the same displacement, horizontal crack appeared at the beam and masonry joint. At 6 mm displacement, a diagonal crack appeared from the top right side of the door and ended at X27. The sequence of displacement cycle history as discussed in section 3.9 has to be followed but after 6mm the crack widened
in width with increase in lateral load. At 32 mm lateral displacement, the test was stopped, because the right side of the infilled was about to collapse as you can see in Figure 3-43. Schematic diagram of crack pattern is shown in Figure 3-42.

Figure 3-42: Schematic diagram of crack pattern of Frame-3
3.10.1.4 Crack pattern of Frame-4

The initial visible cracks appeared in this frame at 1mm lateral displacement at beam and masonry joint and also at left column and masonry joint. At the same displacement, vertical crack appeared at masonry from H8 to J8 and then it became diagonal and ended at K11. At 2mm lateral displacement, the same cracks widened and new diagonal crack appeared from S4 to Y7. At 6mm both left and right sides of the infill wall diagonally cracked. At 52 mm the infilled frame cracked too much and also the ultimate load dropped more than 20%, therefore
the test stopped at 52mm lateral displacement. Schematic diagram of the final crack pattern of the frame is shown in Figure 3-44 while some of the actual pictures are shown in Figure 3-45.

![Figure 3-44: Schematic diagram of crack pattern of Frame-4](image-url)
3.10.1.5 Crack pattern of Frame-5

The first visible crack appeared in this frame at 1 mm lateral displacement (LD) between left side column and masonry joint, while crack between right side column and masonry joint occurred at 1.5 mm. At 2.5 mm LD, horizontal crack appeared at the top of masonry pier between window and door opening. At the same displacement, diagonal crack occurred at the bottom of window opening at both left and right side. With increase in LD, existing crack widened and new crack began. At 14 mm LD, a diagonal crack appeared at masonry pier between window and door opening throughout the height from right side to left side as shown
in Figure 3-47. With increase in LD, the width of this crack increased and at 60 mm LD masonry pier fell down due to this crack, but the test was continued because the strength did not fall down 20% and the frame was displaced laterally up to 100 mm. Schematic diagram of crack pattern is shown in Figure 3-46.

Figure 3-46: Schematic diagram of crack pattern of Frame-5
3.10.1.6 Crack pattern of Frame-6

The first visible crack appeared in this frame at 0.75 mm LD diagonally at the top right corner of the window opening. At 1.5 mm LD, both left and right sides joints between column and masonry cracked. At 2.5 mm LD, diagonal crack appeared at the bottom of both left and right sides of window opening. With increase in LD, existing cracks widened and new crack appeared as shown in Figure 3-49. Most of the cracks were diagonal but horizontal and vertical cracks also occurred. Schematic diagram of crack pattern is shown in Figure 3-48.
Figure 3-48: Schematic diagram of crack pattern of Frame-6
3.10.1.7 Cracking and ultimate failure loads

**Frame-1:** The first crack of bare frame was observed at 57% of the ultimate load, the second and third were at 65% of the ultimate load.
Frame-2: The first crack of the completely infilled frame was observed at 51\% of the ultimate load; the second one was at the 53\%, the third and fourth was at 58\% of the ultimate load.

Frame-3: The first crack of the infilled frame with side door opening was observed at 52\% of the ultimate load, the second and third was at the 54\% of the ultimate load.

Frame-4: The first crack of the infilled frame with center door opening was observed at 31\% of the ultimate load, the second and third was at 37\% of the ultimate load.

Frame-5: The first crack of the infilled frame with both door and window openings was observed at 48\% of the ultimate load, the second was at the 45\% and the third one was at 53\% of the ultimate load.

Frame-6: The first crack of the infilled frame with window opening at the center was observed at 27\% of the ultimate load, the second one was at 31\% and the third one was at 36\% of the ultimate load.

The experimental values of first visible crack and ultimate loads of the tested frames are tabulated in Table 3-12.

Table 3-12: Experimental values of cracking and ultimate loads of frames

<table>
<thead>
<tr>
<th>Frame</th>
<th>P_{cr} (ton)</th>
<th>P_{u} (ton)</th>
<th>P_{cr}/P_{u}</th>
<th>P_{cr}/P_{cr \text{ bare}}</th>
<th>P_{u}/P_{ubare}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame-1</td>
<td>7.30</td>
<td>12.69</td>
<td>0.57</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Frame-2</td>
<td>15.73</td>
<td>30.78</td>
<td>0.51</td>
<td>2.15</td>
<td>2.42</td>
</tr>
<tr>
<td>Frame-3</td>
<td>8.52</td>
<td>16.29</td>
<td>0.52</td>
<td>1.17</td>
<td>1.28</td>
</tr>
<tr>
<td>Frame-4</td>
<td>6.07</td>
<td>19.31</td>
<td>0.31</td>
<td>0.83</td>
<td>1.52</td>
</tr>
<tr>
<td>Frame-5</td>
<td>7.43</td>
<td>15.58</td>
<td>0.48</td>
<td>1.02</td>
<td>1.23</td>
</tr>
<tr>
<td>Frame-6</td>
<td>7.09</td>
<td>25.98</td>
<td>0.27</td>
<td>0.97</td>
<td>2.04</td>
</tr>
</tbody>
</table>

Where $P_{cr}$ = Load at first visible crack and $P_{u}$ = Ultimate load
3.10.2 Hysteresis Curves

Noise filter with a three point moving average procedure was applied on the experimental data. Hysteresis curve was plotted from the corrected data. Along with that, different parameters like displacement ductility, stiffness, response modification factor etc. were also computed on the filtered data. In the following sub-sections, hysteresis curves of the frames are discussed.

3.10.2.1 Comparison of Hysteresis curves of Frame-1 and Frame-2

Figure 3-50 and 3-51 show hysteresis curves of Frame-1 and Frame-2 respectively. Both the hysteretic loops were symmetrical in the forward and reverse direction of loading. With the increase of displacement, there was progressive increase in the lateral load. The response of load deformation was approximately linear up to 10 mm lateral displacement in either direction. After the initiation of cracking, the stiffness of both the frames decreased but the frame continued to resist higher loads. With repeated load cycling, the hysteretic loops became narrowed and subsequently less energy was dissipated at second and third cycle. In the case of Frame-2 the hysteretic loops exhibited pronounced pinching of the loops at the point of load reversal.
3.10.2.2 Comparison of Hysteresis curves of Frame-3 and Frame-4

Figure 3-52 and 3-52 show hysteresis curves of Frame-3 and Frame-4. Just like Frame-1 and Frame-2, both the hysteretic loops were symmetrical in the forward and reverse direction of loading. With the increase of displacement, there was a progressive increase in the lateral loading. The response of load deformation was approximately linear up to 8 mm lateral displacement in either direction. After the initiation of cracking, the stiffness of both the frames were decreased but the frames continued to resist higher loads. The area of the loops of Frame-4 is greater than Frame-3, therefore a greater amount of energy is dissipated in Frame-4 as compared to Frame-3. With repeated load cycling, the hysteretic loops became narrowed and subsequently less energy was dissipated at second and third cycle.
Figure 3-52: Hysteresis curves of Frame-3

Figure 3-53: Hysteresis curves of Frame-4
3.10.2.3 Comparison of Hysteresis curves of Frame-5 and Frame-6

Hysteresis curves of Frame-5 are shown in Figure 3-54. Up to 14 mm LD the curves were symmetrical both in positive and negative direction. At 14 mm LD there was a sudden drop occurred in loading in negative direction due to diagonal crack at the masonry pier between door and window openings, which divided pier into two triangles. In positive direction there was friction between these two tringle, therefore load was not dropped in positive direction, while in negative direction these triangles were separated from each other therefore load was dropped in this direction. Hysteresis curves of Frame-6 are shown in Frame 3-55. With cracking, stiffness was decrease but the frame continued to take higher loads. During the test of Frame-6 bolts were slightly pull out due to which frame start ed sliding motion. At 60 mm the test was stopped due to the increase in sliding motion.

![Figure 3-54: Hysteresis curves of Frame-5](image-url)
3.10.3 Energy dissipation

During seismic load the infilled wall also affected the energy dissipation of the RC frame. Therefore, in this research energy dissipation was studied in the form of equivalent damping ratio $\zeta_{eq}$ calculated by the following formula:

$$\zeta_{eq} = \frac{E_d}{2\pi E_{inp}}$$

Where $E_{inp}$ is the energy required to impose the target displacement in one cycle in the frame [64]. It was computed as the sum of half product of maximum resisting load and corresponding displacement in negative and positive loading as shown in Figure 3-56(a) and $E_d$ is dissipated energy in one cycle of the targeted displacement. It was computed by averaging the area of three cycles of the same targeted displacement as shown in Figure 3-56(b).
3.10.3.1 Comparison of Energy dissipation of Frame-1 and Frame-2

Figure 3-57 shows Equivalent damping variation of Frame-1 and Frame-2. At the initial stage, the computed equivalent damping ratio for Frame-1 was 6.25 %, while for the Frame-2 it was 14.6 %, therefore by the addition of infilled wall the damping was increased 133.6 % at initial stage. In the final stage, the computed equivalent damping ratio for Frame-1 was 8.65 % while for Frame-2 it was 15.51 % therefore infilled wall increased damping in the final stage 79.30 %. It may be due to increase in the crack width which decreases the friction between materials.

3.10.3.2 Comparison of Energy dissipation of Frame-3 and Frame-4

Figure 3-58 shows Equivalent damping variation of Frame-3 and Frame-4. As the quantity of infilled materials was same so the equivalent damping was same at the initial stage. In the case of Frame-3 equivalent damping decreased with increase of drift ratio because a
diagonal crack isolated a part of infill wall from the remaining portion, which was not then taking part in the energy dissipation and same was the case of Frame-4. At the final stages its equivalent damping came close to Frame-1 (bare frame).

![Figure 3-58: Equivalent damping variation of Frame-3 and Frame-4](image)

3.10.3.3 Comparison of Energy dissipation of Frame-5 and Frame-6

Figure 3-59 shows Equivalent damping variation of Frame-5 and Frame-6. The slope of trend line of Frame-5 was smaller as compared to Frame-6. In Frame-5 opening was provided 37% of the total infilled wall while in Frame-6 opening was 12% of the total infilled wall. In Frame-6, with increase in displacement cracks were well distributed throughout the infilled wall due to which greater energy was dissipated as compared to Frame-5 in which masonry pier fell down with the increase in displacement.

![Figure 3-59: Equivalent damping variation of Frame-5 and Frame-6](image)
3.10.3.4 Comparison of Energy dissipation of all the Frames

A combined graph of all the frames is shown in Figure 3-60. From the graph it is clear that energy dissipation of Frame-2 was maximum because of friction across the bounding frame and infilled wall and better distribution of cracks in the infilled wall. In Frame-6 which has window opening at the center of the infilled wall, on loaded diagonal, the infilled wall loosed its tension strength at the interior and a greater deterioration of the piers occurred at low drift levels. In the case of Frame-3 equivalent damping was decreased with increase of drift ratio because a diagonal crack isolated a part of infilled wall from the remaining portion, which was then not taking part in the energy dissipation and same was the case with Frame-4. In the case of Frame-5, at initial drift ratio, its equivalent damping was greater than Frame-1, but when the part of infilled wall between door and window openings fell down its equivalent damping came closer to Frame-1.

![Figure 3-60: Equivalent damping variation of all the Frames](image_url)
3.10.4 Backbone Curves

Backbone or envelope curve were plotted between maximum resisted load and the corresponding displacement of each cycle. In the following sub sections average backbone curves of the push and pull directions of the frames are discussed.

3.10.4.1 Comparison of Backbone curve of Frame-1 and Frame-2

Figure 3-61 shows Backbone curves of Frame-1 and Frame-2. From the figure it is clear that Frame-2 resisted greater load then Frame-1 i.e. Frame-2 resisted 30.78 tons load at lateral displacement of 31.7 mm (1.15% drift) while Frame-1 resisted 12.69 tons at 36 mm (1.3%) lateral displacement. Therefore, by providing complete infill wall in RC frame the strength increased 142.58 %.

![Backbone Curves of Frame-1 and Frame-2](image)

3.10.4.2 Comparison of Backbone curve of Frame-3 and Frame-4

Figure 3-62 shows Backbone curves of Frame-3 and Frame-4. From the figure it is clear that Frame-4 resisted greater load then Frame-3 i.e. Frame-4 resisted 19.31 tons load at displacement of 20.6 mm (0.79% drift) while frame 3 resisted 16.29 ton at 21.65 mm (0.83%) lateral displacement. Therefore, by providing door opening at the center of the infill wall in RC frame the strength increased 18.56 %.

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Now if we compare Frame-4 to Frame-2 then there is 37.26% decrease in the strength. It means that by providing door opening at the centre of infilled wall the strength decreased by 37.26%. Now come to Frame-3, when it is compared to Frame-2, then its strength decreased by 47.08%.

3.10.4.3 Comparison of Backbone curve of Frame-5 and Frame-6

Backbone curve of Frame-5 and Frame-6 are shown in Figure 3-63. From the figure it is clear that Frame-6 resisted greater load then Frame-5 i.e. Frame-6 resisted 25.98 tons load at displacement of 31.96 mm (1.23% drift) while Frame-5 resisted 15.5 ton at 38.5 mm (1.48% drift) lateral displacement. Therefore, lateral strength is also related to the quantity of infill material used, greater the quantity of infill material greater is be the lateral strength; in this case by providing only window instead of both door and window in the infilled wall strength increased by 66.69%.
3.10.4.4 Comparison of backbone curve of all frames

A combine graph of average backbone curves of the push and pull directions of all the frames are shown in Figure 3-64.
### 3.10.5 Bilinear idealization of backbone curves

Different researchers have used different curves for the idealization of backbone curve, the model adopted for this research is shown in Figure 3-65 and is utilized by Dehghani et al. [65], based on equal energy principle.

![Bilinear idealization model](image)

**Figure 3-65: Bilinear idealization model [65]**

According to this model the maximum resistance is $V_{\text{max}}$. $V_s$ is the first actual significant yield level where the structural response deviates significantly from the elastic response. $\Delta_u$ is the ultimate displacement, it is the corresponding displacement when maximum resistance falls down 80 percent i.e $0.8V_{\text{max}}$. $\Delta_y$ is yield displacement. $\Delta_y$ and $V_y$ are found iteratively such that the areas between bilinear and actual curve are equal and the first line of the bilinear curve intersects with the actual curve at a lateral force of $0.75V_y$. Effective stiffness is the slope of this first line of bilinear lines. Displacement ductility is the ratio of ultimate displacement to the yield displacement ($\mu = \frac{\Delta_u}{\Delta_y}$).

Over-strength factor $R_s$ was computed through the relationship $R_s = \frac{V_y}{V_s}$. The ductility reduction ($R_\mu$) representing the energy dissipation capacity of the structure was computed through $R_\mu = \sqrt{2\mu - 1}$. The response modification factor also known as force reduction factor was derived as $R = R_\mu \times R_s$, the product of ductility and over-strength factor [66].
3.10.5.1 Comparison of Elasto-plastic curve of Frame-1 and Frame-2

In Figure 3-66 and 3-67, elasto-plastic curves of Frame-1 and Frame-2 are shown. The displacement ductility for Frame-1 was calculated as 7.37 while for frame 2 it was 5.67. Therefore, displacement ductility ($\mu$) was decreased by 25.69 % with infilled wall. Ductility factor ($R\mu$) for Frame-1 was calculated as 3.71 while for Frame-2 it was 3.24; therefore, the ductility factor decreased by 12.67 % with infilled wall. Effective stiffness for Frame-1 was calculated as 7.62 kN/mm while for Frame-2 was 28.30 kN/mm. Therefore, effective stiffness increased by 271.39 % with infilled wall. Similarly, the Response modification factor for Frame-1 was calculated 8.27 while for Frame-2 it was calculated as 6.87.

![Figure 3-66: Elasto-plastic curve of Frame-1](image)
3.10.5.2 Comparison of Elasto-plastic curve of Frame-3 and Frame-4

In Figure 3-68 and 3-69, elasto-plastic curves of Frame-3 and Frame-4 are shown. The displacement ductility for Frame-3 was calculated as 6.29 while for Frame-4 it was 6.55. Therefore, displacement ductility increased by 4.13 % by providing the door opening at the center of infilled wall instead of the side of the frame. Ductility factor for Frame-3 was calculated as 3.40 while for Frame-4 it was 3.48; therefore, ductility factor increased by 2.35 %. Effective stiffness for Frame-3 was calculated as 25.84 kN/mm, while for Frame- 4 it was 24.20 kN/mm. Therefore, effective stiffness decreased by 4.38 % by providing the door opening at the center of infilled wall. Response modification factor for Frame-3 was calculated 6.15 while for Frame-4 it was calculated as 6.4.
Figure 3-68: Elasto-plastic curve of Frame-3

Figure 3-69: Elasto-plastic curve of Frame-4
3.10.5.3 Comparison of Elasto-plastic curve of Frame-5 and Frame-6

In Figure 3-70 and 3-71, elasto-plastic curves of Frame-5 and Frame-6 are shown. The displacement ductility for Frame-5 was calculated as 7.05 while for Frame-6 it was 6.06. Therefore, displacement ductility decreased by 14.04 % by providing only widow opening instead of both door and window openings. Ductility factor for Frame-5 was calculated as 3.62 and for Frame-6 as 3.33. Therefore, ductility factor decreased by 8.01% by providing only widow opening instead of both door and window openings. Effective stiffness for Frame-5 was calculated as 14.99 kN/mm and for Frame-6 as 26.20 kN/mm. Therefore, Effective stiffness increased by 74.78 % by providing only window opening instead of both door and window openings. Response modification factor for Frame-5 was calculated 6.95 while for Frame-6 it was calculated as 7.09.

Figure 3-70: Elasto-plastic curve of Frame-5
3.10.5.4 Comparison of response parameters of the tested frames

The idealized response parameters of the tested frames are shown in Table 3-13.

Table 3-13: Response parameters of the tested frames

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum lateral load (tons)</th>
<th>Stiffness (kN/mm)</th>
<th>Displacement Ductility (µ)</th>
<th>Ductility reduction factor (Rµ)</th>
<th>Over strength factor (Rs)</th>
<th>Response modification factor (R)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame-1</td>
<td>12.689</td>
<td>7.62</td>
<td>7.37</td>
<td>3.71</td>
<td>2.23</td>
<td>8.27</td>
</tr>
<tr>
<td>Frame-2</td>
<td>30.782</td>
<td>28.30</td>
<td>5.67</td>
<td>3.24</td>
<td>2.12</td>
<td>6.87</td>
</tr>
<tr>
<td>Frame-3</td>
<td>16.29</td>
<td>25.84</td>
<td>6.29</td>
<td>3.40</td>
<td>1.81</td>
<td>6.154</td>
</tr>
<tr>
<td>Frame-4</td>
<td>19.314</td>
<td>24.20</td>
<td>6.55</td>
<td>3.48</td>
<td>1.84</td>
<td>6.4</td>
</tr>
<tr>
<td>Frame-5</td>
<td>15.585</td>
<td>14.99</td>
<td>7.05</td>
<td>3.62</td>
<td>1.92</td>
<td>6.95</td>
</tr>
<tr>
<td>Frame-6</td>
<td>25.98</td>
<td>26.20</td>
<td>6.06</td>
<td>3.33</td>
<td>2.13</td>
<td>7.09</td>
</tr>
</tbody>
</table>
3.10.6 Stiffness Degradation

Beyond the elastic limit stiffness of the structure or member decreases with the increase in the displacement. For the tested frames stiffness degradation was studied by plotting percent story drift on x-axis and a ratio of $K$ and $K_p$ on y-axis [67]. $K$ was determined by peak load divided by corresponding displacement of each cycle while $K_p$ is elastic stiffness obtained from the idealized elastoplastic curve of the backbone curves.

3.10.6.1 Comparison of Stiffness Degradation of Frame-1 and Frame-2

Stiffness degradation curves for positive direction loading (PDL) and negative direction loading (NDL) of Frame-1 and Frame-2 are shown in the Figure 3-72. From the Figure it is clear that stiffness degradation of Frame-2 was greater than Frame-1. With the addition of infilled wall, the stiffness increased but being a brittle material it also increased the stiffness degradation of the RC frame.

![Stiffness degradation curves of Frame-1 and Frame-2](image-url)
3.10.6.2 Comparison of Stiffness Degradation of Frame-3 and Frame-4

Stiffness Degradation curves of Frame-3 and Frame-4 are shown in the Figure 3.73. From the Figure it is clear that stiffness degradation of Frame-3 is greater than Frame-4. By providing door opening at the side of infilled wall stiffness increased because of the resisting panel on one side of the infilled wall but it also increased the stiffness degradation of the infilled RC frame.

![Figure 3.73: Stiffness degradation curves of Frame-3 and Frame-4](image)

3.10.6.3 Comparison of Stiffness Degradation of Frame 5 and Frame 6

Stiffness Degradation curves of Frame-5 and Frame-6 are shown in the Figure 3-74. From the Figure it is clear that stiffness degradation of Frame-5 is greater than Frame-6, because in Frame-5, the portion of infilled wall between door and window opening fell down.
3.10.7 Performance levels according to ASCE

According to American Society of Civil Engineers (ASCE), there are three performance levels, that are, Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) [68]. The IO performance level is displacement corresponding to yielding point on the elasto-plastic curve. Up to this level, there is no strength degradation and no permanent deformation in the structure. The CP performance level is the ultimate displacement where the ultimate resistance falls down to 20%. Similarly, LS performance level is that point where displacement is 75% of the CP performance level.
3.10.7.1 Comparison of Performance levels of Frame-1 and Frame-2

In Figure 3-75 and 3-76, Performance levels of Frame-1 and Frame-2 are shown. The IO performance level for Frame-1 was calculated at 0.56 % drift while for Frame-2 it was at 0.38 % drift. Therefore, IO performance level was decreased by 32.14 % by providing infilled wall. The LS performance level for Frame-1 was calculated at 2.6 % drift while for Frame-2 was at 1.84 % drift. Therefore, LS performance level was decreased by 29.23 % by providing infilled wall. The CP performance level for Frame-1 was calculated at 3.4 % drift while for Frame-2 it was at 2.46 % drift. Therefore, CP performance level was decreased by 27.65 % by providing infilled wall.

Figure 3-75: Performance Levels of Frame-1
3.10.7.2 Comparison of Performance Levels of Frame-3 and Frame-4

In Figure 3-77 and 3-78, Performance levels of Frame-3 and Frame-4 are shown. The IO performance level for Frame-3 was calculated at 0.23 % drift while for Frame-4 it was at 0.26 % drift. Therefore, IO performance level was increased by 13.04 % by providing door opening at the center of infilled wall instead of the side of infilled wall. The LS performance level for Frame-3 was calculated at 0.98 % drift while for Frame-4 was at 1.45 % drift. Therefore, LS performance level was increased by 47.96 %. The CP performance level for Frame-3 was calculated at 1.31 % drift while for Frame-4 it was at 1.94 % drift. Therefore, CP performance level was increased by 48.09 %.
Figure 3-77: Performance Levels of Frame-3

Figure 3-78: Performance Levels of Frame-4
3.10.7.3 Comparison of Performance Levels of Frame-5 and Frame-6

In Figure 3-66 and 3-67, Performance levels of Frame-5 and Frame-6 are shown. The IO performance level for Frame-5 was calculated at 0.36 % drift while for Frame-6 it was calculated at 0.34 % drift; therefore, IO performance level decreased by 5.56 % by decreasing opening area. The LS performance level for Frame-5 was calculated at 1.76 % drift while for Frame-6 it was at 1.62 % drift. Therefore, LS performance level performance level decreased by 7.95 %. The CP performance level for Frame-5 was calculated at 2.35 % drift while for Frame-6 it was at 2.16 % drift; therefore, CP performance level decreased by 8.08 %.

Figure 3-79: Performance Levels of Frame-5
3.10.7.4 Comparison of Performance Levels of the tested frames

In Table 3-14 the performance levels of all the tested frames are summarized.

Table 3- 14: Performance levels of the tested frames

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Immediate Occupancy (I.O) (Percent Drift)</th>
<th>Life Safety (L.S) (Percent Drift)</th>
<th>Collapse Prevention (C.P) (Percent Drift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame-1</td>
<td>0.56</td>
<td>2.6</td>
<td>3.4</td>
</tr>
<tr>
<td>Frame-2</td>
<td>0.38</td>
<td>1.84</td>
<td>2.46</td>
</tr>
<tr>
<td>Frame-3</td>
<td>0.23</td>
<td>0.98</td>
<td>1.31</td>
</tr>
<tr>
<td>Frame-4</td>
<td>0.26</td>
<td>1.45</td>
<td>1.94</td>
</tr>
<tr>
<td>Frame-5</td>
<td>0.36</td>
<td>1.76</td>
<td>2.35</td>
</tr>
<tr>
<td>Frame-6</td>
<td>0.34</td>
<td>1.62</td>
<td>2.16</td>
</tr>
</tbody>
</table>
3.11 Laboratory Seismic capabilities

Analytical methods can predict the response and capacity of a structural element or a structural system but a lot of uncertainties are associated with this method like nonlinear behavior of material and simplified modeling processes; therefore, for the evaluation of inelastic response of a structural element of structural system experimental testing is the most reliable method. In the laboratory, the experimental setup for the test should be such that it replicates the real field conditions like boundary condition, loadings etc. The following five experimental techniques are used to test the effect of earthquake on full scale or reduced scale structures, structural models or structural components.

3.11.1 Quasi-static testing Method

In quasi-static testing method, displacement or loads are applied by means of hydraulic actuators at a very slow rate on a test specimen. Such type of test is normally performed on those structural member of structural system for which crack propagation, collapse hierarchy and damage levels are important to find out. Quasi-static tests are performed only when force or displacement levels are control targets and the rate at which these targets are applies are not important. During the test slow loading provide insights into the post-yielding behavior of a structural member or structural system, but it eliminates the effect of velocity dependent forces and acceleration dependent inertial forces. In Figure 3-81 different types of displacement history that can be used to conduct quasi-static tests are shown. In quasi-static test, actuators or jacks are used which lumped the external action of the structures which are the representative of the mass inertia forces that are produced due to acceleration of earthquake. Therefore, for the structures having distributed mass this test is not very practical.
The Structural Laboratory of University of Engineering and Technology Peshawar is equipped with “Structural Testing Frame” as shown in Figure 3-82 that can be used for quasi-static monotonic testing and Quasi-static cyclic testing. It consists of space frame that has adjustable girders that can be used to adjust the position of test object within the frame and the position of the loading actuators. It has four actuators, two are of 50 metric tons and two are of 25 metric tons capacity and can be installed at various points on the girders for positioning. They can be installed in vertical as well as in horizontal direction. This facility has a manually operated hydraulic supply. The details of actuators and hydraulic power supply are given in Table 3-15.

Figure 3-81: Various loading pattern used in quasi-static cyclic tests [67]

Figure 3-82: Structural testing frame
Table 3-15: Details of actuator and power supply of structure testing frame [69]

<table>
<thead>
<tr>
<th>S. No</th>
<th>Item</th>
<th>Description</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><strong>Hydraulic Actuators 50 ton</strong></td>
<td>Quantity = 2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Capacity</td>
<td>110.2 kips (50tf)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stroke</td>
<td>12 in (300mm)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type</td>
<td>Linear single ended</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum speed</td>
<td>0.256 in/sec (6.5 mm/sec)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Length</td>
<td>81.2 in (2,062mm)</td>
<td>In retracted position</td>
</tr>
<tr>
<td>2</td>
<td><strong>Hydraulic Actuators 25 ton</strong></td>
<td>Quantity = 2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Capacity</td>
<td>55.1 kips (25tf)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stroke</td>
<td>12 in (304.8mm)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type</td>
<td>Linear single ended</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum speed</td>
<td>0.256 in/sec (6.5 mm/sec)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Length</td>
<td>81.2 in (2,062mm)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td><strong>Hydraulic Power Supply</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Operating pressure</td>
<td>3000 psi (~210kg/cm²)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Manifold capacity</td>
<td>Four valves</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Motor</td>
<td>3 phase</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Speed Control</td>
<td>Adjustable</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pressure Control</td>
<td>Adjustable</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Control</td>
<td>Manual</td>
<td></td>
</tr>
</tbody>
</table>

### 3.11.2 Pseudo-dynamic Testing Method

In early 1980s under Japan-US Cooperative Programme, the pseudo-dynamic test (PDT) method which is also known as online testing was developed [70, 71]. PDT is hybrid testing method controlled by computer in which structures are tested dynamically up to nonlinear range by using quasi-static test equipment. Basically PDT combines online computer simulation of the structural dynamic behavior with experimental progress information of the structure. Main steps of this method are identified in flowchart as shown in Figure 3-83. In this method when input earthquake motion is applied of the specimen, computer and connected
hardware solves the equation of motion by time stepping integration method while for the same equation viscous damping and inertial forces are modelled analytically, from which displacement which is required to be apply on the structure is calculated for each time step. From the imposed displacement, the developed forces are obtained experimentally and then it is used for the equation of motion of the next time step.

![PDT method flowchart](Image)

Figure 3-83: PDT method flowchart (Source: Donea, J. and Jones [72])

3.11.3 Shake table test

In 1940s, for simulation of earthquake, load shake tables were firstly used and since 1960s it has been effectively used for the dynamic testing of the structures. Shaking table is comprised of a rigid platform which can move in one or more directions by servo-hydraulic actuators. Degrees of freedom are used for the classification of shake table along which it can produce motion. Advance shake table can produce motion in the directions defined by six degree of freedom. One of such type of shaking table is also installed at Earthquake Engineering Center (EEC) University of Engineering and Technology Peshawar as shown in Figure 3-84.
Characteristics of 6 DOF Shake table installed at UET

The EEC has installed shake table of 6x6 m² size with a 6 degree of freedom. This places Pakistan 4th in the world after USA, France and Japan (according to Guiseppe Maddaloni, Naples, Italy 2008 Page 23-24). Main characteristics of the shake table are given below.

- 60 tons maximum payload
- At 40 tons
  - Acceleration ±1.5g in all the three directions
  - Velocity ±1.1 m/s in all the three directions
  - Frequency 0-50 Hz
  - 40 ton-m overturning moment
  - 2044 liter/min hydraulic supply as shown in Figure 3-84 (a)
  - 1.2 MW electric power to run the system
  - 200 tons cooling tower
• 4 Dynamic Actuator 100 tons each as shown in Figure 3-84 (b)
  o Fatigue rated
  o Displacement ±400mm
  o Peak velocity 250 mm/sec
  o Frequency 0-50Hz

• Can be
  o Force controlled
  o Displacement controlled
  o Strain controlled

![Image](image_url)

Figure 3-85: (a) 2044 liter/min hydraulic power supply (b) 100 ton dynamic actuators

### 3.11.4 Effective Force Method (EFTM)

This method was developed to overcome some of the problems associated with pseudo-dynamic test (PDT) method. According to several authors [73] and [74] in PDT time varying forces could not be applied without computing the required displacement. In this method, the applied forces are free of structural properties like damping and stiffness and are equal to the product of lumped masses and their acceleration.

In this method the test specimen is rigidly fixed to an immovable ground and earthquake forces are simulated by applying effective lateral forces \( P_{eff}(t) = -m_i u_i'' \) to it as shown in Figure 3-86. To demonstrate the concept, equation of motion (EOM) of a single degree of freedom (SDOF) exposed to lateral loading can be used.
\[ m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g = P_{eff}(t) \]

Where m is mass, c is damping coefficient and k is stiffness of the SDOF. The product of the ground acceleration \( (\ddot{u}_g) \) and mass (m) is equal to effective lateral force \( (P_{eff}(t)) \) applied to the mass [75].

![Figure 3-86: Effective force method](image)

**3.11.5 Substructure Hybrid Shaking Table Testing method (SHSTM)**

Several researchers [76], [77] have proposed the concept of this test method in different fields including Konagai et al.[78] who studied the dynamic interaction between foundation and the structure. This method combines the shake-table test and numerical computation of the dynamic response of the combined structural system. Behind this test method, a shake-table and computer are working in an online system, exchanging information with each other. Therefore, testing and computation are done simultaneously. As in this method the structural system is divided into substructure for shake-table and numerical computation; therefore, it is
known as Substructure Hybrid Shake Table Test. Its concept is shown schematically in Figure 3-87.

Figure 3-87: Substructure Hybrid Shake Table Test (Source: Akira Igarashi et al [79])
Chapter 4

3D Reinforced Concrete Model

4.1 Introduction

Due to a large number of variables involved in the design of infill wall, the infill walls are normally neglected in the structural analysis and design. In the previous earthquakes record, it is observed that infilled walls can change load resisting mechanism and failure pattern of the RC structure. Therefore, in the second phase of this research a prototype RC structural system was selected in which infill walls with different combination door and window openings were provided. Keeping in view the capacity of shake table a scaled factor of 2 was selected for the model of RC structural system. This chapter includes the methodology and experimental layout of the ½ scaled 3D RC Model.

4.2 Planning and Designing of Experiment

From the past two decades in the major cities of Pakistan like Karachi, Lahore, Faisalabad, Islamabad and Peshawar, reinforced concrete frame structure is a popular form of construction because of the awareness of the people about the fact that these structures can perform better than traditional unreinforced masonry (URM) load bearing structure especially during an earthquake. The improvement in economic condition of the people is another reason for it [8]. Therefore, keeping in view the construction practice in these cities an RC frame structure was selected, as shown in Figure 4-1. There were six frames in the structural system as shown in Figure 4-2. Frame-1, Frame-2 and Frame-3 were in the in-plane direction while Frame-4, Frame-5 and Frame-6 were in the out of plane direction. Details of these frames are shown in Figure 4-3 to Figure 4-6.

Seismically Pakistan has been divided into five zones on the basis of Peak Ground Acceleration values which were obtained through PSHA (Probabilistic Seismic Hazard Assessment) [60]. There are 23 major active faults which strongly influence the seismic hazard of Pakistan [80]. Building Code of Pakistan Seismic Provisions (BCP SP-07) was developed in 2007 after the devastating Kashmir earthquake 2005. These Seismic Provisions are compatible with Uniform Building Code 1997 and American Concrete Institute ACI 318-05. The structural system was designed for critical seismic parameter as per BCP SP-07 using
SAP2000, details are given in Appendix 1. Structural reinforcement details and section details are shown in Figures 4-7 and Figure 4-8 respectively.

Figure 4-1: Details of structural system

Figure 4-2: Nomenclature of the frame in the structural system
Figure 4-3: Details of Frame-1

Figure 4-4: Details of Frame-2
Figure 4-5: Details of Frame-3, Frame-4 and Frame-6

Figure 4-6: Details of Frame-5
Figure 4-7: Structural reinforcement details

Figure 4-8: Sections details of the Structural System
4.3 Validation of 6DOF Shake table and Calibration of sensors

Before the construction of the model on shake table, validation of the shake table was done because it was operated for the first time; details are given in Appendix 2. All the accelerometers and displacement transducers were calibrated before the start of tests as shown in Figure 4-9.

![Image: Calibration of Accelerometers and Displacement transducers](image)

Figure 4-9: Calibration of Accelerometers and Displacement transducers

4.4 Scale Model Exploration

Keeping in mind the shake table load carrying limitations, control issues and its operation for the 1st time in the history of Pakistan, the large scale (half scale) model was decided to be tested for capturing better and more realistic results.

Similitude relations based on dimensional analysis were done before construction as shown in Table 4-1. Similitude laws do not satisfy inelastic behavior and are very complex; therefore, Reinhorn A. M. [81] suggested using same prototype material in model and holds the same stress strain rates.
Table 4-1: Similitude Requirement for 3D Model

<table>
<thead>
<tr>
<th>Item</th>
<th>General Case</th>
<th>Scale Factor</th>
<th>Model</th>
<th>Prototype</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length, L</td>
<td>$\lambda_l = l_p/l_m$</td>
<td>2.00</td>
<td>11.5</td>
<td>23</td>
<td>ft</td>
</tr>
<tr>
<td>Area, $A = L^2$</td>
<td>$\lambda_A = A_p/A_m = l_p^2/l_m^2 = \lambda_l^2$</td>
<td>4</td>
<td>132.25</td>
<td>529</td>
<td>ft^2</td>
</tr>
<tr>
<td>Volume, $V = L^3$</td>
<td>$\lambda_V = V_p/V_m = l_p^3/l_m^3 = \lambda_l^3$</td>
<td>8</td>
<td>1520.875</td>
<td>12167</td>
<td>ft^3</td>
</tr>
<tr>
<td>Linear Displacement, $D = L$</td>
<td>$\lambda_D = l_p/l_m = \lambda_l$</td>
<td>2</td>
<td>3</td>
<td>6</td>
<td>mm</td>
</tr>
<tr>
<td>Angular Displacement, $\Theta$</td>
<td>$\lambda_\Theta = 1$</td>
<td>1</td>
<td>15</td>
<td>15</td>
<td>deg</td>
</tr>
<tr>
<td>Modulus of Elasticity, $E$</td>
<td>$\lambda_E = E_p/E_m = 1$</td>
<td>1</td>
<td>3420</td>
<td>3420</td>
<td>ksi</td>
</tr>
<tr>
<td>Stress, $\sigma$</td>
<td>$\lambda_\sigma = \sigma_p/\sigma_m = \lambda_E$</td>
<td>[\sigma = E]</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Mass, $Sm$</td>
<td>$\lambda_m = \lambda_l^3$</td>
<td>8.00</td>
<td>15</td>
<td>120</td>
<td>ton</td>
</tr>
<tr>
<td>Lump Mass, $m$</td>
<td>$\lambda_m = \lambda_l^3$</td>
<td>8.00</td>
<td>1.2</td>
<td>9.6</td>
<td>ton</td>
</tr>
<tr>
<td>Poisson's Ratio, $U$</td>
<td>$\lambda_U = 1$</td>
<td>1</td>
<td>0.2</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Strain, $\varepsilon$</td>
<td>$\lambda_\varepsilon = 1$</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Time Period, $t = (L/a)^2$</td>
<td>$\lambda_t = \lambda_l$</td>
<td>2.00</td>
<td>0.116</td>
<td>0.232</td>
<td>sec</td>
</tr>
<tr>
<td>Frequency, $\omega = \sqrt{K/m}$</td>
<td>$\lambda_\omega = 1/\lambda_l$</td>
<td>0.50</td>
<td>8.6206897</td>
<td>4.310345</td>
<td>hz</td>
</tr>
</tbody>
</table>

4.5 Construction of 3D RC model

The construction of half scaled 3D RC model comprised of various stages, starting from finalizing the similitude analysis, design layout and structural detailing to the white washing of frame for helping in detecting the crack pattern. Same materials of full scaled RC frames were used in the model as suggested by Reinhorn A. M., but the materials were scaled down, like the bricks of full scaled infilled wall were cut down into pieces according to the scale of the model in marble factory. Coarse aggregate was sieved according to the scale of the model. Some of the pictures of the materials are shown in Figure 4-10. Different stages of construction of the model are shown in Figure 4-11.
Figure 4-10: Procurement of material according to the scale of the model

(i) Bolt arrangement of RCC Pad

(ii) Framework and Steel arrangement of RCC pad
Fabricated RCC pad

Fabricated first floor RC Column

Formwork fabrication for 1st story slab

Steel fixing of 1st story beams

Steel fixing of 1st story beams

Concreting of 1st story slab
Formwork fabrication for 1st story slab
Steel fixing of 1st story beams
Steel fixing of 1st story beams
Concreting of 1st story slab
Finalized 1st story Concrete slab
Concreting of 2nd story Column
(xv) Concreting of 2nd story Column
(xvi) Formwork fabrication for 2nd story beams
(xvii) Steel fixing of 2nd story slab
(xviii) Concreting of 2nd story slab
(xix) Concreting of 2nd story slab
(xx) Finalized 2nd story Concrete slab
4.6 Instrumentation plan and Test Protocols

Thirteen accelerometers and thirteen displacement transducers were installed at different locations of the model to measure the seismic response of the model as shown in Figure 4-12 and 4-13, details of which are given in Table 4-2.
Figure 4-12: Instrumentation plan of North face of the model

Figure 4-13: Instrumentation plan of West face of the model
### Table 4-2: Details of instrumentations

<table>
<thead>
<tr>
<th>Channel No</th>
<th>Serial No of Sensor</th>
<th>Sensitivity</th>
<th>Type</th>
<th>Range</th>
<th>Location</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>North Face</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ai1</td>
<td>6514</td>
<td>492.2 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>TF3A</td>
<td></td>
</tr>
<tr>
<td>ai14</td>
<td>1708-01</td>
<td>8.5305 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±675 mm</td>
<td>TF3D</td>
<td></td>
</tr>
<tr>
<td>ai2</td>
<td>6519</td>
<td>502 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>TF1A</td>
<td></td>
</tr>
<tr>
<td>ai15</td>
<td>1708-02</td>
<td>8.5332 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±675 mm</td>
<td>TF1D</td>
<td></td>
</tr>
<tr>
<td>ai3</td>
<td>6517</td>
<td>508.9 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>MF3A</td>
<td></td>
</tr>
<tr>
<td>ai20</td>
<td>1308-01</td>
<td>138.2 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±312 mm</td>
<td>MF3D</td>
<td></td>
</tr>
<tr>
<td>ai4</td>
<td>6516</td>
<td>510.1 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>MF1A</td>
<td></td>
</tr>
<tr>
<td>ai21</td>
<td>1308-02</td>
<td>138.1 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±312 mm</td>
<td>MF1D</td>
<td></td>
</tr>
<tr>
<td>ai5</td>
<td>6515</td>
<td>501.1 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>BF3A</td>
<td></td>
</tr>
<tr>
<td>ai22</td>
<td>E1703477C</td>
<td>39.81 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±250 mm</td>
<td>BF3D</td>
<td></td>
</tr>
<tr>
<td>ai6</td>
<td>6513</td>
<td>499.9 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>BF1A</td>
<td></td>
</tr>
<tr>
<td>ai23</td>
<td>E1703471C</td>
<td>39.90 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±250 mm</td>
<td>BF1D</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>West Face</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>ai7</td>
<td>6520</td>
<td>508.5 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>TF4A</td>
<td></td>
</tr>
<tr>
<td>ai24</td>
<td>1708-03</td>
<td>68.632 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±675 mm</td>
<td>TF4D</td>
<td></td>
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<td>323</td>
<td>509 (mv/g)</td>
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<td>TF6A</td>
<td></td>
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<td>ai26</td>
<td>1708-04</td>
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<td>Displacement transducer</td>
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<td>TF6D</td>
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<td>340</td>
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<td>MF4A</td>
<td></td>
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<tr>
<td>ai27</td>
<td>1308-03</td>
<td>137.98 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±312 mm</td>
<td>MF4D</td>
<td></td>
</tr>
<tr>
<td>ai10</td>
<td>322</td>
<td>496.2 (mv/g)</td>
<td>Accelerometer</td>
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<td>MF6A</td>
<td></td>
</tr>
<tr>
<td>ai28</td>
<td>1308-04</td>
<td>170.6 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±312 mm</td>
<td>MF6D</td>
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<tr>
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<td>Accelerometer</td>
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<td></td>
</tr>
<tr>
<td>ai29</td>
<td>E1703479C</td>
<td>39.85 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±250 mm</td>
<td>BF4D</td>
<td></td>
</tr>
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<td>1357</td>
<td>5404 (mv/g)</td>
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<td>Large Accelerometer</td>
</tr>
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<td>Ai30</td>
<td>E1703475C</td>
<td>39.75 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±250 mm</td>
<td>BF6D</td>
<td></td>
</tr>
<tr>
<td>ai13</td>
<td>1358</td>
<td>5272 (mv/g)</td>
<td>Accelerometer</td>
<td></td>
<td>BCVA</td>
<td>Large Accelerometer</td>
</tr>
<tr>
<td>Ai31</td>
<td>E1703476C</td>
<td>40.01 (mv/mm)</td>
<td>Displacement transducer</td>
<td>±250 mm</td>
<td>BCVD</td>
<td></td>
</tr>
</tbody>
</table>

Fifteen cameras were installed at different locations to capture the performance of the model as shown in Figure 4-14. A strong and rigid reference frames of tube steel section were erected on the strong floor of the EEC Laboratory for fixing the accelerometers, displacement transducers and cameras at different levels as shown in Figure 4-15.

Figure 4-14: Cameras installed at different locations

Figure 4-15: Reference frames installed at West and North face of the Model
4.7 Free Vibration & Shake table Tests

Before and after performing each run of destructive test, ambient and free vibration tests were performed. Triaxial accelerometer was installed at the top center of second story slab of the model. Model was excited and the response of the model was recorded with the help of data acquisition system DR-4000 as shown in Figure 4-16. From the response history of the model as shown in Figure 4-17, the natural frequency and damping were calculated as shown in Table 4-3.

**Figure 4-16: Recording of Ambient Vibration**

**Figure 4-17: Free vibration test response history**
Table 4-3: Free vibration test results

<table>
<thead>
<tr>
<th>Testing Phase</th>
<th>North-South Direction</th>
<th></th>
<th>East-West Direction</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time Period (sec)</td>
<td>Natural Frequency (Hz)</td>
<td>Damping Ratio ζ (%)</td>
<td>Time Period (sec)</td>
</tr>
<tr>
<td>Before TR-1</td>
<td>0.16</td>
<td>6.45</td>
<td>2.92</td>
<td>0.29</td>
</tr>
<tr>
<td>After TR-2</td>
<td>0.24</td>
<td>4.21</td>
<td>5.63</td>
<td>0.34</td>
</tr>
<tr>
<td>After TR-3</td>
<td>0.37</td>
<td>2.67</td>
<td>4.01</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Natural frequency calculated before the first run was 6.45 Hz. It was calculated 6.04 Hz from SAP 2000 without infill wall; therefore, with infill wall frequency increased from 6.04 Hz to 6.45 Hz. To capture full range of the performance of the model, it was subjected to a series of sinusoidal motion as shown in Figure 4-18 with increasing frequency in three steps (3 runs), starting from 0.4, 0.5, 0.75 to 7.5 Hz with the increment of 0.25 Hz. The duration of the sinusoidal motion was kept around 20 seconds and the data was recorded for 25 seconds at a sampling frequency of 200 Hz without applying any anti-aliasing filter.

![Figure 4-18: Typical Sinusoidal Motion](image)

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4.8 Model Observed Behavior

Three test runs were performed on the model. In all these test runs the model exited only in x direction as shown in the Figure 4-9. The observed behavior of the model in each test run is discussed in the following sub sections.

4.8.1 Test Run-1

In Test Run-1 the sequence of frequencies followed as 0.4Hz, 0.5Hz, 0.75Hz to 3Hz with increment of 0.25 Hz. The amplitude of vibration was 3mm and the duration of each frequency was kept 25 seconds. As the shake table was operated for the first time, the test was stopped and the recorded data was checked. The model was inspected; there was no visible crack in the model after this test run.

Figure 4-19: Direction of excitation of the model
4.8.2 Test Run-2

The sequence of frequencies followed in this test run were 0.4Hz, 0.5Hz, 0.75Hz to 7.5Hz with increment of 0.25 Hz. The amplitude of vibration was 3mm and the duration of each frequency was kept 25 seconds.

Frame-1: In Frame-1 as shown in Figure 4-20, visible cracks at frequency of 3Hz which widened with increase in frequency occurred. Further cracks occurred at 6 Hz frequency which also widened with increase in frequency. In Panel-1, infilled wall and column joint cracked on both sides. Similarly, infilled wall and beam joint cracked. The top layer of bricks was cracked, in which some bricks in the center were push-in inside. A diagonal cracked occurred at the top right side of the opening. Similarly, a horizontal crack occurred at the top left side of the door opening. In Panel-2 infilled wall and column joint cracked on both the sides. Similarly, infilled wall and beam joint cracked. Most of the cracks in this panel were horizontal and a diagonal crack also occurred. In Panel-3, left side bricks of the first six layer fell down and the left side of the window rotated at 7.5Hz. Horizontal and diagonal cracks occurred at the right side of the window. In the Panel-4, left side bricks of the top layer fell down. Top left side of the infilled wall moved 0.5 inch in out of plane direction.
Frame-2: In Panel-1, infilled wall and column joint cracked on both sides. Similarly, infilled wall and beam joint cracked. 1st brick of the top layer fell down. Cracks in the infilled wall were shear type. Horizontal sliding along the shear crack also occurred as shown in Figure 4-21. Similarly, in Panel-2, infilled wall and column joint cracked on both sides and infilled wall and beam joint cracked. Cracks in the infilled wall were shear type. Horizontal sliding along the shear crack also occurred.
In Panel-3, infilled wall and column joint cracked on both sides and infilled wall and beam joint cracked. From the left side of top layer three bricks fell down. Cracks in the infilled wall were shear type. Horizontal sliding along the shear crack also occurred as shown in Figure 4-22.

In Panel-4, infilled wall and column joint cracked on both sides and the bricks of the top first six layer fell down. Same types of cracks occurred as Panel-3.
Frame-5: In Pannel-1, infilled wall and column joint cracked on both sides and the bricks of 1\textsuperscript{st} and 2\textsuperscript{nd} top layers of the middle portion fell down as shown in Figure 4-23. Cracks in the infilled wall were shear type. Horizontal sliding along the shear crack also occurred.

In Pannel-2, infilled wall and column joint cracked on both sides. Similarly, infilled wall and beam joint cracked.

Flexural cracks of width 0.1mm to 0.2mm formed in the entire columns at a distance of 5 inch to 6 inch from the top of the column as shown in Figure 4-24. Crack width was measured with the help of crack width detector as shown in Figure 4-24.
Figure 4-23: Crack pattern of Panel-1 and Panel-2 of Frame-5

Figure 4-24: Crack pattern of the column (left) Crack width detector (right)
4.8.3 Test Run-3

In Test Run-3 same sequence was followed as Test Run-2 but at the end amplitude was increased to 6mm.

**Frame-1:** Final deformed shape of Frame-1 is shown in Figure 4-25. Brick of Panel-1 and Panel-4 completely fell down. Lintel beam of Panel-4 was damaged but was still in its position. In Panel-3 layer of bricks remained at the bottom while in Panel-2 it remained at the side of column.

**Frame-2:** Final deformed shape of Frame-2 is shown in Figure 4-26. In Panel-1, infilled wall and column joint cracked on both sides and bricks of the upper layer fell down. Cracks in the infilled wall were shear type. Horizontal sliding along the shear crack also occurred. The crack pattern of panel-2 was the same as in Test Run-2 only width of the cracks increased and brick from top right corner fell down. Brick of the Panel-3 completely fell down. The crack
pattern of the Panel-4 was also the same as it was in Test Run-2, only width of the cracks increased.

**Figure 4-26: Final deformed shape of Frame-2**

**Frame-5:** In Panel-1, bricks from the top first eleven layers fell down, after that seven layers twisted as shown in Figure 4-27. The existing cracks of Test Run-2 widened and some new horizontal cracks also appeared in this Panel. In Panel-2, portion of the infilled wall above the lintel beam was twisted. This panel was at the edge of collapse as shown 4-28. Width of the cracks in the columns of Test Run-2 were increased in this Test Run as shown in Figure 4-29 to Figure 4-31. Plastic hinges were also formed at the upper and lower end of the columns.
Figure 4-27: Final deformed shape of Panel-1 of Frame-5

Figure 4-28: Final deformed shape of Panel-1 of Frame-5
Figure 4-29: Cracks at the top of column

Figure 4-30: Cracks at the top of column

Figure 4-31: Cracks at the bottom of column
4.9 Processing Raw Data

DADiSP (Data Analysis and Display) was used for the signal processing. It is a software package for management, display, presentation of analysis of technical and scientific data. It can simply handle data needs through interactional graphical worksheet. DADiSP includes a series based programming language called SPL (Series Processing Language) [82] used to implement custom algorithms. The response histories of the model in term of accelerations and displacements were processed in DADiSP 2002. In the first phase of data processing the required data was extracted. Extraction was such that the same extraction time was applied in all the data of accelerometers and displacement transducers. In the second phase base line correction was applied in all the data. In the third phase anti-aliasing filter was applied with cut off frequency of 10Hz (An anti-alias filter cuts off all the unwanted frequencies and passes all the suitable input frequencies). In the fourth phase noise was removed by apply moving average filter with filter taps of 6 points. In the fifth phase Power Spectral Density (PSD) and Fast Fourier Transform (FFT) were also checked in DADiSP.

4.9.1 Base Shear

According to Beyer et al [81] base shear can be calculated by three different methods during shake table test.

Method 1: In this method base shear can be calculated from the force applied by the actuator minus the inertia force of the model foundation and table of the shake table. Inertial force of the model foundation and table can be found through the accelerometer installed on it and from their masses.

\[ V_{b,act} = F_{act} - [Acc \cdot (M_{Found} + M_{Shake\ table})] \]

Where \( V_{b,act} = \) Base shear calculated from actuator,

\( Acc = \) Acceleration record of accelerometer

\( M_{Found} = \) Mass of the foundation and \( M_{Shake\ table} = \) Mass of the shake table

Method 2: In this method base shear can be calculated by summing the inertial forces of the two stories. Inertial forces can be calculated by acceleration recorded by accelerometers installed on each floor and from mass of the story.

\[ V_{b,acc} = I_{1,acc} + I_{2,acc} \]
Method 3: In this method, base shear can be calculated by the visual measurements of the markers on the slab and foundation.

\[ V_{b,mark} = I_{1,mark} + I_{2,mark} \]

In this research, Method 2 was used for the calculation of base shear. Acceleration response history recorded by accelerometer 6514 installed at the top of Frame-3 and acceleration response history recorded by accelerometer 6519 installed at the top of Frame-1 were averaged and then multiplied with second story lumped mass as shown in Figure 4-32. Similarly, acceleration response history recorded by accelerometer 6517 installed at the middle of Frame-3 and acceleration response history recorded by accelerometer 6516 installed at the middle of Frame-1 were averaged and then multiplied with first story lumped mass. The force response histories of the second story and first story were added and referred as base shear.

**Figure 4-32: Base shear Computation**
4.9.2 Story Drift

The relative displacement was determined by subtracting base displacement from the top displacement [82]. Base displacement was negative; therefore, by subtracting from the top displacement it was added to the top displacement as shown in Figure 4-33, Figure 4-34 and Figure 3-35. The blue, green and cyan colors are showing the displacement response history at the top, mid and at the bottom of the frame respectively. The torsion was predominant; therefore, Frame-3 base displacement response history was subtracted from the Frame-3 top displacement response history. Similarly Frame-1 base displacement response history was subtracted from the Frame-1 top displacement response history. Then both the relative displacement response histories were averaged. Story drift was calculated by dividing average displacement response history with the story height.

![Figure 4-33: Displacement response history of the Frame-1](image)

![Figure 4-34: Relative displacement calculation of Frame-1 at time = 215.87sec](image)
4.10 Test Specimen Response

As discussed in section 4.7.1, in the in-plane direction there was no infill wall in the Frame-3. Frame-2 was completely infilled with brick masonry and in Frame-1 there were door and window openings at different locations; therefore, there were eccentricity between center of mass and center of stiffness due to which torsion occurred in the model.

4.10.1 Test Run-1

As mentioned earlier, in Test Run-1 the model was excited with 3mm displacement for 25 seconds with frequencies \((0.4, 0.5, 0.75, 1, 1.5, 2, 2.5, 3)\) Hz. Power Spectral Density (PSD) of displacement response history from displacement transducer 1308-01 (which was installed at the mid of Frame-3) is shown in Figure 4-36, from which the frequency content can be estimated. PSD was obtained by using command line function in DADiSP. Model was within the elastic limit. Acceleration response histories from accelerometers installed at the top, mid and at the bottom of the frame were plotted on each other to check variation of acceleration at these points. Similarly, displacement response histories from displacement transducers installed at the top, mid and at the bottom of the frame were plotted on each other to check variation of displacement at these points. The over plot of acceleration response histories of Frame-3 is shown in Figure 4-37. The over plot of acceleration response histories of Frame-1 is shown in Figure 4-38. The blue, green and cyan color is showing the response history at the top, mid and at the bottom of a frame respectively. A magnified over plot of displacement response history of Frame-3 and Frame-1 is shown in Figure 4-39 and Figure 4-40 respectively.
Figure 4-36: PSD of displacement response history from DT 1308-01

Figure 4-37: Over plot of acceleration response histories of Frame-3 during Test Run-1

Figure 4-38: Over plot of acceleration response histories of Frame-1 during Test Run-1
4.10.1.1 Hysteretic curve

The hysteretic curves were plotted for first story of each frequency as shown in 4-41, by plotting first story shear against first story displacement. First story shear was calculated by averaging the acceleration response history of accelerometers 6517 and 6516 and then multiplying it with first story lumped mass. First story displacement was calculated by average of the displacement response history of displacement transducers 1308-01 and 1308-02. Similarly, by same procedure hysteretic curves were drawn for second story of each frequency as shown in Figure 4-42. Global hysteretic curves of each frequency were drawn as shown in Figure 4-43 by plotting base shear against relative displacement. Procedures for base shear and relative displacement calculation are discussed in 4.9.1 and 4.9.2. The combine global hysteretic curves of the model of each frequency are shown in Figure 4-44.
Figure 4-41: Hysteretic Curves of First Story for each frequencies
Figure 4-42: Hysteretic Curves of Second Story for each frequencies
Figure 4-43: Global Hysteretic Curves of the model of each frequencies
4.10.1.2 Energy dissipation

Energy dissipation was calculated for the first story, second story and for whole structure by finding area enclosed by the hysteretic curves as shown in Figure 4-45. Details of energy dissipation are given in Appendix 3.
4.10.2 Test Run-2

In Test Run-2, model was excited with 3mm for 25 seconds with frequencies (0.4, 0.5, 0.75, 1, 1.5, 2, 2.5, 3, 4, 5, 6, 6.5, 7 and 7.5) Hz. Power Spectral Density (PSD) of displacement response history from Displacement transducer 1308-02 (which was installed at the mid of Frame-1) is shown in Figure 4-46, from which the frequency contents can be estimated. The over plot of acceleration of left side and right side are shown in Figure 4-64 and 4-65 respectively. Acceleration at top and mid is higher than it is at the base of the model. The over plot of displacement of left and right side is shown in Figure 4-66 and 4-67 respectively. Displacement at base is not in phase with mid and Top. The base of the model is displacing in negative direction while mid and top displace in positive direction which has occurred throughout whole time history in both left and right side of north face.

![Figure 4-46: PSD of the displacement response history from DT 1308-02](image)

![Figure 4-47: Over plot of acceleration response histories of Frame-3 during the Test Run-2](image)
Figure 4-48: Over plot of acceleration response histories of Frame-1 during the Test Run-2

Figure 4-49: Over plot of displacement response histories of Frame-3 during the Test Run-2

Figure 4-50: Over plot of displacement response histories of Frame-1 during the Test Run-2
4.10.2.1 Hysteretic curve

The hysteretic curves were plotted for first story of each frequency as shown in Figure 4-51, by plotting first story shear against first story displacement. Procedures for first story shear and first story displacement calculation are discussed in section 4.10.1.1. Similarly, by same procedure hysteretic curves were drawn for second story of each frequency as shown in Figure 4-52. Global hysteretic curves of each frequency were drawn as shown in Figure 4-53 by plotting base shear against relative displacement. Procedures for base shear and relative displacement calculation are discussed in 4.9.1 and 4.9.2. The combine global hysteretic curves of the model are shown in Figure 4-54.
Figure 4-51: Hysteretic curves of first story for each frequencies
Figure 4-52: Hysteretic curves of second story for each frequencies.
Figure 4-53: Global hysteretic curves of the model for each frequencies

Figure 4-54: Combine global hysteretic curves of the model
4.10.2.2 Energy dissipation

Energy dissipation was calculated for the first story, second story and for the whole structure by finding area enclosed by the hysteretic curves as shown in Figure 4-55. Details of energy dissipation are given in Appendix 3.

![Test Run-2 Energy Dissipation](image)

**Figure 4- 55: Energy Dissipation in Test Run-2**

4.10.3 Test Run-3

Power Spectral Density of displacement response history from displacement transducer 1308-01 (which was installed at the mid of Frame-3) is shown in Figure 4-56, from which the frequency contents can be estimated. The over plot of acceleration of left side and right side is shown in Figure 4-57 and 4-58 respectively. Acceleration at top and mid is higher than it is at the base of the model. In the case of Displacement response history, before the natural frequency (which becomes 4.21 after Test Run-2) base was displaced in negative direction while mid and top displaced in positive direction as shown in Figure 4-59. They were displaced in the same direction as shown in Figure 4-60. Over plot of Displacement Response history of total duration at Right side is shown in Figure 4-61.
Figure 4-56: PSD of displacement response history from DT 1308-01

Figure 4-57: Over plot of acceleration response history of Frame-3 during the Test Run-3

Figure 4-58: Over plot of acceleration response histories of Frame-1 during the Test Run-3
Figure 4-59: Over plot of displacement response history of Frame-3 before Natural Frequency

Figure 4-60: Over plot of displacement response histories of Frame-3 after Natural Frequency

Figure 4-61: Over plot of displacement response histories of Frame-1 during the Test Run-3
4.10.3.1 Hysteretic curve

The hysteretic curves were plotted for first story of each frequency as shown in 4-62, by plotting first story shear against first story displacement. Procedures for first story shear and first story displacement calculation are discussed in section 4.10.1.1. Similarly, by same procedure hysteretic curves were drawn for second story of each frequency as shown in Figure 4-63. Global hysteretic curves of each frequency were drawn as shown in Figure 4-64 by plotting base shear against relative displacement. Procedures for base shear and relative displacement calculation are discussed in 4.9.1 and 4.9.2. The combine global hysteretic curves of the model are shown in Figure 4-65.
Test Run-3 First-Storey Hysteretic Response at 6.52 Hz

Test Run-3 First-Storey Hysteretic Response at 6.80 Hz

Test Run-3 First-Storey Hysteretic Response at 7.00 Hz

Test Run-3 First-Storey Hysteretic Response at 7.25 Hz

Test Run-3 First-Storey Hysteretic Response at 7.52 Hz

Test Run-3 First-Storey Hysteretic Response at 7.80 Hz

First-Storey Displacement in mm.

First-Storey Shear in kN.
Figure 4-62: Hysteretic curves of first story for each frequencies

Test Run-3 First-Storey Hysteretic Response at 4.20 Hz

Test Run-3 First-Storey Hysteretic Response at 5.23 Hz

Run-3 Second-Storey Hysteretic Response at 1.04 Hz

Run-3 Second-Storey Hysteretic Response at 1.32 Hz

Run-3 Second-Storey Hysteretic Response at 1.58 Hz

Run-3 Second-Storey Hysteretic Response at 1.82 Hz
Figure 4-63: Hysteretic curves of second story for each frequencies

- Run 3 - Second-Storey Hysteretic Response at 4.20 Hz
  - (S29)

- Run 3 - Second-Storey Hysteretic Response at 5.23 Hz
  - (S30)

- Run 3 - Global Hysteretic Response at 1.04 Hz
  - (G1)

- Run 3 - Global Hysteretic Response at 1.52 Hz
  - (G2)

- Run 3 - Global Hysteretic Response at 1.58 Hz
  - (G3)

- Run 3 - Global Hysteretic Response at 1.82 Hz
  - (G4)
4.10.3.2 Energy dissipation

Energy dissipation was calculated for the first story, second story and for whole structure by finding area enclosed by the hysteretic curves as shown in Figure 4-66. Details of energy dissipation are given in Appendix 3.
Figure 4-66: Energy Dissipation in Test Run-3
Chapter 5
Summary, Conclusions, Recommendations and Future Work

5.1 General

The effect of infill panel walls with and without openings on the seismic performance of reinforced concrete frames has been investigated in the form of failure pattern, strength, stiffness, energy dissipation, ductility, ductility factor, response modification factor and performance levels. Recognizing that the infill panel walls significantly affect the response of the RC frames, the characteristics of masonry and its constitutional materials were also studied.

This chapter summarizes the entire research endeavor and provides its important conclusions. At the end recommendation based on this research and recommendations for future research are provided.

5.2 Summary

Pakistan is the 6th most populous country of the world. Reinforced concrete frames with infill wall is a popular form of construction in Pakistan. Therefore, in this research six full scale RC frames and one 3D RC model were evaluated. The selection of full scaled RC frame was base of current construction practices in Pakistan and also the available testing facilities. Two types of materials are commonly used as infilled materials in RC frames that are solid brick masonry unit and solid concrete blocks. A typical layout was finalized after reconnaissance survey of the construction practices that includes a bare frame, completely infilled frame, infilled frame with door opening at the side of infill wall, infilled frame with door opening at the center of infill wall, infilled frame with both door and window openings and infilled frame with window opening at the center of infill wall.

Different tests were performed on the procured material for construction of six frames which includes bricks water absorption, bricks initial rate of absorption, brick flexural strength, brick prism compressive strength, brick prism tensile strength, concrete compressive strength and mortar compressive strength. The tests were performed according to ASTM and the results obtained were similar to the values which are adopted in the field practice.

Full scale RC frames were tested under quasi-static loading because in such type of loading arrangement loads are applied at a slow rate due to which crack propagation, collapse
hierarchy and damaged levels can be studied in details. In the test assembly, a vertical load was applied through actuator to simulate the load of the slab and live load on the frame. One actuator was set in horizontal direction to apply horizontal load and displace the frame to the specified displacement. A total of eighteen gauges were installed at various locations of the frames to record deflection at those locations.

From the recorded data hysteresis curves were plotted. Moreover, from the same hysteresis curves energy dissipation was calculated in the form of equivalent damping. Backbone curves were obtained by plotting peak load and corresponding displacement of each cycle. Equal energy principle was used to plot bilinear idealized curves of the backbone curve, from which stiffness, displacement ductility, ductility factor, overstrength factor and response modification factor was also calculated. For stiffness degradation curves were plotted for each frame. In the last of this phase performance levels were calculated by using ASCE guidelines.

In the second phase of this research, a prototype structural system (two stories-two bays) was selected after a reconnaissance survey of construction practice in Pakistan. This structural system was designed for zone IV as per Building Code of Pakistan (BCP SP-2007). Infill walls were provided at various locations with different combination of door and window openings. A scale factor two was selected for the model because of the shake table load carrying capacity and also its operation for the first time in the history of Pakistan.

Similitude relations based on dimensional analysis were done before the construction of the model. Procurement of the materials was done according to the scale of the model. Before the construction of the model, the shake table was validated with 18 tons and 48 tons service loads. All the sensors were properly calibrated. After the construction of the model on shake table, thirteen accelerometers, thirteen Displacement transducers and fifteen cameras were installed at various locations to capture the performance of the model.

To capture full range of the performance of the model, it was subjected to a series of sinusoidal motion with increasing frequency, starting from 0.4, 0.5, 0.75 to 7.5 Hz with the increment of 0.25 Hz. The duration of the sinusoidal motion was kept around 20 seconds and the data was recorded for 25 seconds at a sampling frequency of 200 Hz without applying any anti-aliasing filter.
Three Runs that were performed on the model were Test Run-1, Test Run-2 and Test run-3. Before the start of each test run, free vibration test was performed to find the natural frequency and damping of the model. In Test Run-1, the model was within elastic limit, the applied frequencies in this run were 0.4, 0.5, 0.75, 1.0, 1.25, 1.50, 1.75, 2.0, 2.25, 2.50, 2.75 and 3.0 Hz. In Test Run-2 and Test Run-3 model, vibration was up to 7.5 Hz due to which the model was entered to inelastic region. During the tests and after the tests crack patterns were observed. The recorded data was processed and analyzed in DADiSP software. Base line correction and filters were applied on the data during processing phase. Over plot of acceleration response history and over plot of displacement response history of each side of the model were plotted to check the amplification of acceleration and displacement at each story level of each test run. Power Spectral Density of each test run was plotted to check variation of each frequency. Hysteretic curves of each frequency were plotted for first story, second story and for the global structure of each test run. From these hysteretic curves, energy dissipation was calculated for first story, second story and for global model. In the last capacity curve of the model was plotted. Bilinear idealization was done for the capacity curve, for which effective stiffness and response modification were calculated for the model.

5.3 Conclusions

Conclusions from this research are given below.

In the case of bare frame beam-column joint initially cracked at 8 mm lateral displacement. Most of the cracks occurred at beam-column joint and at the bottom of the columns at a distance of one foot above the footing beam. In the case of completely infilled frame, initial crack occurred at 2.5 mm lateral displacement masonry and beam joint, followed by cracks at masonry at column joints. In the masonry of this frame shear cracking in which horizontal sliding occurred. In the case of infilled frame with side door opening at 6 mm lateral displacement a diagonal crack originated at top right side of the door opening and ended at bottom right side of the infilled panel. With door opening at the side of infill wall the vulnerability of the frames to lateral loading was increased, therefore this frame only resisted maximum 32 mm lateral displacement. In the case of infilled frame with center door opening also diagonal cracked originated at the corner of the opening. But the vulnerability was

195
decreased by providing door at the center of the infilled wall. This frame resisted a maximum of 52 mm lateral displacement. In the case of infilled frame with both door and window openings most of the cracks were diagonal. At 14 mm lateral displacement masonry panel between door and window openings cracked diagonally, with increase in lateral displacement this masonry panel fell down. In the case of infilled frame with window opening, at the initial displacement stages, minor diagonal cracks started from the window corners and prevented the separation of wall frame interface as occurred in completely infilled frame. These cracks did not affect the maximum load, but at larger displacement caused the separation of triangular piece of infilled masonry wall. All the infilled frame with and without openings performed like a cantilever beam till separation between infilled panel and frame occurred.

Hysteretic loops were symmetrical both in positive and negative loading directions in all the Frames. Lateral loadings were increased with the increase in displacement. With the generation of cracks, stiffness was decreased but still higher loads were resisted by the Frames. With repeated load cycling, the hysteretic loops became narrowed and subsequently less energy was dissipated at second and third cycle. In the case of completely infilled frame and with only window opening at the center, hysteretic loops exhibited pronounced pinching of the loops at the point of load reversal.

With the addition of infill wall damping of the RC frame increased, which further increased with the increase of lateral displacement due to generation and well distribution of cracks. But in the case of infilled frames with side and center door opening, damping decreased with the increase in lateral displacement. With the diagonal cracks, the panels were divided into two triangles and the lower triangle then did not take part in the dissipation of energy; therefore, in these two cases instead of increasing, the damping decreased with the increase of lateral displacement.

The strength increased by 142.58% by providing complete infill wall in the RC frame. Similarly, stiffness increased by 271.39%. By providing Door opening at the center instead of the side of infill wall, the strength increased by 18.56 %, but stiffness decreased by 4.38%. Strength and stiffness were related to the quantity of infilled wall used; strength increased by 66.69 %. Similarly stiffness increased by 74.78% when instead of both door and window opening only window opening was provided. Flexibility of the infilled frames increased by
providing openings in masonry infill wall due to which strength decreased and but this decrease was not in proportion to openings size.

Ductility decreased by 23.07 % by providing complete infill wall in the RC frame. By providing door opening at the center instead of the side of infill wall the ductility increased by 4.13 %. Openings increased ductility, by providing both door and window openings in the infilled frame instead of only window opening, ductility increased by 14.04%.

The IO performance level decreased by 41.53 % by providing infilled wall in RC frame, but it increased by 13.04 % by providing door opening at the center instead of the side of infill wall. The LS performance level decreased by 29.23 % by providing infilled wall in RC frame, but increased by 47.96 % by providing door opening at the center instead of the side of infill wall. In the case of infilled frame with both door and window openings and infilled frame with only window opening it decreased 7.95 %. The CP performance level decreased by 27.64 % by providing infill wall in RC frame, but it increased by 48.09 % by providing door opening at the center instead of the side of infill wall. In the case of infilled frame with both door and window openings and infilled frame with only window opening it decreased 8.08 %. In the infilled frames the presence of openings increased the level of damage, thus reduced the deformation capacity of the infilled frame.

Masonry infilled panel wall altered the global response of the structure by decreasing the natural time period or by increasing natural frequency of the structure, the natural frequency of the model without infilled walls was calculated 6.04Hz, while after infilled wall it was calculated as 6.45 Hz after free vibration test. After the appearance of cracks in the structural elements natural frequency decreased while natural time period increased, the natural frequency of the model was 6.67 Hz before the test. After Test Run-2 it became 4.21 Hz and after Test Run-3 it became 2.67 Hz.

After appearance of cracks in the infilled panel of the structure, it followed different patterns. It depends on the properties and geometry of the infilled panels. Infilled panel with door and window openings were more vulnerable to lateral loading. In the case of model Frame-1, door and window openings were severely damaged as compared to Frame-2 in which there were no opening provided. As comparison with full scaled infilled RC frame, a distinct failure mechanism occurred in these frames because of inability to scale the shape of brick
units (frog was not provided) and mortar joints despite of the same frame design and same masonry specification.

Non-uniform distribution of the infilled wall produces torsion in a structure. In the case of model displacements of the left side of the model (Frame-3 which was bare frame) was greater than the right side of the model (Frame-1 infilled with brick masonry having door and window openings).

5.4 Recommendations on the basis of this research

On the basis of six full scaled RC frames and one half scaled 3D RC, the following recommendations are made:

1. The presence of infill masonry panel strongly affects the structural response of the infilled RC frames; therefore, it is recommended that infilled frames may be considered as different types of structural systems which require specific procedure for analysis and design.

2. On the basis of comparison of results of Frame-3 and Frame-4, it is recommended that door opening should be provided at the center of the infill wall.

3. On the basis of equivalent damping coefficient comparison of Frame-2 and Frame-6, it is recommended that window opening should not be provided at the center of the infill wall, on the loaded diagonal. If it is provided near to the edge of infill wall, it may lead to the improvement of the performance as compared to the center of infill wall.

4. On the basis of crack pattern comparison of Frame-4 and Frame-5, it is recommended that lintel beam should be provided throughout the bay length.

5. Response modification factor is sensitive to geometric configuration as compared to the single value of 8.5 for concrete special moment resisting frame and 6.5 for masonry moment resisting wall frame being adopted in BCP SP-07. It is therefore recommended that a range of values depending on building geometry rather than a single value of R should be used.

6. For infilled frames with openings, the openings acted as cracks initiators with cracks propagating from the corner of the openings, therefore it is recommended that window and door opening should be fastened with metal frame.
7. Damage of infill panel wall due to in-plane load can make it vulnerable to out-of-plane load especially after the separation from the bounding frame; therefore, it is recommended to provide light reinforcement in an infill wall to prevent its out-of-plane failure.

8. During lateral loading, masonry frame joint cracked at early stages; therefore, proper attention should be provided to masonry frame joint.

5.5 Recommendations for future research work

Recommendation for future work on the basis of this research are summarized as

1. The effect of lintel beam on the seismic performance of infilled RC frame considering out-of-plane behavior should be studied.

2. The effect of different materials like metal, wood etc. for fastening the door and window openings should be studied.

3. Seismic performance of confined masonry structure should be studied.

4. The effect of aspect ratio of different openings on the performance of infilled frames should studied.

5. Concrete block masonry is also commonly used in RC frame structures; therefore, its effect on the seismic performance of the structure should also be studied.

6. Hollow brick masonry which has good performance in thermal insulation should also be assessed for their performance in RC structure during lateral loading.

7. The seismic performance of masonry infilled walls (with both types bricks and concrete block) with braces should be studied.
References


[8] Yasir Irfan Badrashi, Qaisar Ali, Mohammad Ashraf, “HOUSING REPORT Reinforced concrete buildings in Pakistan”, Earthquake Engineering Research Institute (EERI) and International Association for Earthquake Engineering (IAEE)


[28] Arton D. Dautaj, Qani Kadiri, Naser Kabashi “Experimental study on the contribution of masonry infill in the behavior of RC frame under seismic loading” Engineering Structures, DOI: 10.1016/j.enganstruct.2018.03.013


Appendix 1
Design of Prototype structure

A1.1 Introduction

This section describes the design of the Prototype structure. The model was analyzed and design using SAP2000 and then manual checks were applied to confirm that the design is according to BCP SP-07.

A1.2 Analysis and design using SAP2000

In the first step grid was defined as shown in Figure A1-1.

![Grid of the model](image)

Figure A1-1: Grid of the model
Materials were defined as shown in Figure A1-2
Sections were defined as shown in Figure A1-3

**Figure A1-3: Defining sections**
Load patterns were defined as shown in Figure A1-4. Earthquake loads were defined according to UBC 97 because BCP SP-07 has adopted the seismic provision of UBC 97 for the seismic design of buildings.

![Image of load pattern definition](image_url)

**Figure A1-4: Defining load pattern**
Load combination were defined as shown in Figure A1-5.
Model was generated as shown in Figure A1-6.
Loads were assigned as shown in Figure A1-7.
After the analysis run, deflections were checked as shown in the Figure A1-8.

Figure A1-8: Deformed shape of the model
Moments were checked. Moments of the columns at the controlling combination in kip-ft are shown in Figure A1-9 and A1-10, while moments of the beams at the controlling combination at kip-ft are shown in Figure A1-11 and A1-12.

![Figure A1-9: Columns moments](image-url)
Figure A1-10: Maximum moment and shear of the column
Figure A1-11: Beams moments
Figure A1-12: Maximum moment of the beam
Figure A1-13: Column Axial forces
Figure A1-14: Longitudinal reinforcing areas of the columns and beams
Figure A1-15: Longitudinal reinforcing areas of the model
A1.3 Base shear Calculation

Seismic and site data:
Z = 0.4 (Seismic Zone 4)  (Table 5.9, BCP SP 2007, Page 5-48)
I = 1.0 (standard occupancy)  (Table 5.10, BCP SP 2007, Page 5-48)
Seismic source type = A  (Table 5.20, BCP SP 2007, Page 5-53)
Distance to seismic source = 10 km
Soil profile type = Sd  (Table 5.17, BCP SP 2007, Page 5-53)

Average story weight (for seismic design)
Weight calculations of second story and first story are given in Table A1-1 and Table A1-2 respectively

Table A1-1: Weight calculation of second story

<table>
<thead>
<tr>
<th></th>
<th>No.</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Depth (ft)</th>
<th>Unit weight (lb/ft³)</th>
<th>Weight (Kips)</th>
<th>factor</th>
<th>Net weight (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
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<td>10</td>
<td>1</td>
<td>1</td>
<td>150</td>
<td>13.5</td>
<td>0.5</td>
<td>6.75</td>
</tr>
<tr>
<td>Beams</td>
<td>3</td>
<td>23</td>
<td>1</td>
<td>0.6</td>
<td>150</td>
<td>6.21</td>
<td>1</td>
<td>11.61</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>20</td>
<td>1</td>
<td>0.6</td>
<td>150</td>
<td>5.40</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Slab</td>
<td>1</td>
<td>23</td>
<td>23</td>
<td>0.417</td>
<td>150</td>
<td>33.09</td>
<td>1</td>
<td>33.09</td>
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<tr>
<td>Walls</td>
<td>4</td>
<td>9</td>
<td>0.75</td>
<td>10</td>
<td>120</td>
<td>32.4</td>
<td>0.5</td>
<td>16.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total Weight 67.65</td>
</tr>
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</table>

Table A1-2: Weight calculation of first story

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<th></th>
<th>No.</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Depth (ft)</th>
<th>Unit weight (lb/ft³)</th>
<th>Weight (Kips)</th>
<th>factor</th>
<th>Net weight (kips)</th>
</tr>
</thead>
<tbody>
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<td>Columns</td>
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<td>10</td>
<td>1</td>
<td>1</td>
<td>150</td>
<td>27</td>
<td>0.5</td>
<td>13.5</td>
</tr>
<tr>
<td>Beams</td>
<td>3</td>
<td>23</td>
<td>1</td>
<td>0.6</td>
<td>150</td>
<td>6.21</td>
<td>1</td>
<td>11.61</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>20</td>
<td>1</td>
<td>0.6</td>
<td>150</td>
<td>5.40</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Slab</td>
<td>1</td>
<td>23</td>
<td>23</td>
<td>0.417</td>
<td>150</td>
<td>33.09</td>
<td>1</td>
<td>33.09</td>
</tr>
<tr>
<td>Walls</td>
<td>10</td>
<td>9</td>
<td>0.75</td>
<td>10</td>
<td>120</td>
<td>81</td>
<td>0.5</td>
<td>40.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Weight 98.7</td>
</tr>
<tr>
<td>Live load</td>
<td>1</td>
<td>23</td>
<td>23</td>
<td></td>
<td>40 psf</td>
<td>21.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total Weight 119.86</td>
</tr>
</tbody>
</table>
Design base shear coefficient

Structural period was determined using Method A (UBC 97).

\[ T = C_t(h_n)^{3/4} \]
\[ T = 0.030 \times (22)^{3/4} \]
\[ T = 0.305 \text{ sec} \]

Where \( C_t = 0.035 \) (0.0853) for steel moment resisting frames
\( C_t = 0.030 \) (0.0731) for reinforced concrete moment resisting frames and
\( h_n = \text{Actual height of the building} \)

Near source factors for seismic source type A and distance to source = 10 km

\[ N_a = 1.0 \]
\[ N_v = 1.2 \]

Seismic coefficients for Seismic Zone 4 (0.4) and soil profile type Sd:

\[ C_a = 0.44 \times N_a \]
\[ C_a = 0.44 \times (1.0) \]
\[ C_a = 0.44 \]
\[ C_v = 0.64 \times N_v \]
\[ C_v = 0.64 \times (1.2) \]
\[ C_v = 0.77 \]

The R coefficient was taken on the basis of full scaled six RC frame test results:

\[ R = 7.5 \]

Calculation of design base shear coefficient.

The four equations for design base shear were used

\[ V = \frac{C_vI}{RT}W \]
\[ V = \frac{0.77(1.0)}{7.5(0.305)}(187.51) \]
\[ V = 63.12 \text{ kips} \]

But the design base shear need not exceed:
\[ V = \frac{2.5 C_a I W}{R} \]
\[ V = \frac{2.5(0.44)(1.0)}{7.5} (187.51) \]
\[ V = 27.50 \text{ kips} \]

The total design base shear shall not be less than:
\[ V = 0.11 C_a I W \]
\[ V = 0.11(0.44)(1.0)(187.51) \]
\[ V = 0.11(0.44)(1.0)(187.51) \]
\[ V = 9.07 \text{ kips} \]

In addition, for Seismic Zone 4, the total base shear shall also not be less than
\[ V = \frac{0.8 Z N_v I W}{R} \]
\[ V = \frac{0.8(0.4)(1.20)(1.0)}{7.5} (187.51) \]
\[ V = 9.60 \text{ kips} \]

Therefore, base shear = 27.50 kips

**Vertical distribution of shear**

The building period was calculated 0.305 seconds using Method A. Therefore, the concentrated force at the top was determined as follows
\[ F_t = 0.07T \]
\[ F_t = 0 \]

*(because time period is less than 0.7 second)*

The calculation of story forces and story shears is given in Table A1-3.

**Table A1-3: Story forces and shear calculations**

<table>
<thead>
<tr>
<th>Level</th>
<th>Wi (k)</th>
<th>Σ Wi</th>
<th>hi (ft)</th>
<th>Wihi (k-ft)</th>
<th>( \frac{\Sigma w_i h_i}{w_i h_i} )</th>
<th>Fi (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF</td>
<td>67.65</td>
<td>67.65</td>
<td>22</td>
<td>1488.3</td>
<td>0.53</td>
<td>14.58</td>
</tr>
<tr>
<td>GF</td>
<td>119.86</td>
<td>187.51</td>
<td>11</td>
<td>1318.46</td>
<td>0.47</td>
<td>12.92</td>
</tr>
<tr>
<td>Total</td>
<td>187.51</td>
<td></td>
<td></td>
<td>2806.76</td>
<td></td>
<td>27.50</td>
</tr>
</tbody>
</table>
Appendix 2
Validation of 6DOF shake table

A2.1 Introduction

Earthquake Engineering Center of UET Peshawar has installed its first shake table R-141 (seismic simulator) in 2005. This shake table is 1.5m x 1.5m and has single degree of freedom. Since then it has been extensively used in earthquake research. After the devastating Kashmir earthquake 2005, the Government of Pakistan strongly encouraged the expansion of EEC and the new seismic lab was completed in June 2011. In the new seismic lab, a new shake table was installed which 6m x 6m and has six degree of freedom details of which are discussed in section 3.11.3. Before the construction of the model on the shake table it properly validated with 18-ton and 48-ton service loads as shown in Figure A2-1 and A2-2, because it was operated for the first time for such a model test.

Figure A2-1: Validation of shake table with 18 ton service load
A2.2 Instrumentation Plan

In both the tests, five accelerometers and two displacement transducers were installed at different locations as listed in Table A2-1 and shown in Figure A2-3. Shake table was vibrated in south-north direction. Two accelerometers were installed in the east and west face of the shake table pointed towards south. Its purpose was to measure in-plan acceleration of the shake table. Two accelerometers were installed at north and south face on the shake table pointed towards west. The purpose of these two accelerometers were to detect twisting effect of the shake table. One accelerometer was installed at the center of shake table to measure acceleration of the shake table in the vertical direction. One displacement transducer was installed on H2 and one on H4 actuator to measure in-plane displacement of the shake table. Main purpose of installing two displacement transducer at H2 and H4 was to detect that actuators H2 and H4 are displacing at same amount.
### Table A2-1: Details of instrumentations

<table>
<thead>
<tr>
<th>Channel No</th>
<th>Serial No of Sensor</th>
<th>Sensitivity</th>
<th>Type</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>a01</td>
<td>323</td>
<td>509.0 (mv/g)</td>
<td>Accelerometer</td>
<td>AC-WS</td>
</tr>
<tr>
<td>a02</td>
<td>340</td>
<td>510.1 (mv/g)</td>
<td>Accelerometer</td>
<td>AC-ES</td>
</tr>
<tr>
<td>a03</td>
<td>6516</td>
<td>510.1 (mv/g)</td>
<td>Accelerometer</td>
<td>AC-NW</td>
</tr>
<tr>
<td>a04</td>
<td>6515</td>
<td>501.1 (mv/g)</td>
<td>Accelerometer</td>
<td>AC-SW</td>
</tr>
<tr>
<td>a05</td>
<td>6514</td>
<td>492.2 (mv/g)</td>
<td>Accelerometer</td>
<td>AC-CU</td>
</tr>
<tr>
<td>a06</td>
<td>1708-01</td>
<td>8.5305 (mv/mm)</td>
<td>Displacement transducer</td>
<td>DT-H4</td>
</tr>
<tr>
<td>a07</td>
<td>1708-02</td>
<td>8.5332 (mv/mm)</td>
<td>Displacement transducer</td>
<td>DT-H2</td>
</tr>
<tr>
<td>a08</td>
<td></td>
<td></td>
<td>Actuator</td>
<td>H2</td>
</tr>
<tr>
<td>a09</td>
<td></td>
<td></td>
<td>Actuator</td>
<td>H3</td>
</tr>
<tr>
<td>a10</td>
<td></td>
<td></td>
<td>Actuator</td>
<td>H4</td>
</tr>
</tbody>
</table>

**Notation**

- AC-WS: Accelerometer located at West face of shake table facing towards south
- DT-H4: Displacement transducer located at actuator H4

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**Figure A2-3:** Schematic diagram of instrumentation plan
A2.3 Vibration study and Results

Shake table was vibrated with different frequencies and amplitude through signal generator manually. To check the performance of shake table different combination of frequency and amplitude were applied to the shake table. Details of which are given below.

- In the initial stage frequency was kept 0.5 Hz and the amplitude of vibration was kept ±2.5 mm.
- In the second stage frequency was kept the same i.e 0.5 Hz but the amplitude of vibration was increased to ±5.0 mm.
- In the third stage frequency was increased to 1.0 Hz and the amplitude of vibration was kept the same i.e ±5.0 mm.
- In the fourth stage frequency was increased to 2.0 Hz and the amplitude of vibration was kept the same i.e ±5.0 mm.
- In the fifth stage frequency was increased to 3.0 Hz and the amplitude of vibration was kept the same i.e ±5.0 mm.
- In the sixth stage frequency was increased to 4.0 Hz and the amplitude of vibration was kept the same i.e ±5.0 mm.
- In the seventh stage frequency was increased to 5.0 Hz and the amplitude of vibration was kept the same i.e ±5.0 mm.
- In the eighth stage frequency was increased to 6.0 Hz and the amplitude of vibration was increased to ±10.0 mm.
- In the ninth stage frequency was increased to 10 Hz and the amplitude of vibration was kept ±5.0 mm.
- In the tenth stage frequency was kept 1 Hz and the amplitude of vibration was increased to ±20.0 mm.
- In the eleventh stage frequency was kept 1 Hz and the amplitude of vibration was increased to ±30.0 mm.

As already discussed, two displacement transducers (DT) of capacity ± 675 mm were installed at actuator H2 and H4. Displacement response history from DT1708-02 installed at H2 is shown in Figure A2-4 and displacement response history from DT 1708-01 installed at H4 is shown in Figure A2-5. Over plot of displacement response histories from DTs installed at H2 and H4 actuators is shown in Figure A4-6. Acceleration response history from accelerometer 340 installed at East face of shake table directed towards South (AC-ES) is shown in Figure A2-7 while acceleration response history from accelerometer 323 installed at AC-WS is shown in Figure A2-8. Over plot of acceleration response histories from accelerometers installed at AC-ES and AC-WS is shown in Figure 4-15.
Figure A2-4: Displacement response history from DT installed at H2 actuator

Figure A2-5: Displacement response history from DT installed at H4 actuator

Figure A2-6: Over plot of displacement response histories from DTs installed at H2 and H4 actuators
From the Figure A2-6 it was cleared that actuators H2 and H4 were displaced at same amount. There were no out of plane displacement in the shake table. From the Figure A2-9 it was clear that there is a slight difference of acceleration at AC-ES and AC-WS, but it was negligible.
Appendix 3

Energy dissipation details of Test Run 1-3

Table A3-1: Global energy dissipation of Test Run-1

<table>
<thead>
<tr>
<th>S No.</th>
<th>Duration (sec)</th>
<th>Frequency (Hz)</th>
<th>Energy Dissipated in Single Cycle (kN-mm)</th>
<th>Energy Input in Single Cycle (kN-mm)</th>
<th>Energy Dissipated in all Cycles (kN-mm)</th>
<th>Energy Input for all Cycles (kN-mm)</th>
<th>Damping Ratio $\zeta$</th>
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<td>1</td>
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<td>7.09608</td>
<td>200.75</td>
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<td>0.180193</td>
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Table A3-2: First story energy dissipation of Test Run-1

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Table A3-3: Second story energy dissipation of Test Run-1

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Table A3-7: Global energy dissipation of Test Run-3

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