

Seismic performance of wall-slab joints in Industrialized Building System under out-of-plane reversible cyclic loading

Mohd Ashaari bin Masrom and Nor Hayati binti Abdul Hamid

Faculty of Civil Engineering, Universiti Teknologi MARA, 40450 Shah Alam, Selangor, Malaysia

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Abstract

Joints between floor slabs and shear walls constitute an essential link in the lateral load-resisting mechanism of slab-wall of reinforced concrete building. The seismic performance of wall-slab joint influences the pattern and distribution of lateral forces among the vertical elements of a structure. This study presents the results of experimental work on the seismic performance of wall-slab joints which designed in accordance to BS 8110. The experiment work includes full-scale test of wall-slab connection under out-of-plane reversible cyclic loading. This study focuses on hysteresis loops, ductility, strength, stiffness, damage pattern and modes of failure for wall-slab joints. The results indicate that the wall-slab joints were governed by brittle failure modes in reinforced concrete and fracture of reinforcement bars. This is due to low ductility in concrete structures and the system cannot absorb energy efficiently under nonlinear deformation. In fact, the brittle failure modes did not allow sufficient energy dissipation and lead to sudden failure without warning to the structures as experienced by the RC buildings during earthquakes.

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Keywords: damage pattern, ductility, strength, stiffness, wall-slab connection, out-of-plane cyclic loading

1. Introduction

Nowadays, the construction industry in Malaysia is shifting from conventional method system towards Industrialized Building System (IBS). IBS is the construction system whereby most of the structural elements such as beams, columns, walls, slabs and others structural elements in RC buildings are prepared in the factories/plants and assemble at construction site within short period of time. The structural components of building are transported to the site for erection of RC building using cranes and heavy machinery. Tunnel form construction is one type of Industrialized Building System which widely used in construction of precast reinforced concrete buildings. This system is implemented in the construction of houses and condominiums in seismic and non-seismic regions due to their industrialized modular

construction technique and faster construction. This technique is using cast-in situ concrete where in-situ concrete is poured into two half-tunnel forms to shape load bearing walls (shear-walls) and floor slabs simultaneously. This process is repeated within 24-hour cycle per floor, the residential units can be rapidly built up within short period of time. It makes tunnel form construction system become more attractive proposition for the erection ranging from medium to high-rise buildings having the repetitive elements or layouts of the units. In RC buildings, the entire vertical load which carrying members are made from shear-walls and floor system is flat plate. Both gravity and lateral loads (seismic or wind) are transferred to shear-walls before transmitted to the foundation. In Malaysia, tunnel form building has been used in construction since 1980. This method of construction extensively applied in the construction of high-rise residential house (multistory building) such as condominium, apartment, commercial buildings and hotel.

One of the major issues arise in designing of high-rise RC building is concerning on the lateral resistance of building to resist the lateral force which is commonly comes from wind loading and earthquake or seismic loading. However, nowadays wind loading is not the major problem which did not cause the collapse of building. Many codes of practice were developed to accommodate wind load factor for determining structural integrity and stability of RC buildings. Meanwhile, seismic load is the loading that always impair the building structure and cause the holocaust such as collapse and topple of building. There are a lot of earthquake events in Indonesia which cause tremor to the people who live at high-rise buildings in Malaysia. For instance, the current earthquake with magnitude of 7.9 scales Richter recorded on September 2009 in South Sumatera, Indonesia causes the tremor to the few areas in west coast of Peninsular Malaysia. It was reported that many Malaysian especially those who stay in high-rise building felt the swaying of building, after the earthquakes struck in Indonesia. It was discovered through an inspection that 30 percent out of 65 buildings in entire country inclusion of Kuala Lumpur, Putrajaya, and Klang are vulnerable to earthquake risk. In fact, less than 1 percent of building in Malaysia are comply with the specification of seismic loading effect.

Yuksel and Kalkan (2007) have been undertaken an experiment work on the tunnel form building under quasi-static cyclic lateral loadings. From their experiment work, it was discovered that the interface connection of the wall and slab suffered damages after the testing. In the RC building structure, the crucial zone for determining the stability of the building is the connection/joint whether at beam-column, wall-foundation, wall-slab or slab-beam joint (Paulay and Priestley, 1992). The joints should have enough strength and stiffness to resist the induced stresses and sufficient stiffness to control undue deformations. Large deformations of joints result in significant increase in the storey displacement. Basically, the design requirements of seismic region can be summarized in three requirements which is strength, ductility and stiffness requirements (or toughness) (Gioncu and Mazzolani, 2002). These three requirements should be fulfilled in the design in order to govern the response of reinforced concrete structure subjected to strong ground motion caused by an earthquake. Ductility in the structure will arise from inelastic material behavior and detailing of reinforcement in such a manner that brittle failure is avoided and ductile behavior is induced by allowing steel to yield in controlled manner (Agarwal and Shrikhande, 2007). On the basis of stiffness, the structure may be classified as brittle or ductile (Agarwal and Shrikhande, 2007). Brittle structure having greater stiffness proves to be less durable during earthquake while ductile structure performs well in earthquake. The brittle members need to be strong enough to withstand the forces induced 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 (Booth and Key, 2006). Predominantly, buildings in Malaysia were designed without considering the seismic loading in their building. 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 (Garcia and Sozen, 2004).

Most of civil engineers assume that Malaysia is not undergoing a major or severe earthquake event as compared to Indonesia which located in Pacific Ring of Fire. However, they cannot overlook this matter since Kuala Lumpur is just located 450 km apart from Sunda plate which is one of the most active plates in the world with velocity of 70mm/year. The current code of practice for shear wall and slab in Malaysia are based on British Standard (BS8110) which does not have any provision for seismic loading. 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 true picture of what will happen on RC buildings during earthquake. Hence, this research is focused on the seismic performance of wall-slab connection in IBS (industrialized building system) subjected to reversible lateral cyclic loading. Hysteresis loops of load versus displacement, mode of failure, type of damage and stress-strain relationship will be measured in this experiment.

2. Design of wall-slab connection

The specimen comprises of foundation beam, wall and slab element as shown in Figure 1. The dimension of foundation is 1800mm length and 900mm width. Meanwhile, the dimension of height, width and thickness of wall is 1500mm, 1000mm, and 150mm respectively. The slab width and thickness is identical to wall dimension while the length of span is 2000mm. The reinforcement bars for the foundation are 16mm and 12mm diameter for longitudinal and transverse reinforcement, respectively. The fabric wires mesh (BRC-7) are arranged 200mm vertically and 100mm horizontally used as wall and slab. The lapping bar from foundation has designed to give the fixed connection to the wall-slab element. The cross bracing detailing of wall-slab connection has adopted in the design as shown in Figure 1.

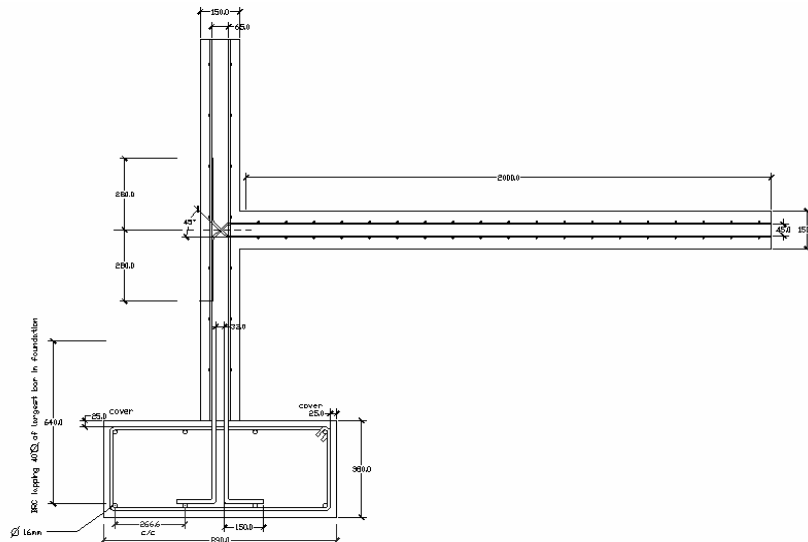


Fig. 1. The arrangement of reinforcement bars in sample.

3. Theoretical background of wall-slab connection

The theoretical background of wall-slab connection is similar to the approach used for beam-column connection which also known as compatibility strain approach by using the stress-strain relationship of concrete and reinforcement bars. Figure 2 shows the cross-section of wall together with reinforcement bars and curvature of the strain at yield 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 and ultimate displacement and load of the structures. From Figure 2, the dimensions of wall width = b , effective depth = d , depth of the neutral axis = $k d$ (in elastic range) and = x_u (in the limit state of collapse), yield strain of tensile steel = ϵ_y , yield curvature = ϕ_y , yield strength of reinforcement = f_y , concrete compressive strength = f_{ck} , area of tension steel = A_{st} , p = ratio of steel area to concrete area.

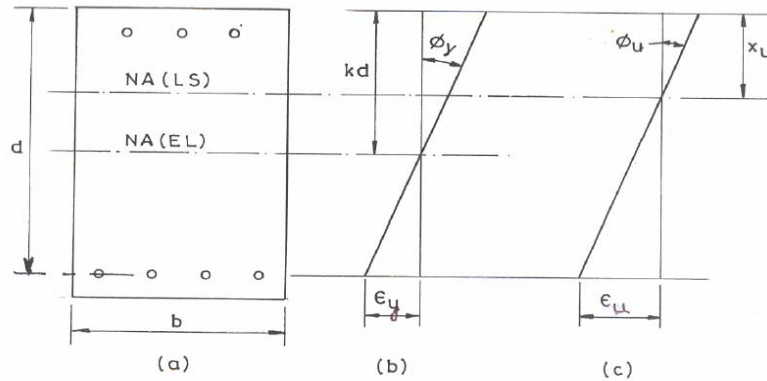


Fig.2. (a) Cross section of wall, (b) Strain at yield state, (c) Strain at ultimate state

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

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

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

where $\epsilon_y = f_y / E_s$ (3)

$$k = -mp + \sqrt{m^2 p^2 + 2mp} \tag{4}$$

$$m = \text{modular ratio} = \frac{280}{3\sigma_{bc}}$$

σ_{bc} = Permissible stress of concrete in bending compression

Similarly,

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

where ϵ_u = ultimate strain of concrete = 0.0035

$$\frac{\epsilon_y}{d} = \frac{0.87 f_y A_{st}}{0.86 f_{ck} b d} = \frac{0.87 f_y p}{0.86 f_{ck}} \approx \left(\frac{\epsilon_{u,max}}{d} \right) \tag{6}$$

Substituting Eqs. (2) and (5) in Eq. (1), we have ductility in terms of strain, effective depth of wall and neutral axis of the wall

$$M = \frac{F_y}{\epsilon_y} \left(\frac{d - k'd}{\lambda_B} \right) \tag{7}$$

$$M = \frac{\epsilon_u}{F_y/E_s} \left(\frac{k \lambda_B m p - \sqrt{(k \lambda_B m p)^2 + 2 m p}}{\lambda_B \epsilon_y / d} \right) \tag{8}$$

The experiment result was validated by comparing with calculated moment resistance according to the analytical model equation (Bhatt et al., 2006). The analytical model equation was derived based on stress diagram as shown in Figure 3.

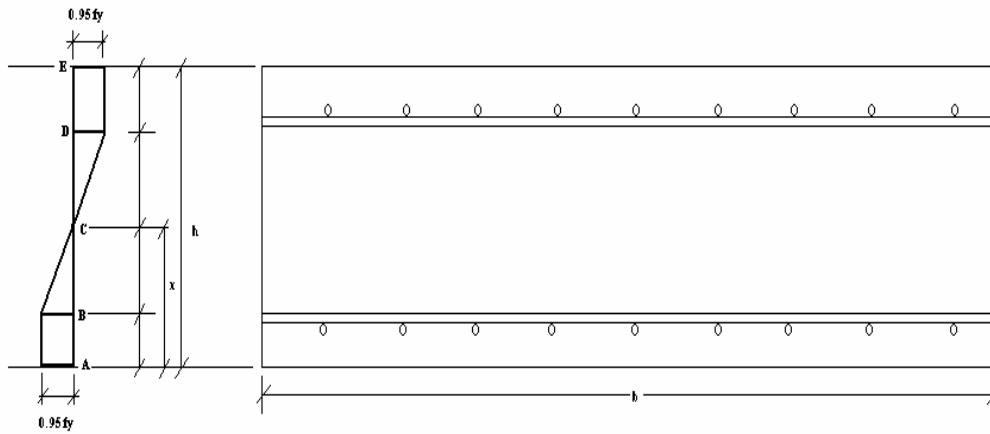


Fig. 3. Stress diagram of wall section

By considering the equilibrium of forces in composite action in the Figure 3, it has given the Moment resistance of the section as:

$$M = \{0.45 f_{cu} b \times 0.9 \times (0.5h - 0.45x) + [AB(0.5h - 0.5AB) + 0.5BC(0.5h - AB - BC/3) - 0.5CD(0.5h - AB - BC - 2/3CD) + DE(0.5h - AB - BC - CD - 0.5DE)] 10^{-3}\} 10^6 \tag{9}$$

where $f_y = 678.01 \text{N/mm}^2$ (from tensile test), Young's Modulus, $E_s = 200 \times 10^3$, then the strain ϵ_y when the stress is $0.95f_y$ is given by

$$\epsilon_y = 0.95 f_y / (200 \times 10^3) = 0.95(678.01) / (200 \times 10^3) = 3.2205 \times 10^{-3}$$

If the maximum compressive strain in concrete is 0.0035 and the neutral axis depth is x , the strain in steel is to ϵ_y at depth c from the neutral axis, where

$$c = (\epsilon_y / 0.0035) x = (3.2205 \times 10^{-3} / 0.0035) = 0.9201x$$

$$x - c = x - 0.9201x = 0.0799x$$

Hence,

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

4. Construction of wall-slab specimen connection

The subassembly of wall-slab connection comprises of foundation, shear wall and slab were constructed in Heavy Structural Laboratory, Faculty of Civil Engineering, Universiti Teknologi Mara, Shah Alam, Selangor. Initially, the foundation beam cage was prepared in the lab before construction of the formwork. Eight holes with 30mm diameter were made to cater for clamping foundation to strong floor. By providing the foundation beam, the wall-slab structure can be fixed to strong floor when imposed the lateral cyclic load. A formwork made from plywood was assembled in order to have a rectangular reinforced concrete foundation beam. Eight holes are drilled at the base of foundation formwork in order to place the steel pipes as shown in Figure 4. The high yield threaded bar will be placed throughout the pipes and fasten them using washers and nuts to fix the foundation beam to the strong floor. Reinforcement bars with caging is prepared for foundation beam. The longitudinal bars with 16 mm diameter were adopted whilst 12 mm diameter have been used for the transverse reinforcement. Figure 4 shows the arrangement of the reinforcement in the foundation formwork. Figure 5 shows the fabric wire mesh (BRC-7) which had been cut according to the size of the slab and wall. Reinforcement bars of 12mm diameter are used to connect wall and slab with the pattern of cross-bracing at the joints with 200mm spacing between each other.



Fig. 4. Preparation of foundation beam



Fig. 5. BRC detailing of wall-slab

Figure 8 shows the steel mould which had been used as a formwork to obtain the intended shape of the specimen. The height of the steel mould to form wall is 1.6m whilst the span length is 2.0m to give a shape of flat slab. The breadth of steel mould is 1.0m. The steel mould had been set-up on the foundation beam before casting of concrete was executed as shown in Figure 8. Then, BRC-7 was placed into the mould with some spacer blocks have been attached at certain places allowing for concrete cover for the specimen. The sides of mould for slab and both front and back mould faces for the wall were set-up and screwed into their position as shown in Figure 8. Then, the concrete mix has pouring into the mould as depicted in Figure 6. The concrete was smoothing to get an even surface as shown in Figure 7. The steel mould was taking off after the concrete hardened and achieves the compressive strength of 30MPa. Figure 9 shows the finish product of wall-slab connection seating on foundation beam. This specimen is ready for instrumentation and experimental set-up before commerce testing.



Fig. 6. Pouring concrete into mould



Fig. 7. Smoothing the concrete surface



Fig. 8. Set-up of mould on foundation beam



Fig. 9. Configuration of the sample

5. Instrumentation and experimental set-up

Figure 10 shows the systematic arrangement of linear potentiometers and double actuator. The load cell with capacity of 250kN is connected to double actuator and supported by the reaction frame. Double actuator will imposed the lateral cyclic loading on the wall with control displacement. While the head of load cell is connected to steel plate and clamped to the wall by screwed up the treaded bars snug tight. 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. A total number of ten (10) LVDT were installed on the sample in order to record the deflection consequential from the lateral cyclic load applied on the sample. Five (5) units of LVDT were place along the height of wall while three units along the span of the slab as shown in Figure 10. One LVDT is placed vertically at the column and horizontally at the end of slab to detect any movement s in-plane and out-of-plane movement of the slab. Another two LVDT are located vertically on the foundation beam to detect any movement in both directions. Strain gauges were installed to read the change in

strain of reinforcement due to alternate tension and compression stress during the experiment work. The strain gauges at reinforcement bar were installed prior to casting of sample. The exact and detail arrangement of strain gauges on the reinforcement can be seen in Figure 11.

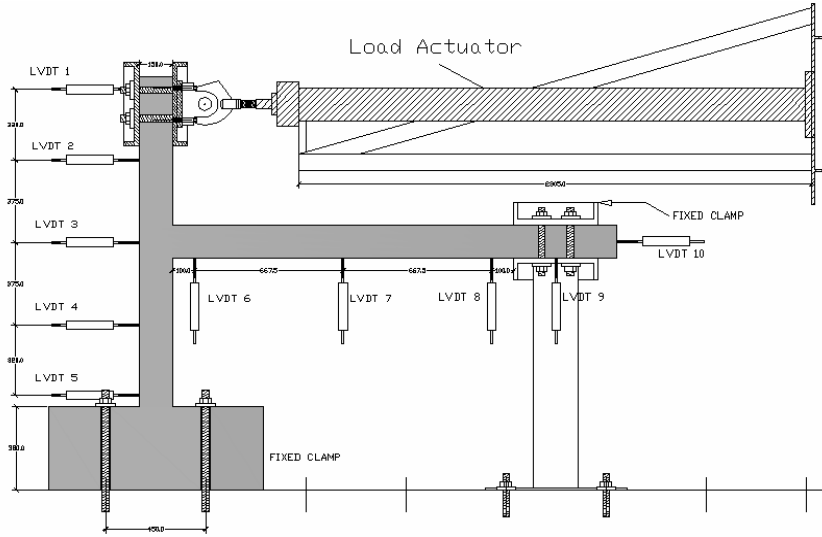


Fig. 10. Experimental set-up

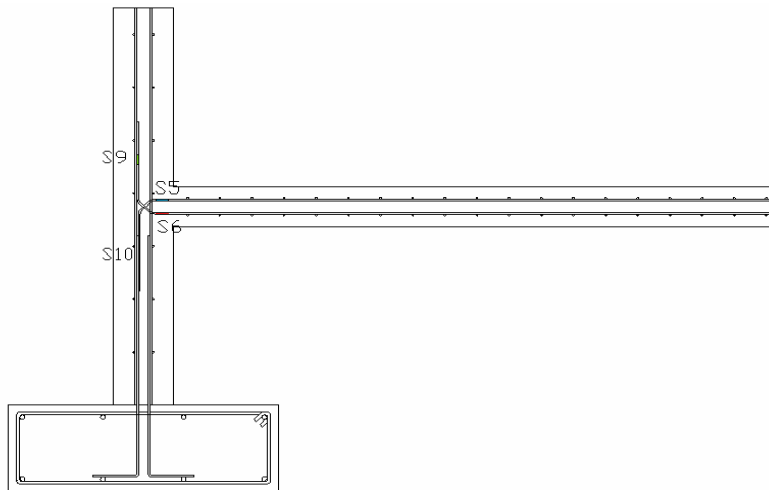


Fig. 11. Location of strain gauges at the wall-slab connection

6. 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. The initial push and pull should be started with very small drift which located within the elastic regions. 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. During testing, the data and

visual observation must be taken for three major states which namely cracking state, yielding state and ultimate state. In all three cases, the horizontal load was applied at the centroid of the distributed lateral forces. Figure 12 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.

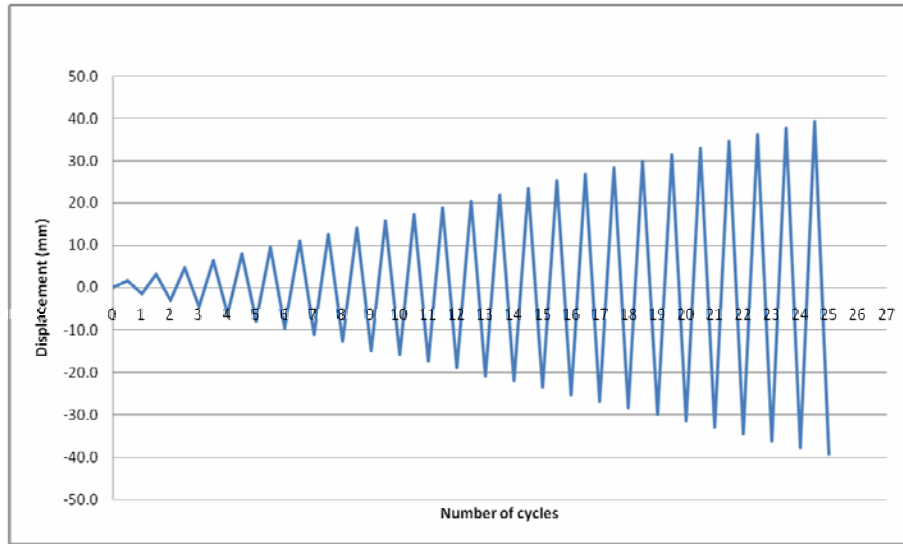


Fig. 12. Testing schedule for wall-slab connection

7. Experimental results and visual observations

Table 1 shows the summary of the experiment result together with their damage state. This table represents the interval drift percentage corresponding to important event occurred during the testing. The wall-slab connection behaves elastically up to 0.9% drift which categorized under Damage State 2. In the elastic range, the strain in concrete and reinforcement were deformed in the same amount of load due to fully bonded strength of concrete-reinforcement bar. Beyond 0.9% drift, the wall-slab connection behaved in inelastic manner in the next 1.0% drift level. The reinforcement bar of BRC-A7 was yielded at 1.3% drift and the ultimate load was reached at 1.6% drift. Both yielding and ultimate state of reinforcement bars have occurred in Damage State 3. In the range of Damage State 4 (1.7% to 2.2% drift), when the lateral drift reached 2.0%, the loose-fitting of wall-slab connection was occurred. It was followed by the sudden drop of applied lateral cyclic load at 2.1% drift. The sudden drop of load indicates that the failure of wall-slab connection was controlled by *brittle* modes in the concrete wall. The crushing of concrete was not observed and the damage was concentrated on the shear-walls only. This failure mechanism occurred due to low longitudinal reinforcement ratio of walls and slab at the connection part only. Consequently, the wall-slab connection cannot absorb a lot of energy and undergo the inelastic deformation under out-of-plane loading. 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 load which come from lateral loading, therefore the

cracking of concrete, longitudinal reinforcements yielded and ruptured suddenly without warning were observed during experimental work. The rupture of reinforcement bars consequently enlarge the concrete cracks have caused the immediate collapse of structure which can be classified as Damage State 5.

Table 1
Classification of Damage States in accordance with drift and visual observations

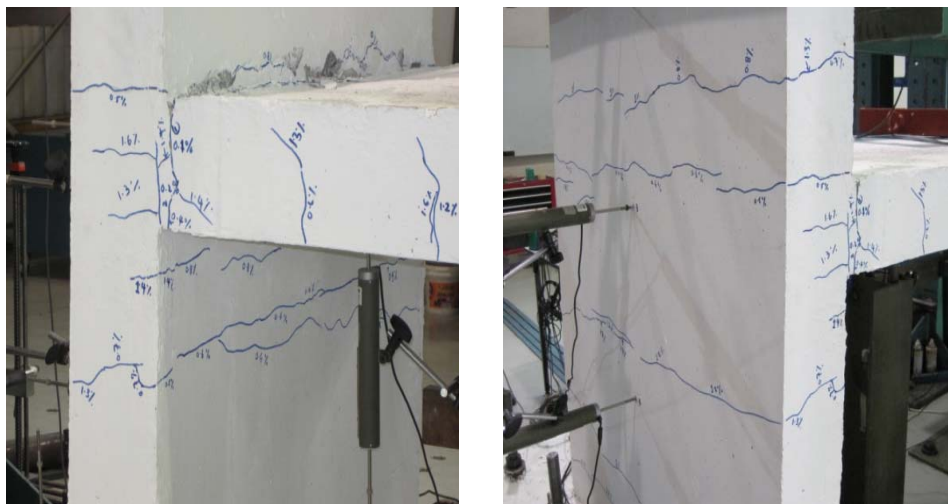
Damage State	Drift interval (%)	Visual Observations
1 (Operational)	0.1 - 0.5	<ul style="list-style-type: none"> • Cracks started • Elastic behavior
2 (Moderate)	0.6 - 1.0	<ul style="list-style-type: none"> • Yielding of Wall-slab connection (At 0.9%).
3 (Major)	1.1 - 1.6	<ul style="list-style-type: none"> • Reinforcement yielded (At 1.3%). • Ultimate state (At 1.6%) • Inelastic behavior
4 (Near collapse)	1.7 - 2.2	<ul style="list-style-type: none"> • Loose-fitting of wall-slab connection (At 2.0%) • Sudden drop of stiffness at 2.1% (indicated fracturing of wall-slab connection) • Inelastic behavior
5 (Collapse)	2.3 - 2.5	<ul style="list-style-type: none"> • No more cracks propagated • Enlargement of cracks • Fracturing of reinforcement • Inelastic behavior

Table 2 shows the comparison between the experimental results and theoretical values of ultimate moment and ductility of wall-slab connection. The theoretical values were obtained based on the equation 1-8 as discuss 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.

Figure 13 shows the visual observation of the cracks which mainly concentrated at the joint between the wall-slab connections. Figure 13(a) shows the cracks observed on wall and slab from side view. There is spalling of concrete occurred at top part of the joint when the lateral load is applied on top of the wall. The applied lateral cyclic loading on the sample was induced the alternate tension and compression stress on the joints. The major cracks have developed whenever the wall and slab surface subjected to tension stress. It can be observed that the higher stress was induced many cracks at the vicinity of wall-slab connection as shown in Figure 13(b). The stress in the upper of wall-slab connection is greater than the bottom part.

Table 2
Validation of ultimate moment and ductility results

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



(a) Side view (b) Rear view
Fig. 13. Visual observation of cracks on surface of specimen

Figure 14 shows the broken in concrete and fracturing of reinforcement in wall-slab connection due to the total loss of strength to resist the lateral cyclic load. The damage pattern of the wall-slab connection at the end of experiment can be observed in Figure 15.

8.0 ANALYSIS OF EXPERIMENTAL RESULTS

Figure 16 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.

Figure 16 represents the lateral displacement of LVDT 1 which located at the top of the wall. Based on the push load-displacement profile, 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. Therefore, the wall-slab connection behaves elastically up to 0.9% drift. The wall-slab connection yielded at 0.9% drift stage and afterwards it was behave inelastic manner. At the yield point, the concrete-reinforcement bar strains commence deviate each other and become significant in the inelastic state. The ultimate load (50.38kN) was reached at 1.6% drift intensity 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.

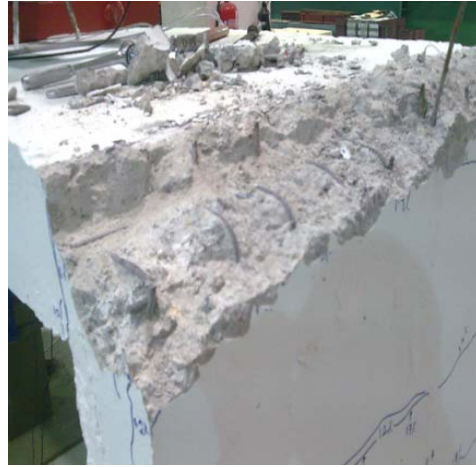
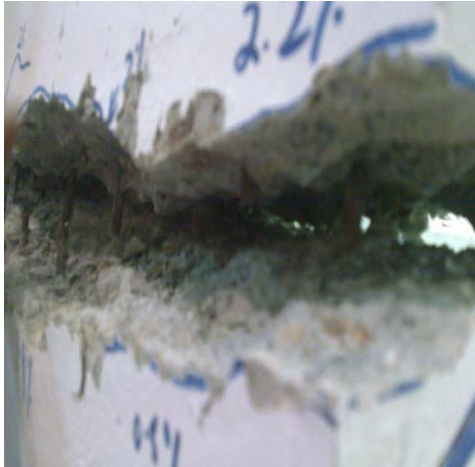


Fig. 14. Fracturing of BRC and Broken in concrete Fig. 15. Total damage of wall-slab connection

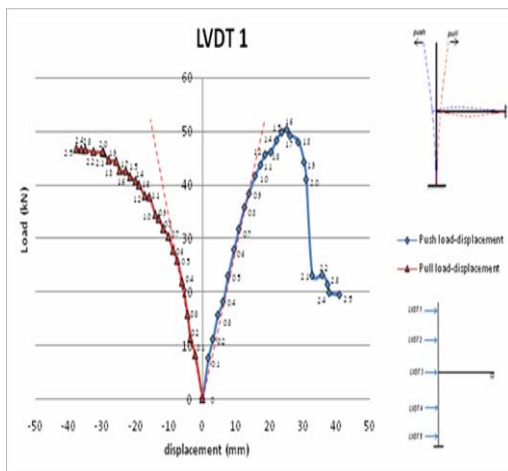


Fig. 16. Load-displacement profile of wall (LVDT 1)

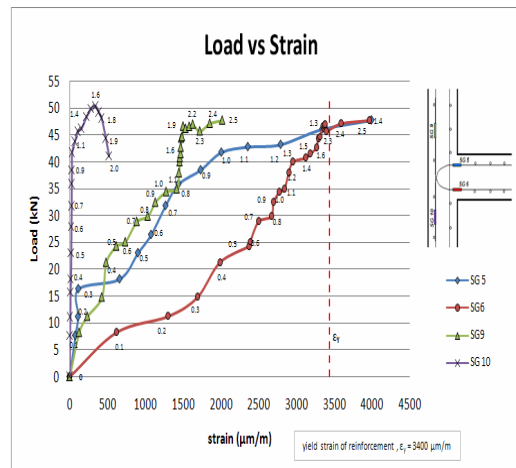


Fig. 17. Load-strain profile of reinforcement

Figure 17 shows the location of strain gauge in the wall-slab connection 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 Figure 17 was concentrated based on tension strain developed in the reinforcement. Figure 13 shows that SG 5 reached a yielding strain at 1.3% drift whilst SG 6 was touched a yielding strain at 2.3% drift. This was signified that the tension stress developed at upper wall-slab interface region is greater than the tension stress developed at the bottom wall-slab interface region. At SG 9 and SG 10, they did not reach the yielding point until at the end of experiment. This was indicated that the tension stress

developed apart from the wall-slab connection is turn out to be alleviate. It was discovered that the numerous crack lines were appeared (especially at wall-slab connection and rear wall (parallel to wall-slab intersection)) after the reinforcement had yielded at 1.3% drift.

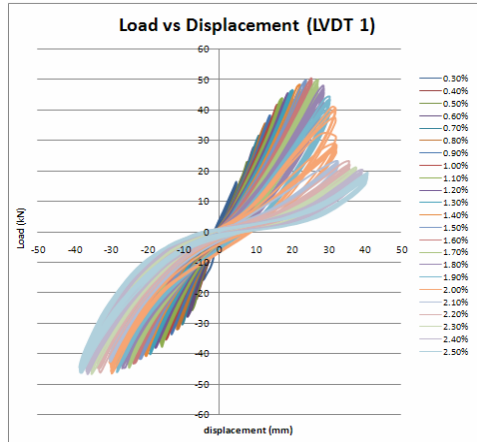


Fig. 18. Hysteresis loops of wall based on LVDT 1

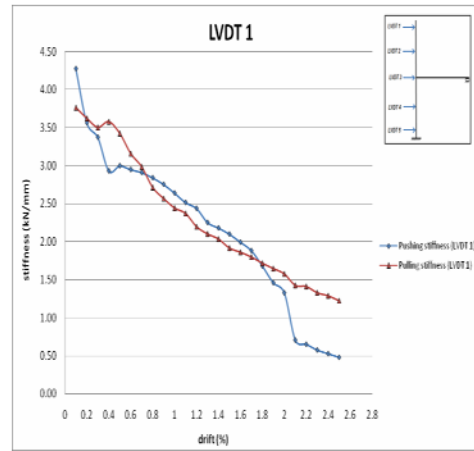


Fig. 19. Stiffness profile of wall based on LVDT 1

The hysteresis loops of wall-slab connection have plotted by using the data obtained from 0.3% drift until 2.5% drift as shown in Figure 18. Figure 18 shows the hysteresis loop of wall-slab connection 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. Figure 19 shows the stiffness profile of the wall-slab connection for LVDT 1. There are two line profiles in red and blue color which represent pulling stiffness and pushing stiffness of the wall respectively. At 0.1% drift pushing stiffness of the wall is greater than its pulling stiffness. Yet, the pulling stiffness of the wall has on the top of the pushing stiffness at 0.2% drift intensity until they gone up to 0.7% drift intensity. The position was return back as had occurred at 0.1 % drift previously within 0.8% to 1.8% drift intensity. By focusing only 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.

9. Conclusion

Connections between floor slabs and shear walls constitute a potential weak link in the structures resisting lateral forces because of the critical stress combinations that develop in those regions during lateral sway. Many cracks have propagated in the vicinity of the wall-slab connection. Most of the cracks developed on the rear wall, bottom of slab and wall-slab connection surface. The ductility of wall-slab connection is low since the enclosed boundary of the hysteresis loops is narrow as shown in Figure 18. It was found that the wall-slab connection was governed by brittle modes failure. This is due to low ductility which causes the connection could not absorb more energy and further undergo inelastic deformation. The minimum amount of vertical and horizontal steel at the wall-slab connection was unable to carry the additional load, therefore following the cracking of concrete, longitudinal reinforcements yielded and ruptured suddenly without warning. The wall stiffness degrades

gradually up to 2.0% drift. The abrupt change of stiffness in the wall due to sudden drop in lateral load was occurred at 2.1% drift. The moment and shear resistance of the wall-slab connection are inadequate to resist the high magnitude of lateral load.

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