SHAKE TABLE TESTING OF GRAVITY LOAD DESIGNED REINFORCED CONCRETE FRAMES WITH UNREINFORCED MASONRY INFILL WALLS

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ABSTRACT :

A large number of existing buildings, particularly those constructed prior to the enforcement of ductile design philosophy of 1970's, were primarily designed and detailed to resist gravity loads. Structures of this type do not have the current reinforcement detailing required by modern codes in high and medium seismic zones and, hence, they are considered potential life–safety hazards. In addition, the presence of masonry infill walls was often ignored by engineers since they are normally considered as architectural elements. However, lessons learned from past earthquakes and from several tests performed have shown that those walls tend to interact with the bounding frame when the structural system is subjected to moderate or severe earthquake ground motions and that such interaction may or may not be beneficial to the performance of the structure.
This paper presents the first part of an experimental testing program carried out at the University of British Columbia (UBC) in Vancouver, Canada testing the performance of 1/2 scale Gravity Load Designed Reinforced Concrete (GLDRC) frames with Unreinforced Masonry Walls. The first part of this testing program consisted of one monotonic loading test on an Infilled frame and two series of shake table tests, one on an Infilled frame and one on a Bare frame with the UBC Earthquake Engineering Research Facility (EERF) unidirectional shake table.

KEYWORDS: nonductile reinforced concrete , infill, lap splice

1. INTRODUCTION

A large number of existing buildings, particularly those constructed prior to the enforcement of ductile design philosophy of 1970's, were primarily designed and detailed to resist gravity loads. The most common deficiencies in the columns of Gravity Load Designed Reinforced Concrete (GLDRC) buildings are: a) low shear strength due to widely spaced and poorly detailed transverse reinforcement; and b) limited flexural strength and ductility often due to short and lightly confined lap splices at the base. In addition, the presence of masonry infill walls was often ignored by engineers since they are normally considered as architectural elements. However, lessons learned from past earthquakes and from several tests performed have shown that those walls tend to interact with the bounding frame when the structural system is subjected to moderate or severe earthquake ground motions and that such interaction may or may not be beneficial to the performance of the structure.

The demand for upgrading strategies of these buildings has become increasingly important in the last few years, especially in light of the damage observed in recent earthquakes like those in Taiwan 1999, Turkey 1999, and Pakistan 2005. Even more, according to requirements of current building codes in several countries these structures require retrofitting in order to comply with the new provisions.

2. BACKGROUND

Common construction practice before modern seismic design codes allowed GLDRC column lap splices located above the slab in each floor or above the foundation. The lap splices were typically 20 or 24 longitudinal bar diameters in length. Shear reinforcement was in the form of closed hoops with 90-degree bends and spaced at half the depth of the frame member. As a result, the section at the base of these columns is unconfined and susceptible to shear failure or bond slip of the rebar along the splice, and occasionally failing before reaching yielding of the bars in tension [Cho and Pincheira, 2006].

Researchers have also performed several tests to determine the interaction between the URM infill wall and the moment resisting frame frames infilled with concrete block masonry. For instance: Brokken and Bertero [1983] performed quasi-static cyclic and monotonic load tests on 1/3-scale of the lower 3-1/2 stories of an 11 story-three bay RC frame designed for high rotational ductility. Brittle shear failure was observed for a specimen under cyclic loading with fully grouted reinforced concrete blocks.
Mehrabi A., Shing B. [1996] did monotonic and quasi-static cyclic testing on half scale reinforced concrete frames with light shear reinforcement but without any lap splice deficiency. The frames were infilled with solid and hollow concrete block masonry. The specimen with solid concrete block showed brittle shear failure of the columns beyond 1% drift. The first observable damage of the specimen with hollow concrete block was a diagonal sliding crack in the infill, which coincided with the maximum lateral load. As the amplitude of displacement cycles increased, large slips occurred along the bed joints.
Saatcioglu M., Serrato F., Foo S. [2004] studied the response of two half-scale GLDRC frame concrete block infill assemblies involving one benchmark specimen and a retrofitted with CFRP sheets, under quasi static cyclic loading. For the unretrofitted specimen, the mode of failure of the unretrofitted specimen showed bond slip of the lap splice. This bond slip was initiated at 1.5% drift, where vertical cracks and spalling at the base of the concrete column were observed indicating bond slip due to splitting of concrete. When the specimen reached 2% lateral drift the column ties opened up and lap splice is debonded from the concrete. A significant portion of the cracked infill wall remained intact, after reaching the maximum lateral drift.

3. EXPERIMENTAL RESEARCH PROGRAM

The primary focus of this research program was to produce and document experimental data on the dynamic behavior of half scale GLDRC frame with hollow concrete block URM infill walls. For this research program ten half scale GLDRC frames with URM infill walls were constructed with identical material and reinforcement characteristics. Only one of these URM walls was built not to have any mortar interface resulting in one Bare frame with an unattached URM infill and nine Infilled frames.

This section describes the design of these specimens,and the setup for the static and shake table tests in this testing program. This paper presents the first part of this experimental testing program consisting of one monotonic loading test on an Infilled frame (Specimen #1), one series of shake table tests on one Infilled frame (Specimen #2) and one series of shake table tests on a Bare frame (Specimen #3), see Figure 1.

Specimen #1 Monotonic Loading Test Setup | Specimen #2 Shake Table Test Setup | Specimen #3 Shake Table Test Setup

Figure 1 Experimental Testing Program on Half Scale GLDRC Frames with URM Infill walls

3.1. Description of Model and Material Properties

The design of the reinforcing details for the frame members was prepared at the University of Ottawa, [Saatcioglu M., Serrato F., Foo S. 2004]. This design follows the requirements of ACI 318-[1963], and represents older buildings before modern seismic design codes. An illustration of the reinforcement details of the frame members are presented in Figure 2.

After completion of the frames, the infill walls were constructed using Mortar Type N and half-scale hollow concrete blocks, which were provided by the Masonry Institute of B.C. The concrete blocks nominal dimensions were 100x200x100 mm and a mortar interface of 19mm between the wall and the frame and bed and header joints with a thickness of 9mm.

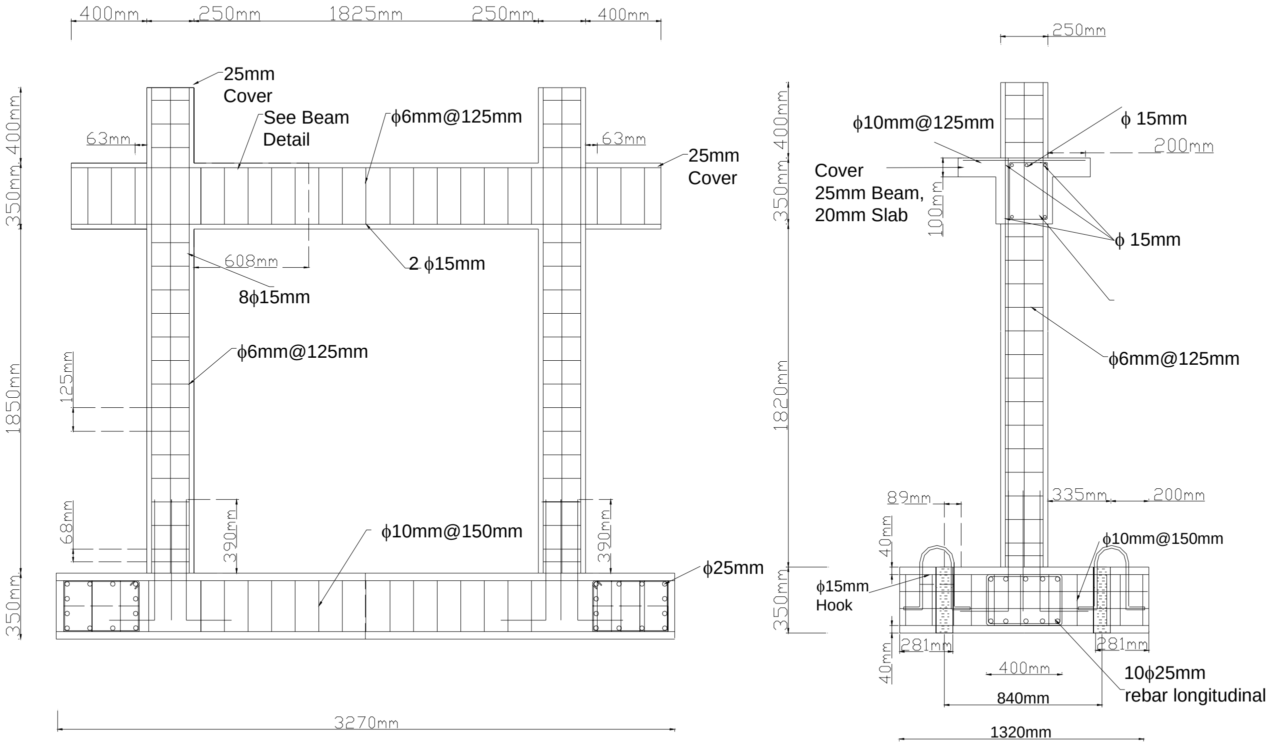


Figure 2. Gravity load designed reinforced concrete dimensions and reinforcement

3.2. Monotonic Loading Test Setup

The objective of this test was to determine the force vs displacement relation and failure mode for this structure under static loading. Specimen #1 was tested under monotonic loading until reaching a maximum drift of 3.80%.

This specimen was secured to the laboratory strong floor using four threaded steel rods and two steel HSS beams to prevent uplift, and two threaded steel rods and a steel retaining wall to prevent sliding. The system provided full fixity for the purpose of the test. No simulation of gravity loading was provided. A hydraulic actuator with a force capacity of 500 kN and a stroke capacity of 150 mm was used to apply in plane monotonic lateral loading. The actuator was connected to a steel reaction column at one end and at the other end to a loading plate connected to the top beam of the infilled frame to apply the horizontal load.

The instrumentation setup monitored the applied load by the actuator and the lateral displacements at the base of the foundation and the top of the specimen. Lateral loading was applied by the actuator in deformation control mode, as the increments of lateral drift ratios were increased.

3.3. Shake Table Test Setup

The UBC Earthquake Engineering Research Facility (EERF) uni-axial shake table allows simulation of ground motions in one direction. The dimensions of the shake table are 3000mm by 4000mm. This pump is driven by a 200 HP electric motor that can produce a rotational velocity of 1800 rpm. The shake table is displacement controlled and allows the shake table to displace +/- 450mm, with a maximum applied force of 260 kN. The hydraulic pressure controls the displacement position of the table and allows up to a maximum velocity of 75 cm/s.

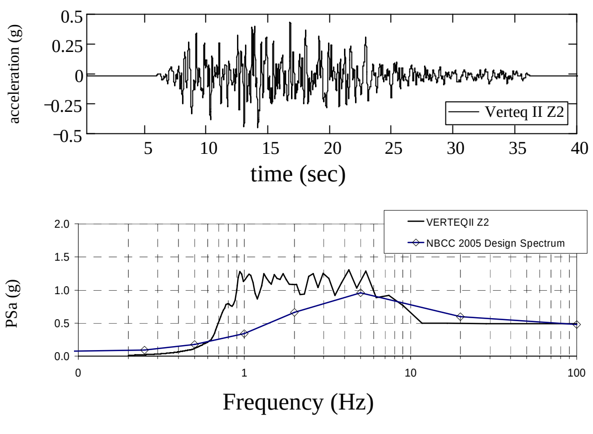
A surcharge load, added to the top of the specimen, was selected to generate inertial forces in the specimen during the simulated earthquakes. The surcharge load was achieved by the use of a set of steel plates. Each plate weighing approximately 4.45 kN. Threaded fasteners, ϕ22mm, were used to assemble the surcharge mass by passing through predrilled holes in the steel plates. The surcharge assembly was connected to the specimen slab by four threaded fasteners, ϕ38mm. A surcharge assembly of 62 kN was used for the initial nine tests for both specimens.

The test specimens were set to be subjected to different types of ground motions, increasing the seismic level from low to severe seismic demand. All acceleration records were normalized to have a peak ground acceleration of 0.5g.. The extreme-level acceleration records, called VERTEQII, were selected for this study. These records were synthetically generated with a broadband frequency spectrum and synthesized from several typical earthquakes and for different building and site soil conditions. They were originally developed for testing of telecommunications equipment [Telcordia Technologies, 1995]. The acceleration record VERTEQII Z2 is spectrally compatible with a low seismic zone and was used as the ground motion for the first nine shake table tests in the testing program, varying the amplitude as detailed in table 1. Figure 3 shows the acceleration time history and a comparison of the acceleration Response Spectrum with the NBCC 2005 design spectrum

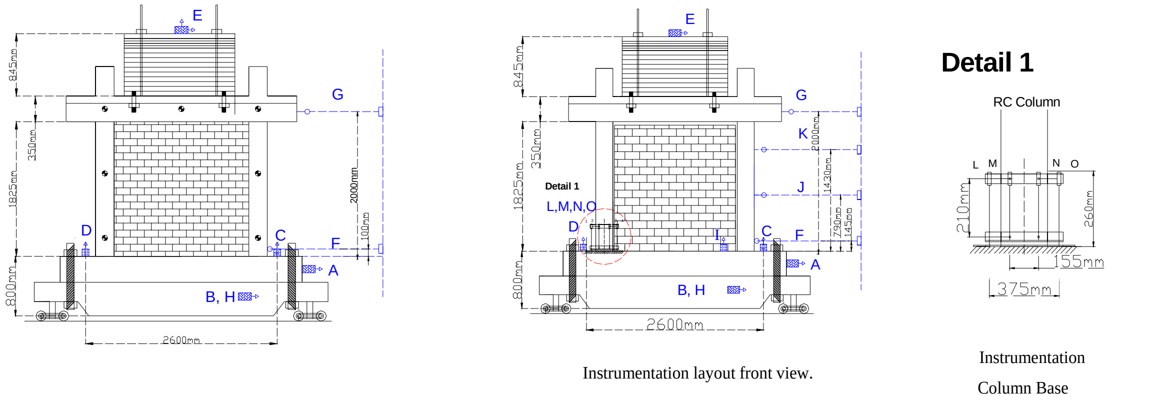
Table 1: Shake table testing protocol for specimen #2 & #3

Earthquake Record	Test #	Amplitude	PGA (g)	PGD (mm)
VERTEQIIZ2	01	50%	0.25	16
	02	100%	0.50	32
	03	150%	0.75	55
	04	200%	1.00	70
	05	250%	1.25	86
	06	300%	1.50	100
	07	320%	1.60	115
	08	340%	1.70	125
	09	400%	2.00	128

Figure 3 Acceleration Time History and Acceleration Response Spectrum for VERTEQ II Z2 record



The instrumentation for this experimental program consisted of piezo resistive accelerometers, position transducers and LVDTs to measure the lateral motion of the specimen and shake table. Figure 4 presents a layout of the location of the instrumentation on both specimens and table 2 presents the instrumentation identification.



a) Specimen #2 b) Specimen #3

Figure 4 Instrumentation Layout for Shake Table Tests

Table 2: Instrumentation Description

Instrumentation Specimen 2	
Uniaxial Piezo Resistive Accelerometer	A
	B
	C
	D
Triaxial Piezo Resistive	EX
	EY
	EZ
Position Transducer	F
	G
LVDT	H

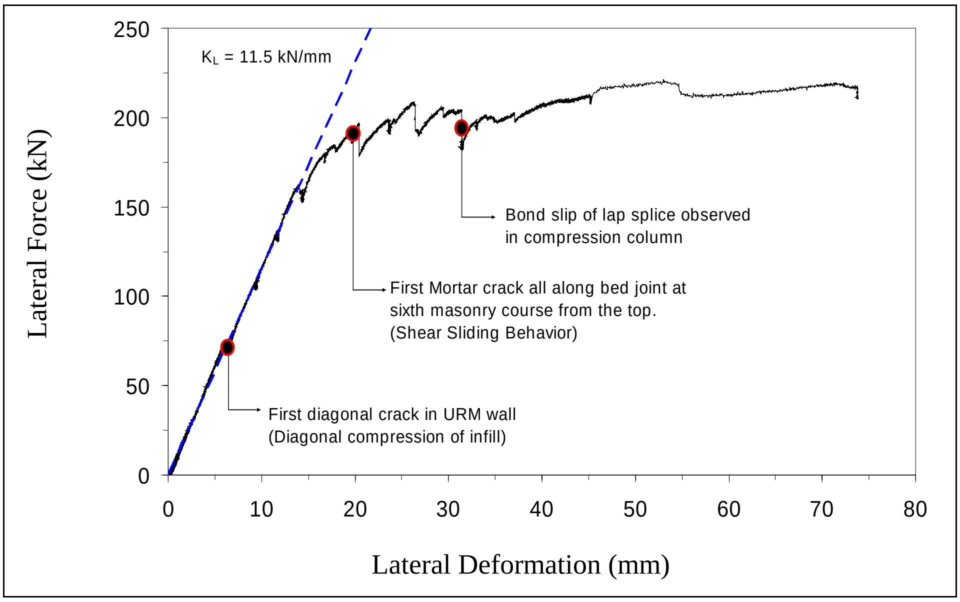
Instrumentation Specimen 3	
Uniaxial Piezo Resistive Accelerometer	A
	B
	C
	D
	E
	I
Position Transducer	F
	G
	J
	K
	L
	M
	N
	O
LVDT	H

4. TEST RESULTS

4.1. Force-Displacement Relation for Monotonic Loading

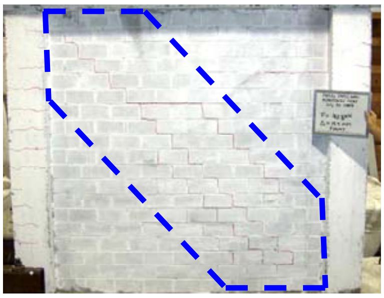
Figure 5a shows the force vs displacement relation obtained from the monotonic loading test. The linear stiffness was determined to be 11.5 kN/mm. Yielding was observed at 1.25% drift, reaching a lateral load of 205 kN. The constant slope in the force vs displacement relation, prior to yielding, indicates the contribution in stiffness of the infill is present throughout loading. This may suggest that the mortar interface transmits compression stresses to the masonry infill even under small displacement amplitudes. After yielding, there was a slight strain hardening effect reaching a maximum value of 220 kN. No strength degradation occurred before reaching the maximum tested displacement. After reaching the maximum lateral displacement a.significant portion of the cracked infill wall remained intact, showing very little damage on the masonry blocks. The specimen lateral load resisting behavior changed from a diagonal compression mode to a shear sliding behavior, to a full mechanism governed by bond slip of the rebar, as shown in figures 5b, 5c, and 5d, respectively.

There was no physical indication of the formation of equivalent plastic hinges at the base of either columns despite having reached a perfectly plastic behavior and a drift of 3.55%. Both columns, along the length of their lap splices, showed no clear development of flexure cracks, as shown in figure 5e. This suggests that the free rotation and splitting of concrete are evidence of problems due to a cold construction joint and inadequate lap splice, resulting in no development of the rebar yielding stress, as stated in the literature.

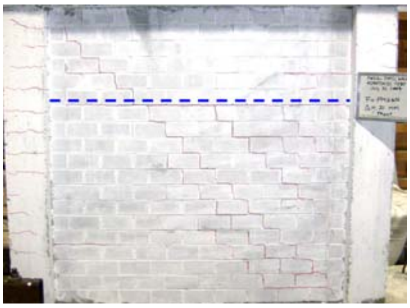


a) Force-Displacement Relation for Monotonic Loading

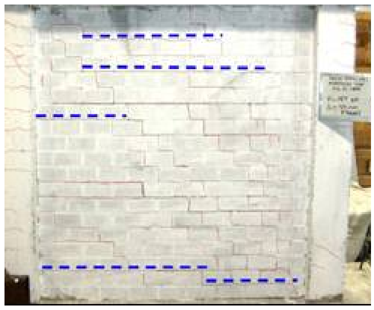
e) Evidence of rotation at the base of the compression column at 71.6mm



b) Diagonal Crack Pattern at 15 mm



c) Shear Sliding Pattern at 21 mm



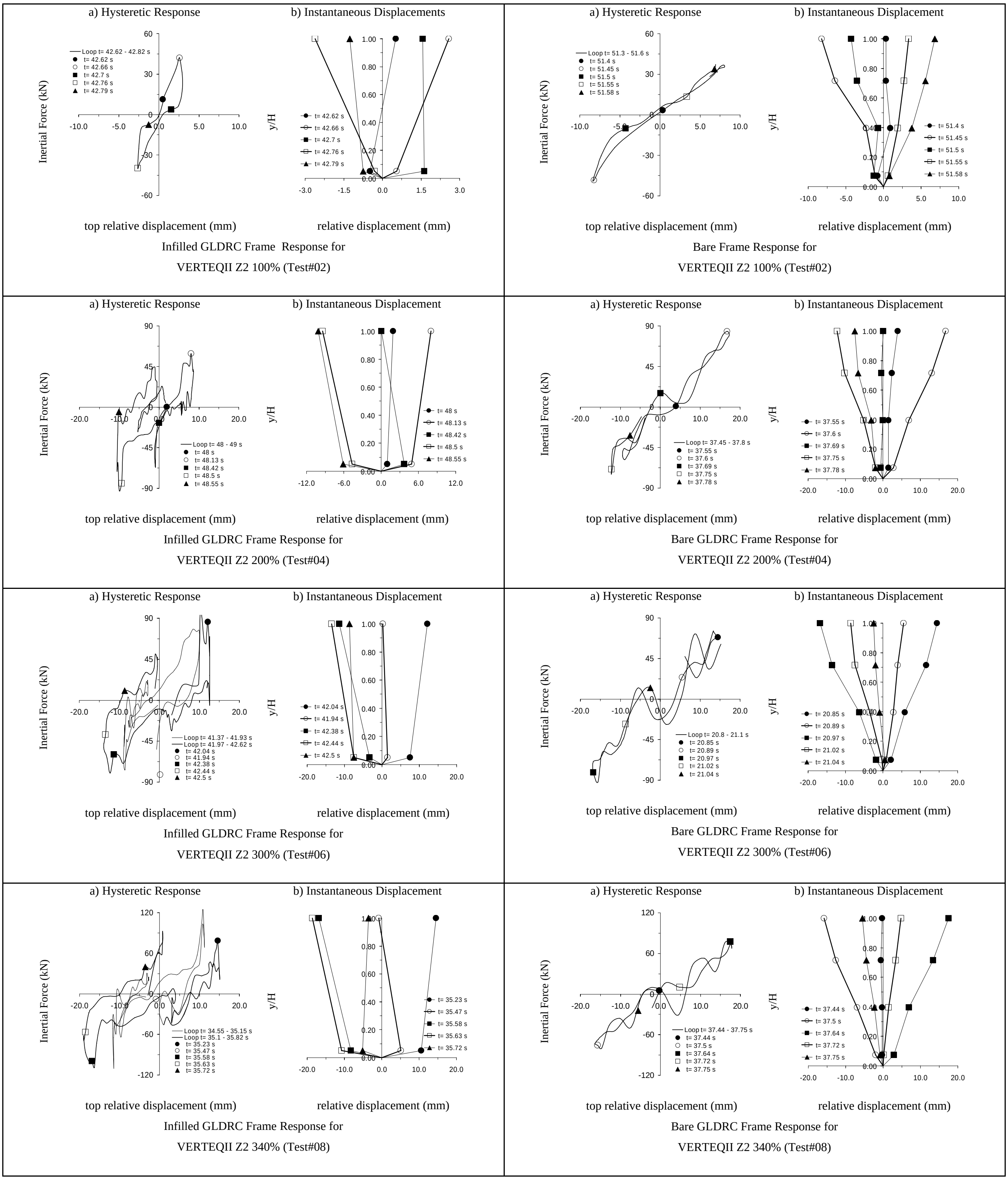
d) Bond Slip of lap splice in compression column at 33mm

Figure 5 Results from Monotonic Loading Test

4.2. Force-Displacement Relation for Dynamic Response

From the shake table tests performed on specimen #2 and specimen #3, the force vs displacement relation is determined through their measured lateral response. Figure 6 presents the hysteretic relation and the instantaneous displacement along the height of each structure for the time interval corresponding to the hysteretic loop which the highest displacements measured.

The hysteretic response of the infilled frame is shown to be highly nonlinear throughout the response for tests #01through?#08. Two opposing 45^{o} angle diagonal crack patterns formed during test #06 resulting in a significant decrease of the lateral stiffness. For tests #04, #06 and #08 the hysteretic response shows an instantaneous increase in stiffness as the structure reaches a high lateral displacement. In comparison the bare frame shows a more linear elastic behavior with higher amplitude displacements.



a) Specimen #2 Infilled GLDRC Frame b) Specimen #2 Bare GLDRC Frame

Figure 6 Hysteretic and Instantenous Displacement Response from Shake Table Tests

The instantaneous displacements shows that the infilled frame shows concentrated local lateral deformations at the lower end of the structure. These local deformations are observed to be often highest when the hysteretic response is unloading. All hysteretic plots also show a negative slope prior to unloading. These characteristics are best observed for test #02 at t=42.66s, test#04 at t=48.55s, test#06 at t=42.44s and test#08 at t=35.58s. For the response of the bare frame, smaller local deformations were also found at the lower end of the specimen but without the previously described behavior.

By comparing the instantaneous displacements of the two specimens, it can be observed that the presence of the masonry infill greatly influences the deformed shape of the GLDRC frame. The concentrated local lateral deformation at the lower end of the infilled frame is equal to 60% of its top lateral deformation, while the bare frame develops its maximum deformation distributed along its height.

5. CONCLUSIONS

A bare gravity load designed reinforced concrete frame shows little capacity to dissipate energy. The unreinforced masonry wall stiffens the frame, reducing the deformations, and allows dissipating energy through nonlinear response for several cycles of deformation. This system sustained a ground motion of 3.4 times the amplitude used for design. The first results from this testing program found consistent evidence of local deformations at the base of the gravity load designed columns when interacting with the masonry infill wall.

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