Seismic Performance of Wall-Slab Joints in Industrialized Building System (IBS) Under Out-Of-Plane Reversible Cyclic Loading

N. H. Abdul Hamid and M. A. Masrom

***Abstract*—This paper presents the performance of a full-scale wall-slab joint in tunnel form system subjected to lateral cyclic loading. The objectives are to determine seismic behavior of the hysteresis loops, strength capacity, stiffness, ductility, damage states and modes of failure. Wall-slab joint was tested up to failure drift of 2.5%. Theoretical equations were developed to calculate ductility and moment resistance of wall section. From visual observation, a lot of cracks were occurred at joint and upper part of wall panel. Spalling of concrete was observed at the top part of the joints and fractured of longitudinal reinforcement bar when the wall is pushing under out-of-plane direction. Validation between the theoretical values and experimental results were compared. There was a good agreement between them. Therefore, the moment resistance and ductility of the joint were determined and designed accordingly.**

***Index Terms*—Damage states, ductility, strength, stiffness.**

I. Introduction

Currently, the construction industry in Malaysia is shifting from conventional method toward modular system which known as Industrialized Building System (IBS). IBS is the construction system where most of the structural elements such as beams, columns, walls, slabs and staircases are prepared in the factories/plants and assemble at site within a short period of time. Precast structural elements are transported to site for erection using lorries, cranes and heavy machineries. Tunnel form system is one type of Industrialized Building System widely used in construction of precast reinforced concrete buildings. This system has been implemented in the construction of houses and condominiums either in seismic or non-seismic regions. This technique uses wet concrete at site and pours into two half-tunnel forms steel mould to make load bearing wall and floor slab, simultaneously. It becomes more popular as compare to conventional method in erecting medium to high-rise RC buildings.

One of the major issues addresses in designing of high-rise RC building is to determine the capacity of RC structures subjected to lateral force consists of wind and earthquake loads. However, wind load is not a major problem in Malaysia because wind load is too small as compare to earthquake load. Many codes of practice were developed to accommodate wind load factor in determining structural integrity and stability of the RC buildings.

Meanwhile, seismic load is always impair to the building structure and cause the holocaust to buildings either partial or full collapse.

A lot of past earthquakes events in Sumatra caused tremor to the people who live in high-rise RC buildings in Malaysia. It was reported that many Malaysian especially those who stay in high-rise buildings felt the swaying of the buildings during these earthquakes. It was discovered that through building inspections, there were about 30 percent out of 65 buildings in Kuala Lumpur, Putrajaya, and Klang are very vulnerable to earthquake. The main reason is that these RC buildings were designed in accordance to British Standard (BS 8110) where there is no provision for earthquake loading.

The initial work and analysis of tunnel form building was conducted by Balkaya and Kalkan [1]. Followed by Yuksel and Kalkan [2] who were carried out experiment work using tunnel form building by testing its under quasi-static cyclic lateral loadings. They discovered the wall-slab interface suffered severe damages after the testing. In the RC building, the crucial zone in determining the stability of the building is the seismic performance of joint in beam-column, wall-foundation, wall-slab and slab-beam [3]. The reinforced concrete joints should have sufficient strength to resist the induced stresses and sufficient stiffness to control undue deformations. Large deformations of joints result in significant increase in the inter-storey displacement. Basics seismic design requirements for RC buildings are to avoid any collapse of the structures under strong earthquake and remain functional under low earthquake excitations [4].

There are three basic minimum parameters need to be fulfilled in seismic design of RC buildings in medium and high seismic regions. The first parameter is the ductility of structures starting from elastic to inelastic behavior which can be measured in term of displacement, strains and curvature. The amount of reinforcement bars in concrete is very important in determining the ductility of structures [5].

The second parameter is the stiffness of the structure which can be classified as brittle or flexible [6]. Brittle structure having greater stiffness proves to be less durable during earthquake while ductile structure performs well in earthquake. The brittle members need strong enough to withstand the lateral force. This force induces by yielding of the ductile members, allowing a suitable margin to give a high level of confidence that the brittle elements will not reach their failure loads [7]. Predominantly, RC buildings in

Manuscript received October 22, 2011; revised December 31, 2012.
N.A. Abdul Hamid and M.A. Masrom are with Faculty of Civil Engineering, University Teknologi MARA, Shah Alam, 40450, Selangor, Malaysia (e-mail: norha454@salam.uitm.edu.my; jej_1606@yahoo.com.my)

Malaysia were designed without considering the seismic loading in RC buildings. Members in the structure should have adequate strength to carry the design loads safely. It should be pointed out that the designer should avoid brittle type of failure, by making a capacity design [8].

The third parameter is the capability of the structures to absorb earthquake energy during ground motion. The construction materials such as concrete, steel and timber are capable to absorb earthquake energy up to 5% only [9]. However, structures with base isolation system or active/passive damper can absorb the energy up to 20% [10].

Most of civil engineers assume that earthquakes will not happen in Malaysia as compare with Indonesia which located closed to the Pacific Ring of Fire. However, they cannot overlook this matter because Kuala Lumpur is located 450 km away from Sunda Plate. This plate is one of the most active plates in the world with velocity movement of 70mm/year. Furthermore, there are a few sleeping fault lines in West Malaysia such as Kuala Lumpur Fault, Bukit Tinggi Fault and Kenyir Fault which cause very small magnitude of earthquake between 2.8 to 4.2 scale Richter.

Therefore, the buildings in Malaysia are susceptible to damage and risk of collapse if bigger earthquake happened in the neighboring countries or in Malaysia. Due to that situation, the aim of this research is to perform wall-slab connection designed according to BS 8110 and tested under earthquake loading. It is important to conduct an experiment work in order to give the real scenario of the overall behaviour of RC buildings during minor and major earthquake. Thus, this paper is focused on the seismic performance of wall-slab connection in IBS (industrialized building system) subjected to reversible out-of-plane lateral cyclic loading.

II. Desig n Of Wall-Slab Connection

The specimen comprises of reinforced foundation beam, wall and slab as shown in Fig. 1. The length of foundation is 1800mm, 900mm width and 400mm height. Meanwhile, the height, width and thickness of wall panel which is seating on foundation beam are 1500mm, 1000mm and 150mm, respectively. The width, thickness and length of the floor slab are 1500mm, 150mm and 2000mm, respectively. The diameter of longitudinal reinforcement bars for the foundation beam are 16mm and diameter of transverse reinforcement bars are 12mm. The fabric wires mesh (BRC-7) is with dimension of 200mm vertically and 100mm horizontally were used in wall and slab as double layer of wires mesh. The lapping bars from foundation beam and wall are designed as fixed joint and comprises of fixed moment and shear force.

III. Theoritical Background

The theoretical background of wall-slab joint is similar approach for beam-column joint which is known as Strain Compatibility Approach. Under this approach, stress-strain relationships of the concrete and reinforcement bars are determined and analyzed. Fig. 2 shows the cross-section of wall together with reinforcement bars and curvature of the strain at yield point and ultimate state. The performance of wall panel under earthquake excitation can be measured in terms of ductility, strength, stiffness and stability. Ductility of a wall is normally determined for a particular cross section by taking into account the yield displacement, ultimate displacement, yield load and ultimate load of the structures.

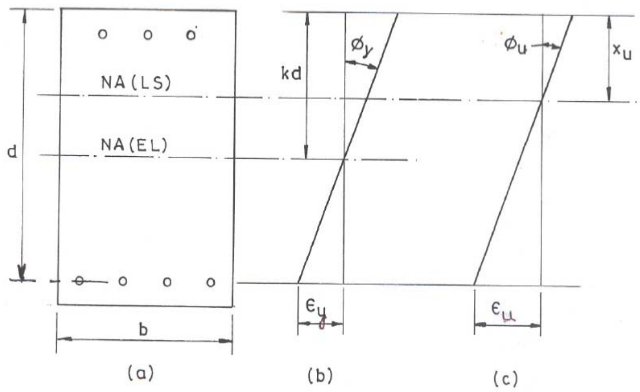

Fig. 2. (a) Cross section of wall, (b) Strain at yield state, (c) Strain at ultimate state

The derivation of ductility (μ) in terms of curvature based on Fig. 2 is defined as

$$\mu = \frac{\phi_u}{\phi_y} \tag{1}$$

$$\phi_y = \frac{\varepsilon_y}{d - kd} \tag{2}$$

where $\varepsilon_y = \frac{f_y}{E_s}$

$$k = -mp + \sqrt{m^2 p^2 + 2mp} \tag{3}$$

$$m = \frac{280}{3\sigma_{cbc}} = \text{modular ratio} \tag{4}$$

where σ_{cbc} = permissible stress of concrete in bending compression and similarly,

$$\phi_u = \frac{\varepsilon_u}{x_u} \tag{5}$$

where ε_u = ultimate strain of concrete = 0.0035

$$\frac{x_u}{d} = \frac{0.87 f_y A_{st}}{0.36 bdf_{ck}} = \frac{0.87 f_{yp}}{0.36 f_{ck}} \le \left(\frac{x_{u.\max}}{d}\right) \tag{6}$$

By substituting Equations (2) and (5) in Equation (1), the ductility can be derived in terms of strain, effective depth of wall and neutral axis of the wall as follows:

$$\mu = \frac{\varepsilon_u}{\varepsilon_y}\left(\frac{d - kd}{x_u}\right) \tag{7}$$

$$\mu = \frac{\varepsilon_u E_s}{f_y}\left(\frac{1 + mp - \sqrt{m^2 p^2 + 2mp}}{x_u / d}\right) \tag{8}$$

The experiment result was validated by comparing with calculated moment resistance according to the equation (8). The theoretical value of ductility in equation (8) is derived based on stress diagram as shown in Fig. 3.

$$M = 0.45 f_{cu} b(0.9(0.5h - 0.45x)) + AB(0.5h - 0.5AB) + 0.5BC(0.5h - AB - BS/3) - 0.5CD(0.5h - AB - BC - 2/3CD) + DE(0.5h - AB - BC - CD - 0.5DE) \tag{9}$$

where $f_y = 678 N/mm^2$ (from tensile test), Young's Modulus, $\mathbf{E_s}$ = 200 x 10^{-3} N/mm^2 then the strain ε_y when the stress is 0.95fy is given by

$$\varepsilon_y = 0.95 f_y / (200x10^{-3}) = 3.22x10^3 \tag{10}$$

By considering the equilibrium of forces in composite action in the Fig. 3, it has given the moment resistance of the section as:

If the maximum compressive strain in concrete is 0.0035 and the neutral axis depth is x, the strain in steel is to $\boldsymbol{\varepsilon_y}$ at depth c from the neutral axis,

where

$$c = (\varepsilon_y / 0.0035)x = 0.9201x \tag{11}$$

$$x - c = x - 0.9201/x = 0.0799x \tag{12}$$

Hence,

AB =0.0799x, BC = 0.9201x,
CD = 0.9201x, DE = (h – 1.9201x)

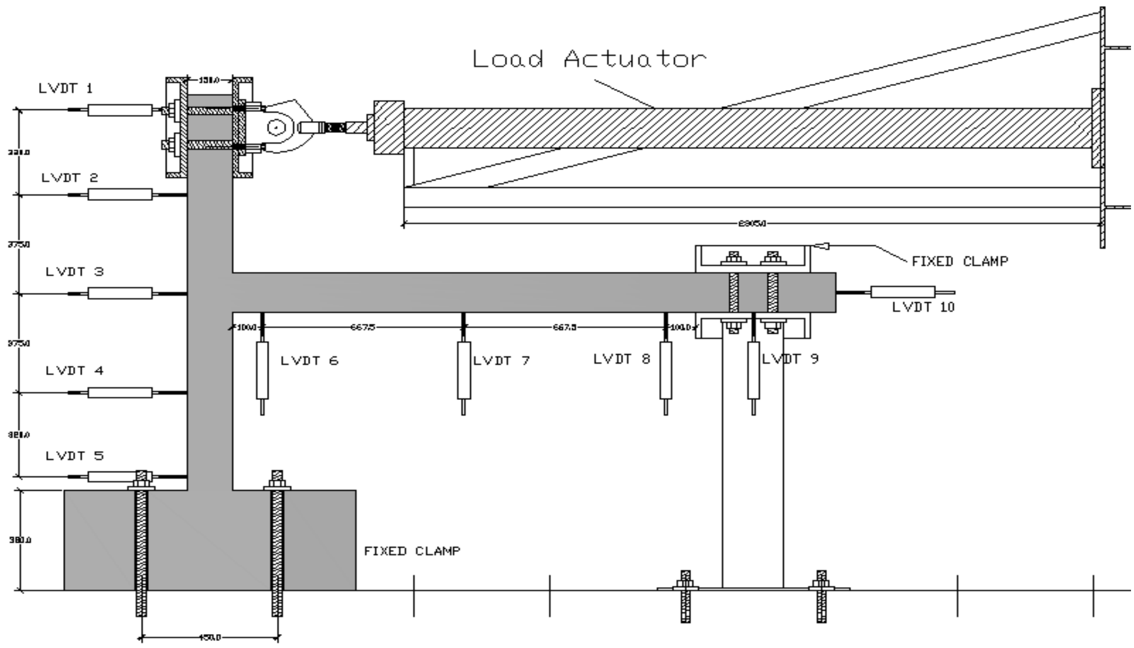

Fig. 1. Sample of wall-slab connection together with positions of LVDT's

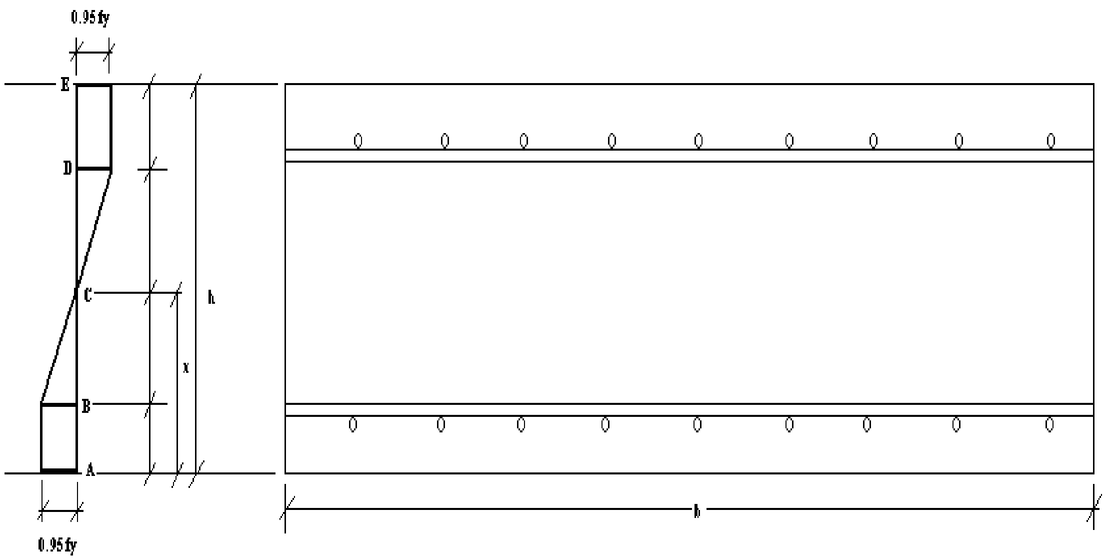

Fig. 3. Stress diagram of wall section.

IV. Construction of Wall-Slab Joint

The sub-assemblage of RC wall-slab joint comprises of foundation shear wall and floor slab were constructed on strong floor. Initially, the foundation beam's cage was prepared in the lab prior to the construction of formwork as shown in Fig. 4.

Fig. 4. Preparation of foundation beam cage.

Fig. 5 shows the skeleton wall-slab joint using fabric wires mesh (BRC-7) were cut according to the size of the floor slab and wall panel. Reinforcement bars with 12mm diameter were used to tie wall and slab skeleton at cross-bracing joints with 200mm spacing between each other.

Fig. 5. Skeleton of wall-slab consist BRC-7.

Fig. 6. Pouring of ready-mix concrete into the mould.

Fig. 6 shows the pouring of ready-mix concrete into the steel mould through top opening of wall panel. The wet concrete was poured up to the level of slab steel mould and let the wet concrete to cure and hardened for a few days.

Fig. 7 shows the finish product of wall-slab joint is seating on foundation beam. This sub-assemblage of wall-slab joint was painted with color before any experimental work took place. This specimen is ready for instrumentation and experimental set-up before start testing.

Fig. 7. The sample is ready for testing

V. Instrumentation and Experimental Set-Up

Fig. 1 shows the systematic arrangement of linear potentiometers and location of double actuator. Load cell with capacity of 250kN is connected to double actuator and supported by the reaction frame. A total number of ten (10) LVDT were installed to record the deflection of sample. Strain gauges were installed to record the strain of bars due to alternate tension and compression stress. Strain gauges can detect the elongation of reinforcement bar starting from yielding, elasto-plastic, plastic and ultimate strain. The strain gauges at reinforcement bar were installed prior to casting of sample. The exact and detail arrangement of strain gauges attached to the reinforcement bars (BRC-A7) are shown in Fig. 8.

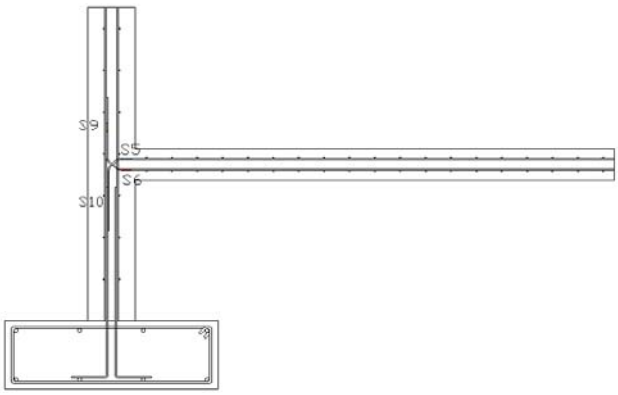

Fig. 8. Location of strain-gauge on the BRC-A7

Double actuator imposed the lateral cyclic loading on to the wall using control displacement. While the head of load cell is connected to steel plate and clamped to the wall by screwed up snug tight the treaded bars. The RC wall became sandwiched by steel plate clamping to the double actuator head so that the wall can be pushed and pulled laterally during the experiment work without any gap between the steel plates. At the of end floor slab, two steel plates are attached to wall using high yield threaded rods. The slab is supported by UB steel section to ensure that the support is fixed and the wall is free to rotate in-plane. The foundation beam is clamped to strong floor by penetrating the high yield threaded bar through the holes located in foundation beam.

VI. Testing Procedure and Loading Regime

In order to evaluate the seismic performance of the wall-slab connection under out-of-plane cyclic loading, a proper arrangement of loading regime and testing procedure needs to be adopted in this experimental work. The initial calibration of the instruments need to be carried out before imposed any lateral load to the structures. After all the instruments were tested and functional well, then the real test can take place. Each drift will be tested for two cycles in order to get better graph for the hysteresis loops. Fig. 9 shows the loading regime which used for the tests in terms of lateral displacement and number of cycles. The specimen were loaded with a hydraulic actuator having 250 KN capacities through a load cell with incremental of lateral displacements. The load is applied in full cycles which involves push and pull activities. At each incremental of displacement, the maximum load was maintained constant for a few seconds in order to measure and record the load, displacement response of the walls and the steel strain via electronic data logger.

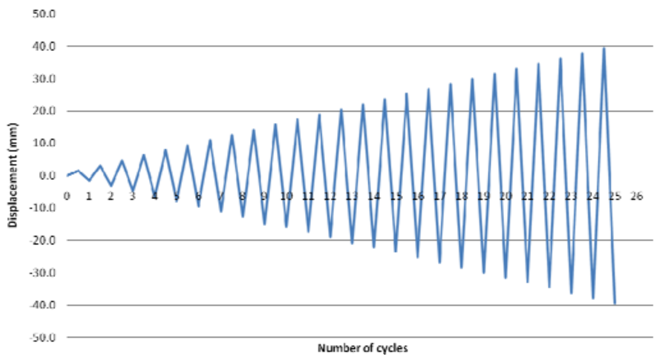

Fig. 9. Loading regime for the testing procedure.

VII. Visual Observations and Experimental Results

Table I shows the classification of Damage States of wall-slab joint according to percentage drift and visual observation during experimental work. The wall-slab joint behaves elastically up to 0.9% drift before yielding and categorized it's under Damage State 2. In the elastic range, the strain in concrete and reinforcement were deformed in the same rate.

TABLE I: Classification of Damage States

Damage State	Drift interval (%)	Visual Observations
1(Operational)	***0.1 - 0.5***	• Cracks started • Elastic behavior
2(Moderate)	***0.6 - 1.0***	• Yielding of Wall-slab joint at 0.9% drift.
3(Major)	***1.1 - 1.6***	• Reinforcement yielded at 1.3% drift. • Ultimate state at 1.6% drift • Inelastic behavior
4(Near collapse)	***1.7 - 2.2***	• Loose-fitting of wall- slab connection at 2.0% drift • Sudden drop of stiffness at 2.1% drift which indicated fracturing of wall-slab connection • Inelastic behavior
5(Collapse)	***2.3 - 2.5***	• No more cracks propagated • Enlargement of cracks • Fracturing of reinforcement • Inelastic behavior

At 0.9% drift, the wall-slab joint behaved inelastic up to 1.0% drift. The reinforcement bar of BRC-A7 was yielded at 1.3% drift with ultimate load reached at 1.6% drift. This phenomenon can be categorized under Damage State 3.

Damage State 4 occurred between 1.7% to 2.2% drift where wall-slab joint loose-fitting between them. It was followed by the sudden drop of applied lateral cyclic load at 2.1% drift. As soon as the tensile stress in the concrete exceeding the modulus of rupture (tensile strength), the cracking took place and the concrete immediately experienced cracking and spalling of the concrete. The minimum amount of longitudinal reinforcement bars in the concrete is unable to carry the additional loads which come from lateral loading. Fractured of longitudinal reinforcement bars consequently enlarged the concrete cracks and caused immediate collapse of structure can be classified as Damage State 5.

Table II shows the comparison between the experimental results and theoretical values of ultimate moment and ductility of wall-slab joint. The theoretical values were obtained based on the equation 1 to 8 as described above. The experimental value for ultimate moment has similar value with theoretical value with percentage difference of 0.01%. The corresponding ratio of ultimate moment between experimental and theoretical values is 0.99. Therefore, there are a good agreement between the experimental value and theoretical value. The ductility ratio between the experimental value and theoretical valus is 0.89 which shows that the theoretical value is slightly higher than the experimental. Basically, the ductility of wall-slab connection can be considered as low since it was failed under brittle failure mode.

TABLE II: COMPARISON BETWEEN EXPERIMENTAL AND THEORETICAL RESULTS OF MOMENT AND DUCTILITY.

	Experiment Value (1)	Theoretical Value (2)	Ratio values (1) / (2)
Moment resistance (kNm)	37.79	37.97	0.99
Ductility	2.22	2.5	0.89

Fig. 10 shows the cracks occurred at wall-slab joint. Spalling of concrete occurred at top part of the joint when the lateral load is applied on top of the wall. Applied lateral cyclic loading on the sample was induced the alternate tension and compression stress at the joints.

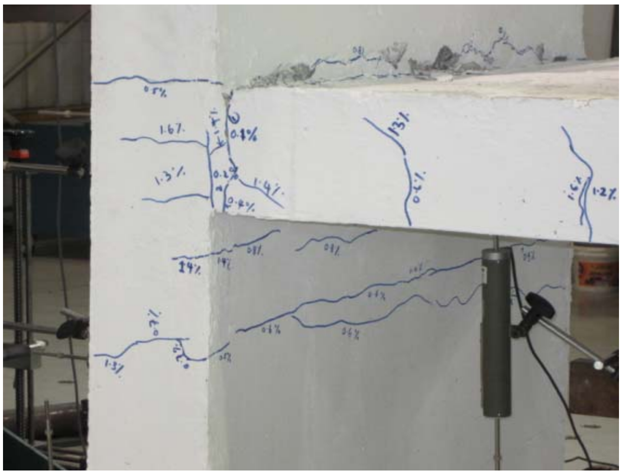

Fig.10: Cracks occurred wall-slab joint from side view.

Fig. 11 shows the crack on the wall from rear view during 1.6% drift. It can be observed that the higher stress was induced many cracks at wall which closed to the vicinity of wall-slab joint. Stress in the upper part of wall-slab joint is greater than the bottom part. Therefore, a lot of cracks occurred at upper part of the wall and the cracks started to propagate from slab to both sides of the wall.

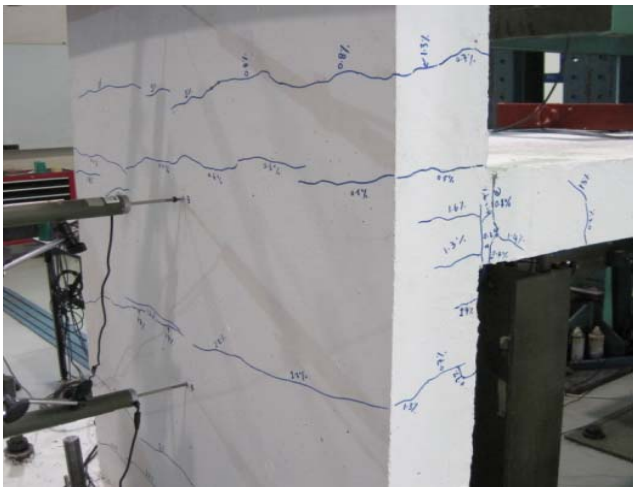

Fig. 11. Cracks at wall-slab joint from rear view.

Fig. 12 shows concrete was broken into two pieces and fractured of reinforcement bar at wall-slab joint due to the total loss of strength to resist the lateral cyclic load. The upper of the wall collapsed and categorized it under Damage State 5.

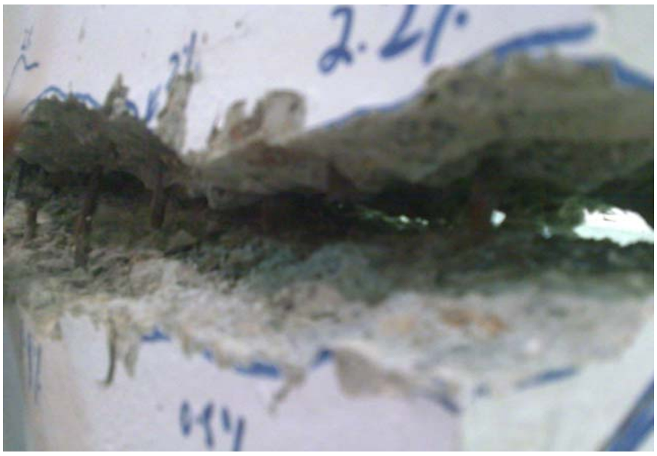

Fig. 12. Fractured of BRC-A7 and broken of joint.

VIII. ANALYSIS OF RESULTS

Fig. 13 shows the load versus displacement for the specimen starting from 0.1% to 2.5% drift. It can be seen that there are two profile lines which differ in color. The blue line profiles represent a push load-displacement characteristic whilst the red line profile is corresponding to pull load-displacement characteristic. It can be observed that the load-displacement shows a proportional linear characteristic up to 0.9% drift while in pull-load displacement up to 0.6% drift. The wall-slab joint yielded at 0.9% drift stage and afterwards it was behave inelastic manner. The ultimate load is 50.38kN at 1.6% drift under pushing load and 40.62kN under pulling load. At 1.7% drift, there is a reduction of pushing loading due to strength degradation. Then, it was followed by a sudden drop of load at 2.1% drift.

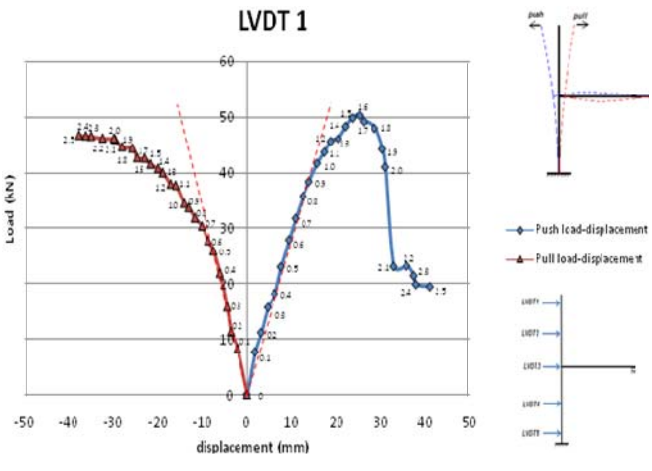

Fig. 13. Load-displacement profile of wall (LVDT 1)

Fig. 14 shows the location of strain gauge in the wall-slab joint corresponding to their load-strain profile. The lateral cyclic load imposed on the sample was caused alternate compression and tension stress in the reinforcement. Nonetheless, the tension strain was induced the greater effect in the reinforcement rather than the compression strain. Thus, the analysis on the load-strain behavior presented in Fig. 14 was concentrated based on tension strain developed in the reinforcement bar.

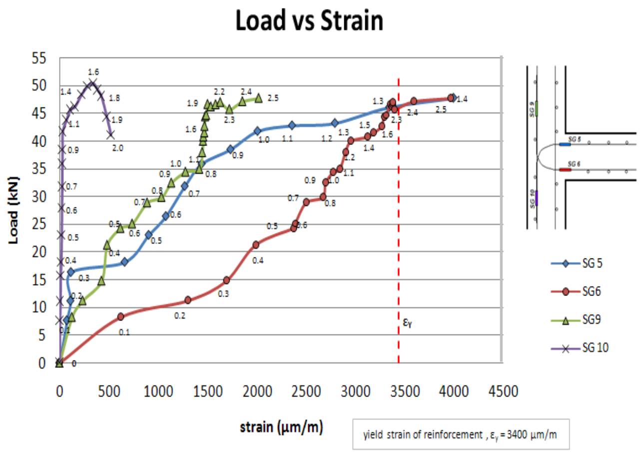

Fig. 14. Load versus strain for reinforcement.

The hysteresis loops of wall-slab joint have plotted by using the data obtained from 0.3% drift until 2.5% drift as shown in Fig. 15. Fig. 15 shows the hysteresis loop of wall-slab joint which based on data obtained in LVDT 1. By observing the individual hysteresis loop at every drift percentage, it can be discovered that the individual loop shows the small enclosed pattern of loop. This indicate the small energy dissipation in the system which not effective to maintain longer under lateral cyclic loading. Consequently, the brittle failure happened in the wall-slab connection.

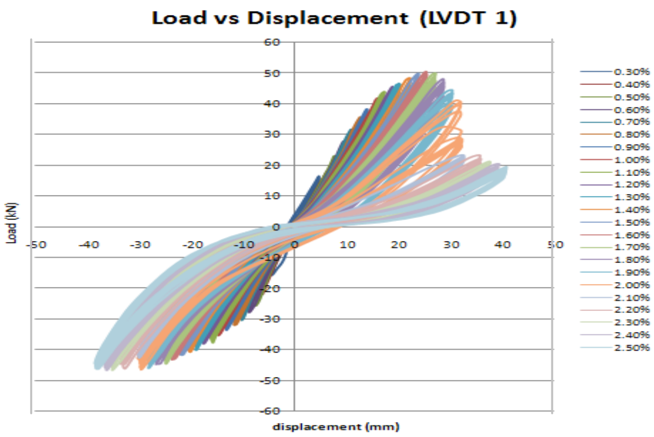

Fig. 15. Hysteresis loops of wall based on LVDT 1

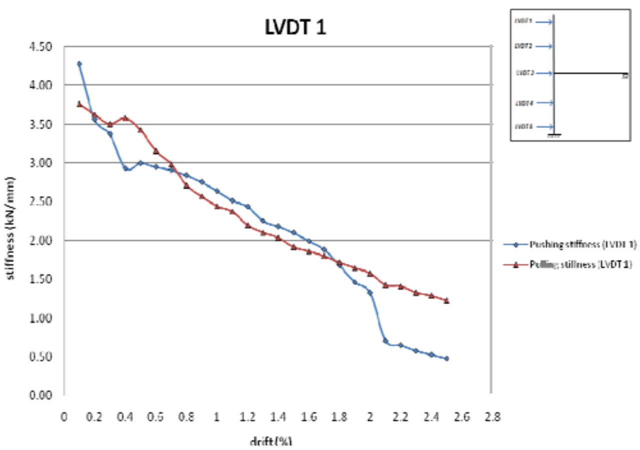

Fig. 16. Stiffness profile of wall based on LVDT 1.

Fig. 16 shows the stiffness profile of the wall-slab joint for LVDT 1. There are two line profiles in red and blue color which represent pulling stiffness and pushing stiffness. At 0.1% drift pushing stiffness of the wall is greater than its pulling stiffness. The position was return back as had occurred at 0.1 % drift previously within 0.8% to 1.8% drift intensity. By focusing on pushing stiffness of the wall, it can be observed that the sudden drop in stiffness was take place at 2.1% drift. Basically, the stiffness of wall in both load direction are showing degradation in stiffness with respect to an ascending in drift intensity. It was discovered that LVDT 2 was showed the similar pattern of stiffness as discussed in LVDT 1.

IX. Conclusion

The biggest hysteresis loops were occurred at LVDT1 which located the closest to the double actuator. The theoretical moment capacity is slightly higher than experiment moment capacity. However, there is a good relationship between them. The stiffness of wall-slab joint started to decrease from 0.2% drift until 2.1% drift and lost it stiffness after 2.1% drift. The theoretical ductility is higher than the experimental ductility. Nevertheless, these ductility of the wall-slab joint is still below the requirement for seismic ductility which normally between 3 to 6. Therefore, this type of structure needs to increase the percentage of reinforcement bars in the concrete and proper detailing at joint is required for seismic loading. Many cracks were observed in the vicinity of the wall-slab joint. Most of the cracks developed at the rear wall, bottom of slab and wall-slab joint surface. It was discovered that the wall-slab joint was governed by brittle modes failure. The minimum amount of vertical and horizontal steel at the wall-slab joint was unable to carry the additional load. Therefore, spalling and cracking of concrete cover were observed, longitudinal reinforcements yielded and fractured suddenly without any warning.

Acknowledgement

The authors would like to thank the Research Management Institute, University Teknologi MARA, Malaysia and the Ministry of Science, Technology and Innovation (MOSTI), Malaysia for the funding this research work. Nevertheless, the authors also want to express their gratitude to the technicians of Heavy Structures Laboratory, Faculty of Civil Engineering, UiTM for conducting this research work successfully.

References

[1] Balkaya, C, Kalkan, E., "Nonlinear seismic response evaluation of tunnel form building structures. Computational Structural Journal, 81:153-165, 2002 .

[2] Yuksel, S.B and Kalkan, E. "Behavior of tunnel form buildings under quasi-static cyclic lateral loading," *Structural Engineering and Mechanics*, Vol. 27, No. 1,pp 99-115, 2007.

[3] Paulay, T and Priestey, M.J.N., "Seismic design of reinforcement concrete and masonry buildings. *J. Wiley & Sons*, New York, 1992.

[4] Gioncu, V. and Mazzolani, F.M., "Ductility of seismic Resistant Steel Structures," *Spon Press*, New York, 2002.
[5] Agarwal, P. and Shrikhande, M. "Earthquake Resistant Design of Structures, " *Prentice-Hall India"*, New Delhi, 2007.
[6] Agarwal, P. and Shrikhande, M. "Earthquake Resistant Design of Structures, " *Prentice-Hall India*, New Delhi, 2007.
[7] Booth, E and Key, D." Earthquake designs practice for buildings," *Thomas Telford Ltd*, London,Edition 2, 2006.
[8] Garcia, L.E and Sozen, M.A. "Earthquake Resistant Design of reinforced Concrete Buildings. In Earthquake engineering from engineering seismology to Performance-based Engineering Book," edited by Yousef Bozorgnia & Vitelmo V. Bertero, *CRC Press*, New York, 2004.
[9] Jacobsen, L.S. (1930). "Steady Forced Vibrations as Influenced by Damping." ASME Transactione 1930, 52(1); 169-181.
[10] Saiful, A.B.M, Jameel, M. and Jumaat, M.Z. "Seismic Isolation in Buildings to be a practical reality: Behavior of Structure and Installation Technique," *Journal of Engineering and Technology Research*, Vol. 3(4), pp. 99-117, April 2011.

N.H. Abdul Hamid was born in Selangor, Malaysia in 1966. She obtained her BSc in Civil Engineering from U.S.A and MSc. in Structural and Management from United Kingdom. She completed her PhD (Earthquake Engineering) from University of Canterbury, Christchurch, New Zealand in 2007. She has been working as a lecturer at University Teknologi Mara, Shah Alam, Selangor for 19 years. Her research interests are seismic performance of precast buildings under earthquake excitation, Damage Avoidance Design, Direct Displacement Based Design, design of rocking precast hollow core walls under earthquakes, wall-slab connection, precast beam-column joints, fragility curves and Incremental Dynamic Analysis.

M.A. Masrom is currently working as a lecturer at University Teknologi MARA, Shah Alam, Selangor. He has completed his Master (Structural Engineering) from University Teknologi Mara, Shah Alam, Selangor in 2010. His research mainly focuses on the seismic performance of wall-slab joint using tunnel form system of construction.