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FOREWORD

Welcome to ICEDyn 2017, in the lovely fisherman's village of Ericeira, As it happened in previous editions, the conference embraces a large spectrum of subjects that attract the interest of a wide community devoted to Engineering Dynamics, where theory and experiment follow side by side. Common to all ICEDyn conferences is the high scientific standard of the papers that have been submitted and the high reputation of the participants. Senior scientists, representatives of the industrial community and young students developing their Master or Ph.D studies are encouraged to meet and develop a healthy, stimulating and entertaining interaction around the same kind of interests and objectives.

The conference model is based on a two and a half days of technical presentations, structured into three parallel sessions. Four keynote lectures will be addressed, preceding and framing the parallel sessions that follow.

The social programme includes a reception cocktail (free for everybody), the conference banquet and an optional trip and dinner in the afternoon of the last day. Registered accompanying persons are welcome to the working day lunches, coffee breaks and conference banquet.

The village of Ericeira provides the necessary warm, peaceful and friendly environment to accommodate such an event.

Acknowledgements are due to my colleagues of the Organizing Committee, Profs. Miguel de Matos Neves and Chedas de Sampaio, to our collaborators and former PhD students Hugo Policarpo and Tiago Silva, to our secretary Mrs. Maria José Branquinho, to the members of the Scientific Committee, to the Keynote Speakers, to the Special Session developers, to the sessions' Chairpersons, to our Sponsors and naturally to all contributing authors.

The organizing committee sincerely hopes that you enjoy in our company the ICEDyn 2017.

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(Conference Chairman)
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ICEDYN 2017 OVERVIEW

Sunday, July 2nd, Afternoon

15:00-18:00 - Registration *(Room José Franco)*
19:00 - Welcome Cocktail *(in the hotel)*

Monday, July 3rd, Morning

08:30-09:15 - Late Registration
09:15-09:40 - Opening Session
09:40-12:20 - Technical Sessions
12:30-14:00 - Lunch

Monday, July 3rd, Afternoon

14:00-17:30 - Technical Sessions

Tuesday, July 4th, Morning

08:30-12:40 - Technical Sessions
12:40-14:00 - Lunch

Tuesday, July 4th, Afternoon

14:00-18:10 - Technical Sessions
19:00 – Conference Banquet *(Cocktail/BBQ + Dinner)*
(Buses leave the hotel at 18:30)

Wednesday, July 5th, Morning

09:30-12:20 - Technical Sessions
12:30 – Lunch
14:30 – Trip and dinner for registered attendees

TECHNICAL PROGRAMME

Sunday, July 2nd, Afternoon

15:00 – 18:00 – Registration

19:00 – Welcome Cocktail (at the hotel)

Monday, July 3rd, Morning

08:30	Late Registration		
09:15	Opening Session		
09:40	**Keynote Lecture #01 – *On the Use of Principal Component Analysis for Parameter Identification and Damage Detection in Structures*** Prof. Jean-Claude Golinval, University of Liège, Belgium Chair: Link, M., Univesity of Kassel, Germany		
10:30	**Coffee Break**		
	Room A – Session 1A **Special Session: Damage/SHM** (Developed by Araújo dos Santos, J., Moreno-Garcia, P., Lopes, H.) Chair: Araújo dos Santos, J.V., Instituto Superior Técnico, Portugal	**Room B - Session 1B** **Numerical Techniques/Simulations I** Chair: Rixen, D. J., Technical University of Munich, Germany	**Room C - Session 1C** **Special session: Model Based Testing and Virtual Sensing** (Developed by Herman Van der Auweraer) Chair: Cassio Faria, *Siemens Industry Software NV*, Belgium
11:00	007 ***Feature Variability in Pattern Recognition on Acoustic Emission Signals From Plastic Deformation In Carbon Steel*** Tang, J.[1,2], Mares, C.[1], Soua, S.[2] [1] Brunel University London, United Kingdom [2] Integrity Management Group, TWI Ltd, United Kingdom	032 ***Dual Craig-Bampton Method for General State-Space Models*** Gruber, F. M., Rixen, D. J. Technical University of Munich, Germany	072 ***Virtual Sensing on Offshore Wind Turbines: Long Term Experience*** Weijtjens, W., Iliopoulos, A, Henkel, M., Devriendt, C. Free University of Brussels, Belgium
11:20	033 ***Crack Detection in Rotating Shafts by using the Harmonic Balance Approach*** Cavalini Jr[1], A. A., Morais[1], T, S., Bachschmid[2], N., Steffen Jr[1], V. [1]Federal University of Uberlândia, Brazil [2]Politecnico di Milano, Italy	068 ***Dynamic Simulations of Nonlinear Systems using Piecewise Linear Mode Shapes*** Bonisoli, E., Scapolan, M. *Politecnico di Torino, Italy*	114 ***Mechatronic Powertrain Virtual Sensor: System-Level Model and Sensor Selection*** Rottiers, W.[1,2], Naets, F.[1,2], Forrier, B.[1,2], Sarrazin, M.[3], Van der Auweraer, H.[3], Desmet, W.[1,2] [1]K. U. Leuven, Belgium. [2]Member of Flanders Make, Belgium [3]Siemens Industry Software NV, Belgium
11:40	012 ***Finding Natural Frequencies with High Accuracy by Applying Sound Excitation*** Mituletu, I.C., Nitescu, C., Gillich, N. University "Eftimie Murgu" of Resita, Romania	104 ***Comparison Between Frequency and Time Domain Approaches for Estimation of Flutter Speed in High Aspect-Ratio Wings*** Afonso, F., do Vale, J. L, Oliveira, É. L., Correia, R., Lau, F., Suleman, A. University of Lisbon, Portugal	120 ***Virtual Sensing Based on Design Engineering Simulation Models*** Van der Auweraer, H.[1,2], Tamarozzi, T.[1,2], Risaliti, E.[1], Sarrazin, M.[1], Croes, J.[2,3], Forrier, B.[2,3], Naets, F.[2,3], Desmet, W.[2,3] [1]Siemens Industry Software NV, Belgium [2]K. U. Leuven, Belgium [3]Member of Flanders Make, Belgium
12:00	013 ***On the Detection of Damages in Beams in Changing Environmental Conditions*** [1]Furdui, H., [1]Ntakpe, J. L., [2], Wahab, M. A., [1]Gillich, G. R. [1]*University "Eftimie Mugu" of Resita, Romania* [2]*Ghent University, Belgium*	119 ***A New Technique for Fractional Enlargement of Integration Steps in Transient Analysis against Digitized Excitations*** Soroushian, A.[1], Zarabimanesh, Y.[1], Soleymani, K.[2], Khalkhali, S. M.[2] [1]Int. Inst. of Earthquake Eng. and Seismology (IIEES), Iran. [2]Islamic Azad University, Iran.	058 ***Aspects of Rotational Motion Reconstruction via Nonlinear State-Observers*** Faria, C T. Siemens Industry Software NV, Belgium
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Monday, July 3rd, Afternoon

14:00	**Keynote Lecture #02 – *Wave-Based Computational Approaches for Acoustics,Vibrations and Transient Dynamics*** Pierre Ladevèze, ENS Cachan, France Chair: Arruda, J. R. F., University of Campinas, Brazil		
	Room A - Session 2A **Special Session: Damage/SHM (cont.)** (Developed by Araújo dos Santos, J., Moreno-Garcia, P., Lopes, H.) Chair: Moreno-Garcia, P., University of Cádiz, Spain	**Room B - Session 2B** **Analytical Methods I** Chair: Schmidt, A., University of Stuttgart, Germany	**Room C - Session 2C** **Smart Structures** Chair: Steffen Jr., V., Federal University of Uberlândia, Brazil
14:50	**Short break to split the room**		
15:00	020 ***Structural Health Monitoring Techniques for Detecting Wind Turbine Rotor Faults*** Di Lorenzo, E., Manzato, S. Peeters, B., Marulo, F., Desmet, W. Siemens Industry Software NV, Leuven Belgium University of Naples "Federico II", Italy KU Leuven, Belgium	136 ***The Instability Constraint in the Design of Closed Loop Eigenstructures for Sensitivity Enhacement*** Bernal, D. Northeastern University, Boston	017 ***Complete Band Gaps in Two-Dimensional Magnetoelectroelastic Phononic Crystals*** Miranda Jr., E. J. P. [1,2], Dos Santos, J. M. C.[1] [1]University of Campinas, Brazil. [2]Federal Institute of Education, Science and Technology of Maranhão, Brazil
15:20	130 ***The Fatigue Failure Analysis of Composite Plate with Hole by Elastic Wave Propagation Method*** Stawiarski, A. Cracow University of Technology, Poland	101 ***Derivation of the Weak Form for a First-Order Differential Equation by Application of Fractional Calculus*** Schmidt, A. University of Stuttgart, Germany	132 ***Actuation of an Adaptive Shock Control Bump using Pressurized Chambers*** Sousa, N. A.[1], Suleman, A.[1], Kintscher, M.[2] [1]University of Lisbon, Portugal [2]Deutsches Zentrum für Luft- und Raumfahrt, Germany
15:40	009 ***Robust Output-Only Multi-Damage Localisation using Common Eigenvector Analysis and a Measure of Covariance Changes*** Shancheng Cao and Huajiang Ouyang The University of Liverpool, U.K.	108 ***Numerical and Experimental Investigation of Interface Modes in Periodic Acoustic Waveguides*** Rosa, M. I. N., Lima, V. D., Dos Santos, J. M. C., Arruda, J. R. F. University of Campinas, Brazil	
16:00	**Coffee Break**		
	Room A - Session 3A **Special Session: Damage/SHM (cont.)** (Developed by Araújo dos Santos, J., Moreno-Garcia, P., Lopes, H.) Chair: Lopes, H., Instituto Politécnico do Porto, Portugal	**Room B - Session 3B –** **Operational Modal Analysis** Chair: Brandt, A., University of Southern Denmark, Denmark	**Room C - Session 3C** **Seismic Topics** Chair: Li, Y., Case Western Reserve University, U.S.A.
16:30	074 ***Relations Among Position of Damage, Mode Shape and Derivative Order*** Moreno-García, P.[1], Araújo dos Santos, J. V.[2], Lopes, H.[3] [1]University of Cádiz, Spain [2]University of Lisboa, Portugal [3]Instituto Politécnico do Porto, Portugal	100 ***Global Scaling of OMA Modes Shapes with the OMAH Method*** Brandt, A.[1], Berardengo, M.[2], Manzoni, S.[3], Vanali, M.[2], Cigada, A.[3] [1]University of Southern Denmark, Denmark [2]Università degli Studi di Parma, Italy [3]Politecnico di Milano, Italy	097 ***Monte Carlo Simulation of Semi-Active Isolation Systems under Historical Earthquakes*** Öncü-Davas, S, Alhan, C Istanbul University, Turkey
16:50	059 *Steady State Shift Damage Localization in a Residential-Sized Wind Turbine Blade* Markvart[1], M. K., Sekjær[1], C., Bull[1], T., Johansen[1], R. J., Ulriksen[1], M. D., Tcherniak[2], D., Damkilde[1], L. [1]Aalborg University, Denmark [2]Brüel & Kjær Sound & Vibration Meas. A/S, Denmark	123 ***Advantages of using Long DFT Computation for Signal Processing in Operational Modal Analysis*** Brandt, A. University of Southern Denmark, Denmark	096 ***Seismic Risk Assessment of Spatially Distributed Infrastructure Systems: Comparing Two Approaches*** Salman, A. M., Li, Y. Case Western Reserve University, U.S.A.
17:10	060 ***Statistical Discrimination of Steady State Shift Damage Localization Metrics*** Bull[1], T., Markvart[1], M. K., Sekjær[1], C., Ulriksen[1], M. D., Johansen[1], R. J., Tcherniak[2], D., Damkilde[1], L. [1]Aalborg University, Denmark [2]Brüel & Kjær Sound & Vibration Meas. A/S, Denmark	016 ***Fundamental Natural Frequency of Integral Bridge System Considering Soil Structure Interaction Effects*** Samimifard, N.[1], Gandomkar, F. A.[2] [1]Institute of Higher Education ACECR Khouzestan, Iran [2]Islamic Azad University, Iran	002 ***The Effect of Infill Wall Frames on Seismic Performance Levels of Reinforced Concrete Buildings*** Kumbasaroglu, A., Yalciner, H., Aydin, Y. F. Erzincan University, Turkey

THE EFFECT OF INFILL WALL FRAMES ON SEISMIC PERFORMANCE LEVELS OF REINFORCED CONCRETE BUILDINGS

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ABSTRACT

Although most of the earthquake codes and engineers do not consider the effect of infilled walls on seismic performance levels of reinforced concrete buildings, infilled walls have remarkable effects during earthquakes. They mainly effect the initial stiffness and thus roof drift ratios of buildings. The aim of the present study was to examine the effect of scaled infilled wall frames on seismic performance levels of reinforced concrete buildings. An experimental study was performed to compare the different designed infilled wall systems. Three frames were tested under lateral loading by considering the impact of bond-slip relationships for different scenarios as frames without infilled walls, frames with infilled walls having anchor bars and frames with infilled walls not having anchor bars. Failure patterns for each frame was presented. The contribution of slippage of reinforcement bars to the total lateral displacement for three frames were evaluated as an indirect bond-slip relationship. It is believed that obtained results in this study would provide a guideline for the reinforced concrete buildings in seismic prone area.

KEYWORDS: Infilled walls, anchor bars, bond-slip.

1. INTRODUCTION

Even though many earthquake codes do not consider the seismic effects of infilled wall frame on structural performance at the stage of design process, those structural members have significant effects on seismic performance levels of reinforced concrete (RC) buildings. Mainly infilled wall frames improve ductility, stiffness and energy absorptions of structural system. However, not adequate anchorage infilled wall frames may cause to reduce the seismic performance levels of RC buildings due to subjected lateral loads along the length of the columns. Since the columns are designed based on the loads acting on the end edge of that members, inadequate anchorage infilled walls which may cause to add additional loads on the columns through the bottom to the end edge which are not considered during design process of the columns. Particularly, that loads may reduce the bending stiffness of the columns. Therefore, in the case of inadequate anchorage, it is also possible to have higher performance levels for the frames without infilled walls compared to the frames having infilled walls. For this purposes, there are many studies have been performed in the available literature to investigate the effect of anchorage bars on infilled walls.

One of the novel studies was done by Kumbasaroglu and Budak [1] to investigate the effect of anchor bars on steel frame systems. In that study, experimental test results showed that failure mode of anchorage and non-anchorage systems were extremely different by separation of the walls occurred at free edges. The cracking load of the infilled walls having anchorage bars four times greater than the walls not having anchorage bars. Another study done by Huang et al. [2] tested the behavior of strong frame with weak infilled wall frames. In that study among the different infilled wall system of solid clay bricks, hollow concrete blocks and aerated concrete blocks; the hollow concrete

blocks showed the most serious damage. As it was shown in the study done by Huang et al. [2], not only anchorage bars but also used different materials to construct the infilled walls effect the seismic levels of the structures.

In a parametric study was performed by Chrysostomou and Asteris [3], simplified methods for predicting the in-plane failure mode of mainly solid panels was discussed and checked against experimental results. In that study an equation was developed for quantifying the in-plane stiffness, strength and deformation capacity of infills, recommendations made for the in-plane material properties, failure modes, strength and stiffness as well as deformation characteristics of infilled frames.

In an experimental study was done by Basha and Kaushik [4], the behavior of eleven half-scale, single-story masonry infilled RC frames under slow cyclic in-plane lateral loading was studied in two stages. Results obtained in the first stage showed that the frames infilled with full-scaled and half-scale bricks exhibited higher strength, stiffness, and energy dissipation than their bare frame counterparts. In order to delay the shear failure in columns, shear design of columns was enhanced as per the existing earthquake standards, and tests were repeated on three improved frames in the second stage. Finally based on the experimental results, idealized load-displacement relationships for masonry infilled RC frames was developed for different performance levels.

In other experimental study was done by Jiang et al. [5], seven full-scaled, single-bay and single-story RC frame specimens, five with masonry infilled wall and flexible connection, one with masonry infill walled and rigid connection, and one without infilled wall were tested under low-cycle reversed loads. Based on the experimental test results, the damage behavior, lateral strength, displacement ductility ratio, stiffness degradation and energy dissipation of the specimens were compared. In that study, also the influence of constructional details of infilled walls on the seismic behavior of RC frames were analyzed. It was found that with the addition of masonry infilled wall rigidly connected to the frame, the lateral strength, the stiffness and the energy-dissipation capacity of the bare RC frame was increased significantly while the displacement ductility ratio was decreased significantly.

Many studies for the effect of anchorage bars (e.g; Yilmaz et al. [6]; Kaplan and Yilmaz [7]), effect of infilled wall (Pujol and Fick [8]; Yuen et al. [9]; Shing and Mehrabi [10]), strengthening methods of infilled walls (Grubišić and Sigmund [11]; Pujol et al. [12]), failure modes of infilled walls (Basha and Kaushik, [4]; Yuen and Kuang [13]), and also bond-slip relationships of structural members (Alsiwat and Saatcioglu [14]; Sezen and Setzler [15]) has been widely studied.

To the knowledge of the authors there has been no any study has been performed to investigate the structural behavior of different designed infilled wall frames by considering bond-slip relationships. Therefore, in this study, three different infilled wall frames were tested under lateral loading by considering the impact of bond-slip relationships for different scenarios as frames without infilled walls, frames with infilled walls having anchor bars and frames with infilled walls not having anchor bars.

2. MATERIALS AND SECTION PROPERTIES

Ready mixed concrete was used for all specimens. No any admixtures were used for the casting of concrete. The water proof plywoods were used for formworks. The recorded average 28 days of concrete compressive strength of the test specimens were 35 MPa obtained from on 150-mm cube specimens. Deformed reinforcement bars were used. Three randomly selected reinforcement bars were tested to calculate the mechanical properties of the reinforcement bars. The mechanical properties of the reinforcement bars; the yield strength was 490 MPa, the rupture strength was 600 MPa and modulus of elasticity was 200.000 MPa. The strains at yielding and rupture were 0.0024 and 0.0115, respectively. The frame details with section properties are given in Fig.1. As shown in Fig.1 one story of frame having single degree of freedom system was designed. The elongation of the columns and deformation of the beams were considered to be act as a rigid. Therefore, the lateral displacement at the level of columns-beams joints were considered to have the same lateral displacement. In Fig.1 story height of the frame was 1500 mm with a clear span of 900 mm. All columns and beams were designed as having 300 mm width and 300 mm height. The cross-sectional properties of the columns and the beams are also shown in Fig.1 As shown in Fig.1 each column was designed as having 6ϕ14 longitudinal reinforcement bars. Beams were designed as under reinforced concrete beams having top reinforcement bars of 2ϕ12 and the bending reinforcement bars were 4ϕ12. The stirrup intervals of the columns at span and densification were 17 mm and 10 mm; respectively. The intervals of the stirrups of the all beams were 17 mm at spans and 10 mm at densifications. The hooks of the stirrups were bended as 135 degree based on the described code by Turkish earthquake code 2007 (TEQ, 2007). Rigid foundations were used for all test specimens. The width and the height of the foundations were 500 mm. In this study anchor bars were designed by considering rehabilitated modes of behavior that the anchor members can

exhibit. They were embedded under favor of the approach that it was found from the available performed previous experimental studies that it can also be used (Kumbasaroglu and Budak [1], Kaplan and Yilmaz [7]). As shown in Fig. 1(c), 8 mm diameter of anchor bars were selected to provide the ratio of total area of anchor bars to the area of infilled wall to be 0.8 (Phan et al. [16]) and the placing of anchor bars at column-beam joints were done according to the conventional densification. Fig. 2 shows three different designed frames after pouring concrete.

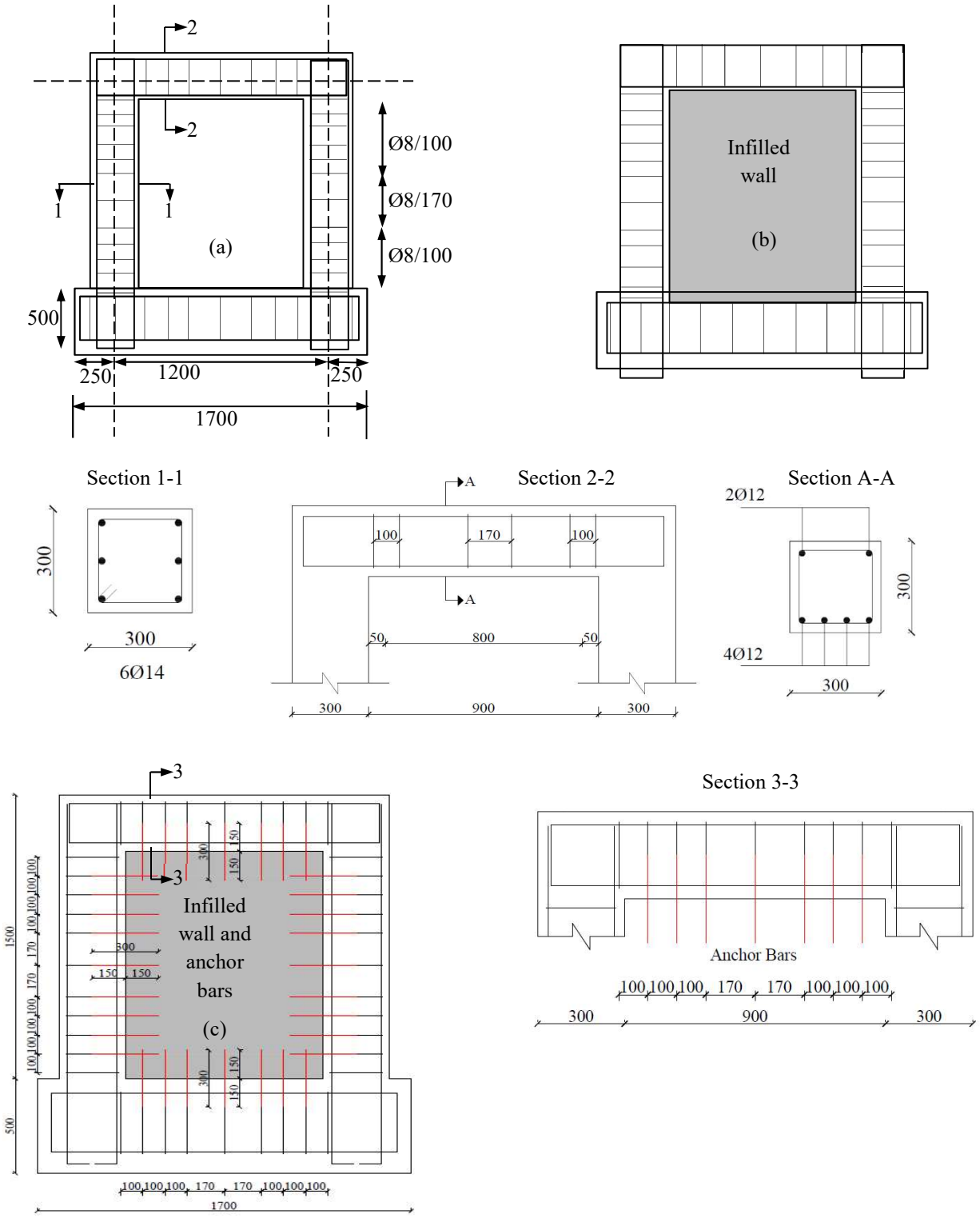


Figure 1: Frame details with section properties: (a) Frame without an infilled wall; (b) Frame with an infilled wall not having anchor bars; (c) Frame with an infilled wall having anchor bars.



Figure 2: Concrete frames: (a) Before constructing infilled walls; (b) Frame with an infilled wall not having anchor bars; (c) Frame with an infilled wall having anchor bars; (d) Frame without an infilled wall; (e) Details of a frame with an infilled wall having anchor bars before constructing weak wall.

3. TEST SETUP AND INSTRUMENTATION

The loading frame, setup system and instrumentation layout are shown in Fig. 3. In Fig. 3, the foundation of the frame was mounted to the rigid floor by using steel rods to prevent any sliding of the frames. Sixty tons capacity of a hydraulic actuator having a stroke length of 270 mm was used. The actuator was supported by a strong reaction wall. Test specimens were subjected incrementally increased lateral loading to the top edge of the column-beam joints. A load cell was used to measure the applied loads through experimental study and corresponding maximum displacements were measured by linear variable differential transformers (LVDT) which were located to the other side of frame. The length of the plastic hinges was taken according to the Park and Paulay [17] as $L_p = 0.5H$ where H is the related section depth of a member. The critical locations for the formation of plastic hinges were taken as half of the plastic hinge lengths to place the electrical-resistance strain gauges at the longitudinal reinforcement as shown in Fig. 3. Beside of critical location of plastic hinges two other locations were defined to endorse the all strain readings. All tests data then were recorded by using a data acquisition system. The relative vertical displacements measured by the displacement potentiometers on the left and right side of the segments, respectively. For the current study, experimental tests were terminated when the significant damage occurred for the specimens.

To reflect the contribution of bond-slip provided by anchorage bars, the components of the total displacement values were decomposed by considering of flexural deformation. Displacement occurred due to flexural was calculated by given Eq. (1):

$$\Delta_{flexural} = \int_0^L \phi x dx \tag{1}$$

where ϕ is the curvature along the length of the columns. For both elastic and inelastic region slippage in reinforcement bars yield as integrating the strains along the development length and can be calculated as (Sezen and Setzler [15]):

$$slip = \int_0^{l_d + l_d'} \varepsilon(x) dx \tag{2}$$

In Eq. (2), $\varepsilon(x)$ is the strain along the length of reinforcement bars, l_d is the development length of the bar in elastic region and l'_d is the development length of the bar in inelastic region. Slip rotation (θ_{slip}) was then calculated by dividing the bar slip to the depth of section and corresponding neutral axis (θ_{slip}=(slip/(d-c)). Strains along the length of the bars were recorded to calculate the slip rotations for columns at each level of loading. The lateral displacement due to slip was then calculated my multiplying the slip rotation with the height of the column (Yalciner et al., [18]). Thus, displacement occurred due to slippage of reinforcement bars were calculated by given Eq. (3); where L is the total length of the member.

$$\Delta_{slip} = (\theta_{slip}).L \tag{3}$$

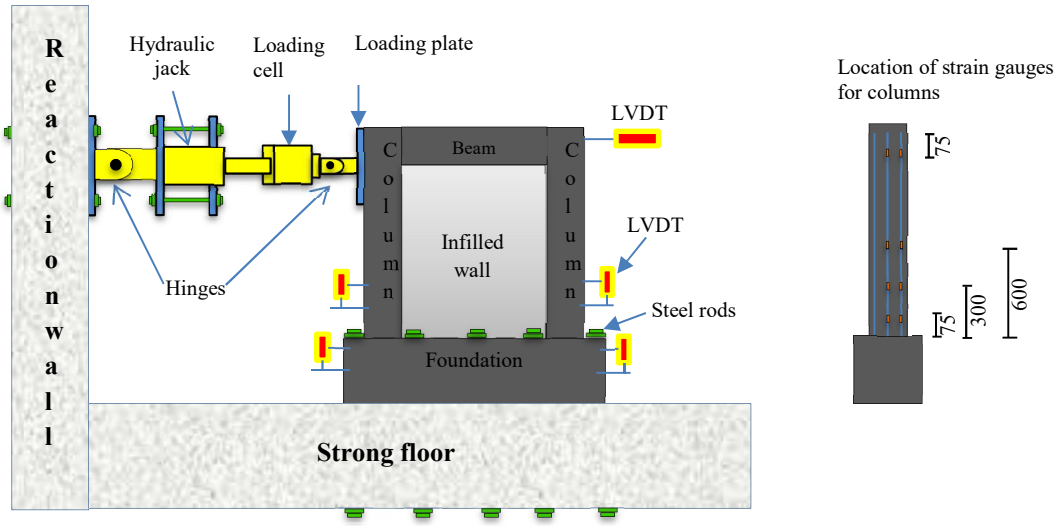


Figure 3: Details of experimental setup and instruments

4. EXPERIMENTAL TEST RESULTS

4.1 Load displacement behavior

Experimentally obtained load-displacement curves of three different designed frames are presented in Fig. 4. As shown in Fig. 4 there are significance differences regarding with ductility ratios, ultimate load capacities and energy dissipations of three tested frames. The recorded yielding capacity of the bare frame was 174 kN having a displacement of 0.03 m. The yielding capacity of the frame with an infilled wall not having anchor bars was 289 kN having a displacement of 0.05 m. These values were increased to 352 kN at 0.05 m for the frame with an infilled wall having anchor bars. With an applied load of 174 kN, in another word until the yielding point of the bare frame, the behavior of three different designed frames were acted almost same. This could be also explained by the design approach of RC members which does not consider the effect of infilled wall frames at the stage of elastic region. Although available codes found in the literature consider the elastic earthquake loads; the behavior of an earthquake is inelastic. The calculated theoretical moment resisting capacity of the columns of the bare frame was almost same as the obtained results from experimental data where premature yielding occurs for the bare frame compared with other two frames. The ductility ratio of the bare frame was 3.0 where the ductility ratios of frame with an infilled wall not having anchor bars and frame with an infilled wall having anchor bars were 1.60 and 1.40, respectively. This behavior reflects the increased rigidity and reduced ductility behavior by an over demand strength. Fig. 5 shows the crack patterns of the frames until the applied failure loads. In Fig. 5(a) the failure mode occurred particularly with the plastic hinge deformation at column-beam joints. Other obvious plastic hinge deformations occurred at the bottom columns ends based on the calculated locations of the plastic hinges with the increasing applied load to the frame. In Fig. 5(b), the initial crack form of the infilled wall frame was similar as the bare frame. Despite the separation of the infilled wall from the frame; the weak infilled wall begun to absorb the strain energy with the increasing load that prevented to occur the large shear cracking occurred at the column-beam joints for the bare frame. The recorded ultimate load capacity of bare and frame with an infilled wall not having anchor bars were 142 kN and 250 kN, respectively. As it was expected in contrast to two discussed frames, the failure mechanism of the frame with an infilled wall having anchor bars showed better performance. The ultimate recorded load capacity of the frame having anchor bars was 2.25 times of the bare frame and 1.28 times of the frame with an infilled wall not having anchor bars.

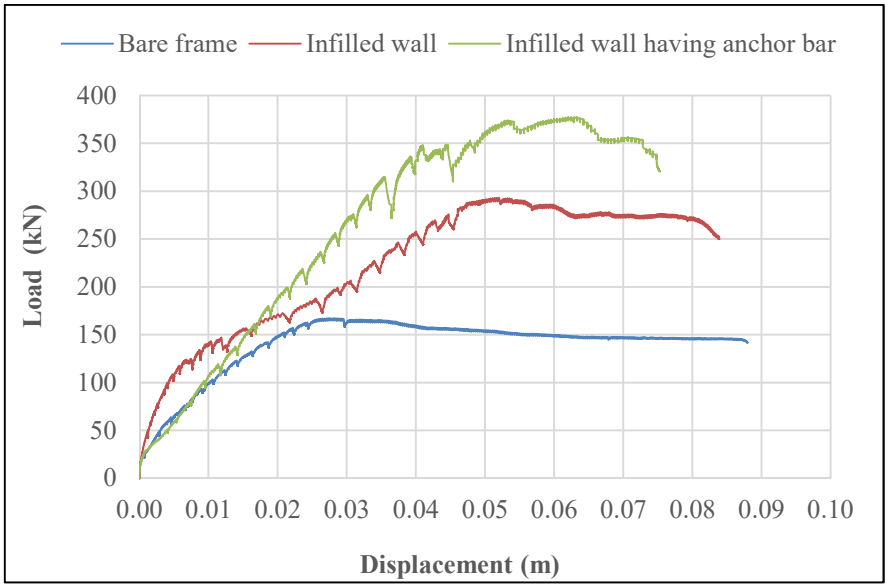


Figure 4: Load displacement curves.

In Fig. 5(c), initial cracks occurred at column-beam joints and bottom ends of the columns. With the increasing load fewer diagonal cracks begun to exist. Meanwhile, infilled wall did not separate from the frame. Thus, bearing load capacity of the frame was increased with the help of developed strain energy by anchor bars. Increased load capacity can be also explained by internal force distribution provided by anchor bars. Observed larger cracks at the column-beam joints did not occur for the frame designed by anchor bars due to the absorbed energy by infilled wall. The calculated energy dissipations of the frame without an infilled wall, frame with an infilled wall not having anchor bars and frame with an infill wall having anchor bars were 13074.94, 21083.30, 22607.19 joule, respectively.



Figure 5: Failure mechanisms: (a) Frame without an infill wall; (b) Frame with an infill wall not having anchor bars; (c) Frame with an infill wall having anchor bars.

4.2 Moment curvature relationships

Experimentally measured moment-curvature relationships were compared for three frames along the length of the columns. Addition of measured load-displacement relationships, moment-curvature relationships identify the ductility of a member based on sectional ductility. Obtained moment-curvature relationships were parallel to the behavior obtained for load-displacement curves. The ductility ratio of the bare frame was more than the two other frames having anchor bars and not having anchor bars. The calculate sectional ductility ratios of the bare frame, infilled wall not having anchor bars and infilled wall having anchor bars were 5.6, 4.7, 1.75, respectively. The infilled wall frames with improved shear capacity enhanced the load carrying capacity. It is clear that lower yield strength provided with bare frame improved the ductility ratio compared with other two infilled wall frames. This is why many earthquake codes define the over strength reduction and ductility reduction factors to provide more ductile structure during an earthquake according to the design purposes of the structures. Such as an earthquake response reduction factor was defined as eight by Turkish Earthquake Code [20] for the structure having a life safety performance levels having an exceedance probability of 10% in 50 years. As shown in Fig. 6, if the response reduction factor which was commonly employed in many seismic design codes was not considered during the design approach of a structure, it would not be possible to obtain a performance level of a structure compared to bare frame due to the linear elastic force demand.

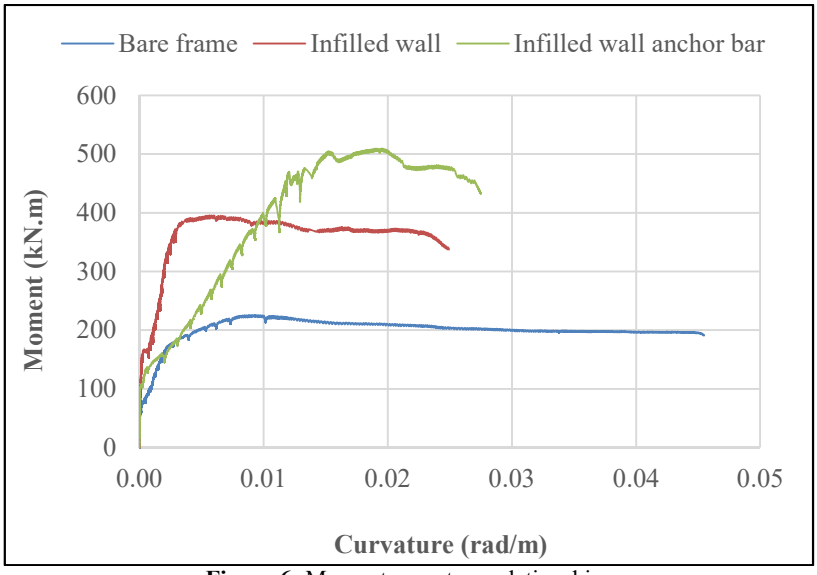


Figure 6: Moment curvature relationships

4.3 Slippage of reinforcement bars

Recorded strains were used to calculate the slippage of reinforcement bars by using Eq. (2). It should be noted that recorded curvatures for the current study was obtained from the bottom edge of the columns. Therefore, for further studies it may also be considered top region of the columns to separately calculate the slip displacements of the reinforcement bars which yields the Eq. (3) to summations of rotations based on top and bottom of the columns and multiplying total length of the columns. Figs. 7(a-b) shows the calculated slippage of reinforcement bars and displacements due to slip of three different designed RC frames corresponding to the applied lateral loads.

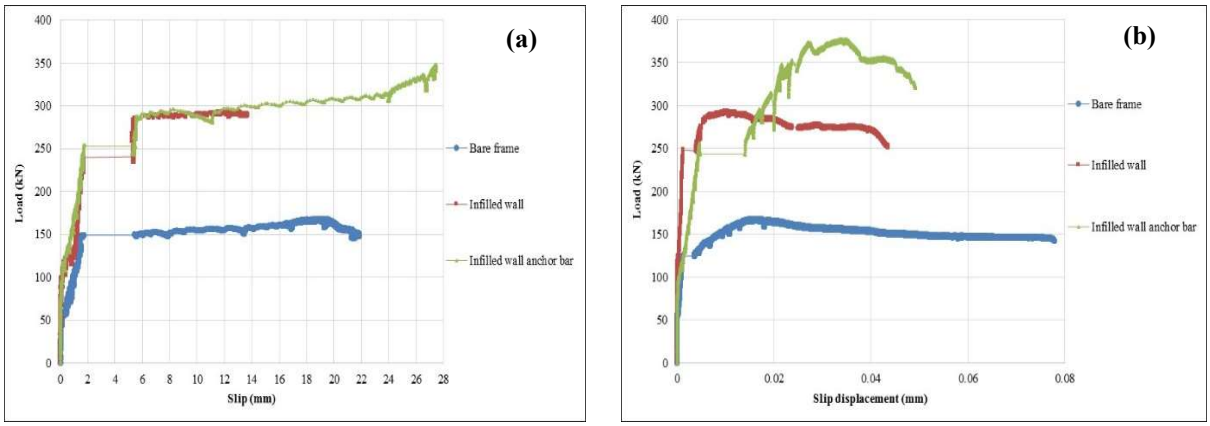


Figure 7: (a) Load-slippage; (b) Load-slip displacement

As shown in Fig.7(a), after elastic plateau there were visible smooth region occurred for the slippage of reinforcement bars. This region was also explained in the available literature and there are available suggestions (e.g., Sezen and Setzler [15]) to modify the strain hardening region by using quadratic function. In Fig.7(a), slippage almost linearly increases along the length of the reinforcement bars until the reinforcement bars reach to the strain hardening region. At this point, sudden jumping of the slippage occurs having same stresses in the reinforcement bars. Since the slope of the elastic modulus is very smooth having constant stresses in the steel bar at hardening region it causes to have also linearly increasing the slippage until to the inelastic region. At inelastic plateau with the contribution of the resisting stresses by reinforcement bars slip values were increased for three designed RC frames. When the slippage of three different designed frames were compared; in contrast to previous obtained experimental test results of load-displacement curves (see figure 4) and moment-curvature relationships (see figure 6); test results were different for the frame with an infill wall having anchor bars. By using Eq. (3) calculated maximum displacement due to slippage of reinforcement bars were, 50.61 mm, 12.49 mm, 32.22 mm for the bare frame; frame with an infilled wall not having anchor bars and frame with an infilled wall having anchor bars, respectively. Calculated curvatures and slippage of reinforcement bars converted into displacements and Fig.8 shows the contribution of curvatures and slip displacements to the total lateral displacements.

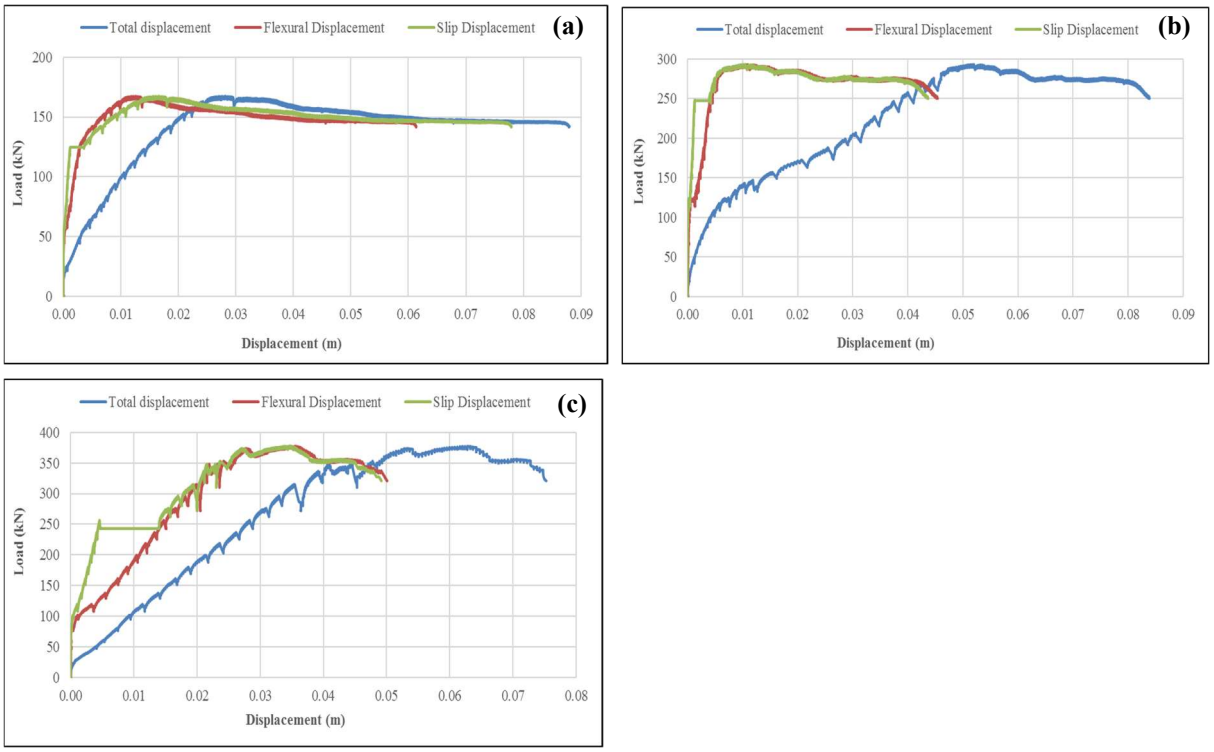


Figure 8: Comparisons of displacement values: (a) Bare frame; b) Frame with an infilled wall not having anchor bars; (c) Frame with an infilled wall having anchor bars.

In Fig. 8(a), in the case of bare frame the contribution of the slip displacements were more than the contribution provided by flexural displacement. It is believed that although design approach of RC members assumed exact bond strength between concrete and reinforcement bars, slippage of reinforcement bars may also provide ductile behavior. Obtained experimental test results also support these phenomena. In Fig. 7(b), recorded strains along the length of the reinforcement bars and calculated slip rotations thus obtained slip displacements of the bare frame was more than other two frames. Higher slip displacements require more rotations due to slippage of reinforcement bars. This behavior can be obtained only if neutral axis of a section decreases. Slip displacement of the bare frame was increased with the help of decreasing neutral axis of a section which showed more ductile behavior. In Fig. 7(a), although obtained slippage of infilled wall having anchor bars was more than the bare frame, slip displacements of infilled wall having anchor bars was less than the bare frame. Having higher slippage of reinforcements bars but obtained less slip displacements for infilled wall having anchor bars can be explained by only if neutral axis of a section increases. In can be concluded that all these observed behaviors were under the control of neutral axis based on the obtained tests results. The comparisons between infilled wall having anchor bars and not having anchor bars also sign the neutral axis. In Fig. 7(b) slip displacements of these two frames were almost same at different applied loads. However, the slippage of infilled wall having anchor bars were more than the infilled wall not having anchor bars (see figure 7(a)). Therefore, to be able to have the same slip displacement

at different slippage of reinforcement bars require well-matched slip rotations. This could be provided if only while the neutral axis of the infilled wall having anchor bars was increasing the neutral axis of the infilled wall not having anchor bars was decreasing against to infilled wall having anchor bars.

5. CONCLUSIONS

An experimental study was performed to investigate the structural behavior of infilled wall frames. Based on the test results, the following conclusions can be drawn:

1. Compared with the bare frame, lateral strength, energy-dissipation capacities of infilled wall frames were increased. Frame with an infilled wall having anchor bars showed the higher performance for the mentioned parameters.
2. Compared with infilled wall frames the displacement and curvature ductility ratios of the bare frame remarkable was higher.
3. Displacement due to slippage of reinforcement bars shows nonlinear behavior which is affected by changes in the neutral axis of a section.
4. Although obtained bars maximum slippage of reinforcement bars of infilled wall having anchor compared with the frame with an infilled wall not having anchor bars and the bare frame, slippage of reinforcement bars of infilled wall having anchor bars could not provide more displacement due to slippage as much as other two frames provided.
5. Type of connections with the consideration of slip displacements are needed to be considered for better and more accurate prediction of performance levels of RC buildings. Not only connection type but also material properties of infilled walls are needed to be considered better design approach of RC buildings.
6. Even though many modern codes and design approaches consider exact bonding between steel and concrete with involving many uncertainties, displacement due to slippage cannot be neglectable which has serious impact on structural performance levels of RC buildings.

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(Conference Chairman)