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SEISMIC PERFORMANCE OF INTERIOR WALL-SLAB JOINT IN TUNNEL-FORM BUILDING DESIGNED TO BRITISH STANDARD

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A B S T R A C T

In Malaysia, tunnel form building has been utilized in building construction since 1960. This method of construction was applied extensively in the construction of high-rise residential house (multistory building) such as condominium and apartment. Most of the tunnel form buildings have designed according to British Standard (BS) whereby there is no provision for seismic loading. The high-rise tunnel form building is vulnerable to seismic loading and the joint between slab and shear walls in tunnel form building constitute an essential link in the out-of-plane lateral load-resisting mechanism. As known, Malaysia is no longer safe from earthquake disaster consequent to the damage of building. In line with that, this study is intended to investigate the performance of interior wall-slab joint in tunnel form structure designed to British Standard. The experiment work includes full-scale test of wall-slab joint sub-assemblages under out-of-plane lateral cyclic loading. One sub-assemblage specimen of interior wall-slab joint was designed and constructed according to the code. The specimen was tested based on lateral displacement control (drift control). It was found that the specimen is able to survive up to 1.5% drift with significant loss of stiffness in the early of drift level. The analysis results indicate that the specimen was governed by brittle failure modes with ductility class low (DCL) as stated in Eurocode (EC) 8. This was resulted from insufficient reinforcement provided. Consequently, the specimen was unable to dissipate sufficient energy to sustain longer in inelastic zone.

1. Introduction

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 and earthquake loading. However, nowadays wind loading is not the major problem which does not cause the collapse of building. Many codes of practice have established wind load effect in designing reinforced structure (RC) structural. Meanwhile, seismic load is the loading that always impair the building structure, cause the holocaust such as collapse, and topple of building. There are many 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.Recently, earthquake was strucked in Ranau, Sabah in 2015. It was reported some infrastructure damages around 25 schools in six different districts affected and Ranau Mosque was damaged due to the tremor. Serious damages occurred

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to the hostels and rest house near the summit of the Mount Kinabalu. 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 resistance. Due to rapid demand on the residential apartment in Malaysia, most of the high rise buildings in Malaysia are constructed using tunnel form buildings. Most of the buildings in Malaysia were designed according to BS8110 (1997) where there is no provisions for earthquake loading at all. Therefore, it is expected that most of these buildings will suffer moderate or severe damages if the magnitude of earthquake more than 6.5 scale Richter strike Malaysia. In relation to that, the numbers of researchers in Malaysia that have undertaken an investigation on the seismic performance of structures using existing designed code is getting increased as compared previously. For instance,Yee et. al (2011) were reviewed on performance of IBS precast concrete beam-column connections under earthquake effects in Malaysia. This was followed by Masrom et. al (2012) that was studied about the seismic performance of exterior wall-slab joint. He was found that the joint was governed by brittles modes. Further, Ghani et. al (2013) were undertaken an investigation of non-seismic precast RC beam-column exterior joint under lateral cyclic loading. It was discovered that a very limited research study have been carried out about the interior wall-slab joint. It has been started by Pantazoupouloul et. al (1992). They were carried out the laboratory works on the wall-slab connection subjected to in-plane lateral cyclic loading. Kudzys (1996) was carried out a finite element study to evaluate the wall-slab connection behaviour under extreme lateral actions. Further, Kaushik et. al (2016) was carried out a nonlinear time history analyses, under different levels of recorded earthquake ground motion using the computer program ABAQUS to study the seismic damage in shear wall – slab junction of an RC wall-frame building. To date, it was found that no specified study has ever undertaken to investigate the seismic performance of physical interior wall-slab joint designed to BS subjected to out-of plane cyclic loading. The joint between slab and shear walls in tunnel form building constitute an essential link in the out-of-plane lateral load-resisting mechanism. Therefore, this study is intended to investigate the survivality of interior wall-slab joint designed to British Standard under seismic loading.

2. Problem Statement

Most of structural engineers assume that Malaysia will 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 BS8110 (1997) which does not have any provision for seismic loading. Therefore, the buildings structure especially at the joint part are susceptible to damage and risk of collapse if bigger earthquake happened in the neighbouring countries or in Malaysia. Due to that situation, the aim of this research is to investigate the seismic performance of interior wall-slab joint in tunnel-form structure designed according to BS8110 (1997) and tested under seismic loading.

3. Objectives

1. To observe the visual damage pattern of interior wall-slab joint subjected to out-of-plane lateral cyclic load.
2. To study the strength degradation of interior wall-slab joint in response to lateral cyclic loading.
3. To investigate the adequacy of ductility capacity of interior wall-slab joint over seismic design code (EC8) requirement.

4. Methodolgy

4.1 Construction of Interior Wall-slab Subassemblage

The sub-assemblage of interior wall-slab connection comprises of foundation, shear wall and slab panels as shown in Figure 1.0.The height of the wall panel is limited to 1500mm while the length of slab panel is 1500 mm measured from the wall surface. All the panels' thickness is 150mm. The length, width and thickness of foundation beam are 1800mm, 965mm, 325mm respectively. This specimen has designed according to BS8110 (1997).

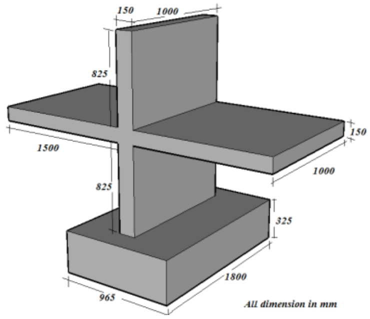


Figure 1: Schematic diagram of Interior Wall-slab joint design to British Standard

Figure 2 shows the reinforcement detail at interior wall-slab joint. Fabric wire mesh (BRC-7) with 200mm by 100mm grid has utilized. The arrangement of the wire mesh in specimen can be seen in this figure. Hogging bars of 12mm diameter have placed 100mm apart in transverse direction to cater negative moment in the slab panels.

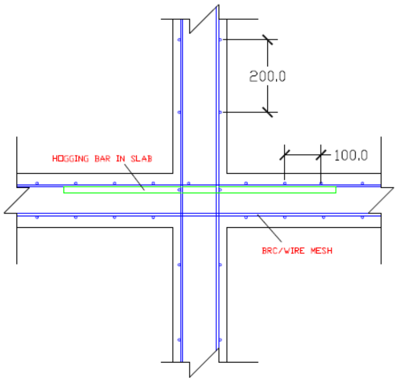


Figure 2: Reinforcement detail of Interior Wall-slab joint (dimension in mm).

Figure 3.0 and 4.0 show the photo of joint reinforcement and the completed specimen respectively.

Figure 3: Reinforcement detail of Interior Wall-slab joint

Figure 4: Configuration of the specimen under testing

4.2 Instrumentation and experimental set-up

Figure 5 shows the systematic arrangement of linear potentiometers and double actuator. The load cell with capacity of 250kN has connected to double actuator and supported by the reaction frame. Double actuator has 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.

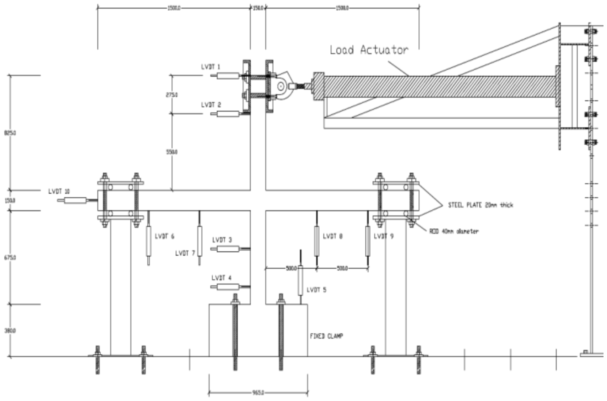


Figure 5: Schematic diagram of Experimental set-up of specimen

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. 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 has clamped to strong floor by penetrating the high yield threaded bar through the holes located in foundation beam. A total number of 10 LVDT have installed on the specimen in order to record the deflection consequential from the lateral cyclic load applied on the sample. Five units of LVDT have installed horizontally along the height of wall while another five units along the slab span and foundation as depicted in Figure 5..

4.3 Testing procedure and loading regime

Figure 6 shows the loading regime procedure imposed on the specimen during the testing. The specimen has loaded with a hydraulic actuator having 250 KN capacities through a load cell with lateral displacements control. The push and pull load has applied in full two cycles at each drift level. 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.

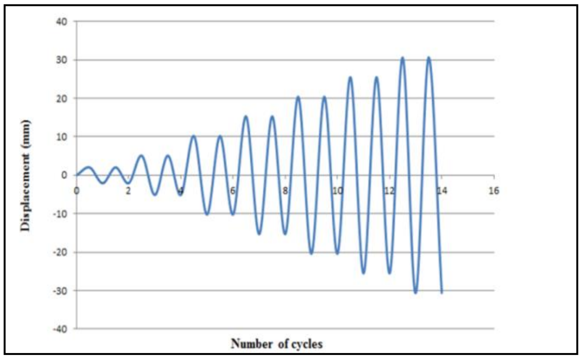


Figure 6: Testing schedule for BS wall-slab joint

5. Experimental Results and Visual Observations

5.1 Cracks propagation on specimen

All the cracks have undergone opening and closing during the experiment due to alternate compression and tension stress generated by the lateral cyclic loading. These cracks have opened by tension stress while closed by compression stress. Basically, the specimen can be divided into five regions, which are wall-slab joint, upper wall panel, lower wall panel, right and left slab panel. The cracks have visible in the early of 0.1% drift level at wall-slab joint, upper wall and slab panel. These cracks have continuing propagated up to 0.5% in those regions which can be observed in Figure 8(a), (b) and 9(a). It was found that the speciemen has started to lose it fitting against the upper wall panel within 0.75% to 1.0%. This behaviour can be observed graphically in Figure 7.

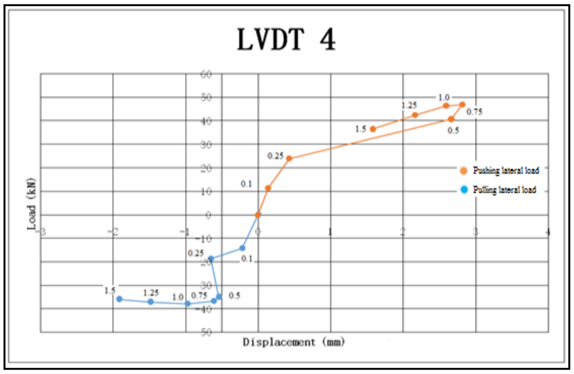
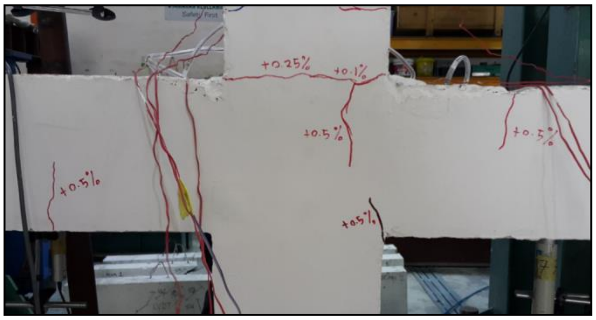
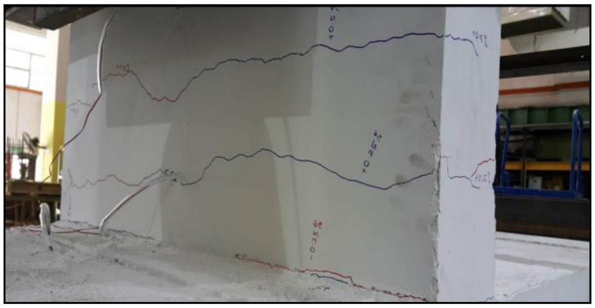


Figure 7: Lower wall load-displacement response of specimen at LVDT 4

According to Figure 7, the displacement of the lower wall has begun to drop at 1.0% drift. This indicates that the joint between upper walls against specimen has starting to break. Approximately, at 0.75% drift, the concrete wall has broken and the enlargement of cracks has continuing penetrate towards wall reinforcement. The slab panels to the upper wall panel are connected effectively up to 0.75% drift and lose their connection when reached 1.0% drift. No more new cracks have propagated on wall and slab panels beyond 1.0% indicating that the surface stresses have reduced in entire specimen. The significant enlargement of cracks has observed in between upper wall to wall-slab joint at 1.25% drift as depicted in Figure 10(a) and (b).

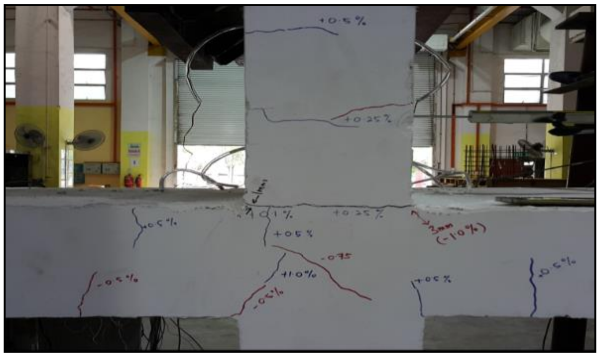


(a) Wall-slab joint and slab panel

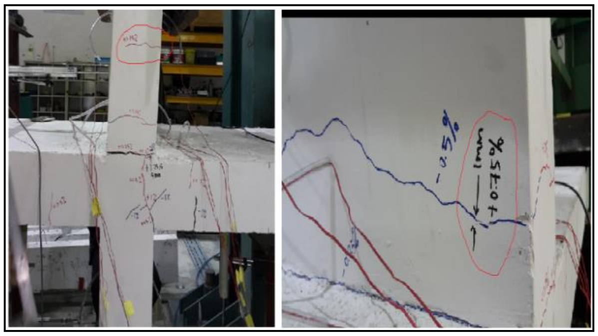


(b) Upper wall panel

Figure 8: Visual observation of cracks propagated on specimen within 0.1% to 0.5% drift level



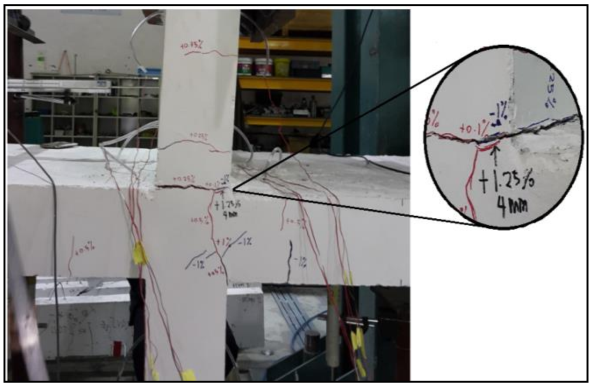
(a) Wall-slab joint



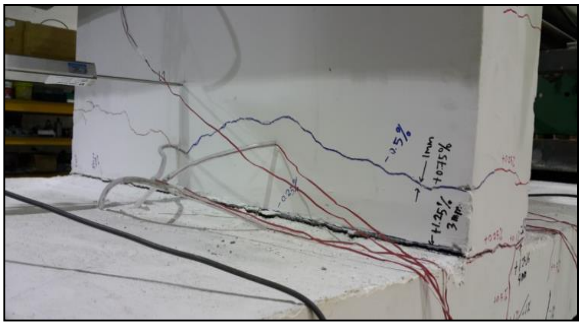
(b) Upper wall panel

(c) Slab panel surface

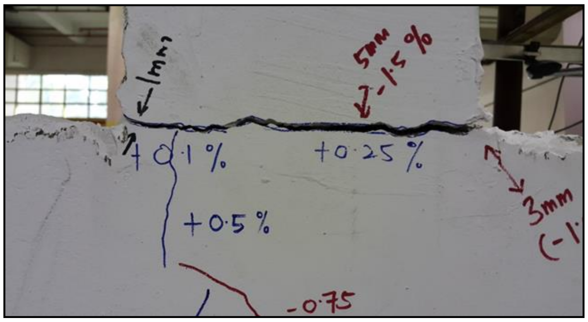
Figure 9: Visual observation of cracks propagated on specimen within 0.75% to 1.0% drift level



(a) Wall-slab joint



(b) Wall-slab joint viewed from side



(c) Fracturing of upper wall panel

Figure 10: Visual observation of cracks propagated on specimen within 1.25% to 1.5% drift level

5.2 Hysteresis Loop and Ductility Capacity

Figure 11 illustrates the hysteresis loop of the specimen at LVDT 1 that has superimposed by the equivalent monotonic pushing/pulling load-displacement response. This has signified that the dynamic response of wall-slab joint is resembled to the static behaviour that commonly encounter in the reinforced concrete structures.

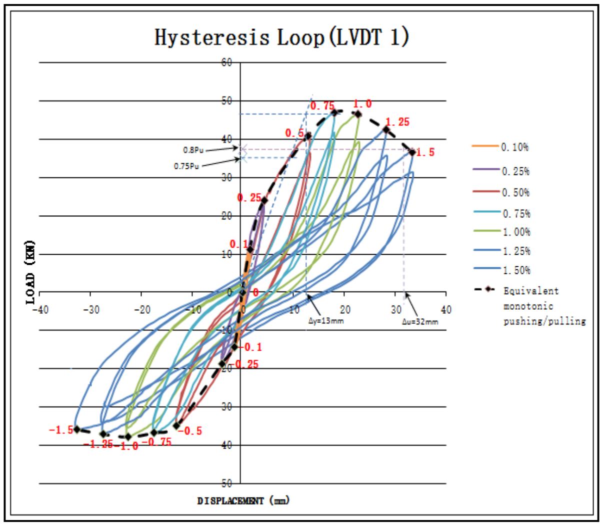


Figure 11: Hysteresis loop at LVDT 1.

The strength degradation of the joint can be observed clearly in this figure. The ultimate capacity of lateral resistance of the specimen is 46.9 kN at 0.75% drift. The joint behaved elastically up to 0.25% drift. Yielding has found to take place on the first cycle to ± 0.25% drift at about 24kN lateral force. The ductility factor is defined as the ultimate deformation divided by the corresponding deformation present at the yield point. In this study, the definition of displacement of yield is adopted from Park (1988) and the ultimate displacement can be estimated as that post-peak deformation when the load drop in strength (commonly taken as 20%) from the peak load or maximum attained strength (Priestley et al., 2007b). Consequently, the ductility, μ capacity of this unit was 32/13=2.5. The accepted range of ductility which had been specified in Eurocode 8 is 3 to 6 for Ductility Class Medium (DCM). The calculated displacement ductility for specimen was less than 3 and it is classified as Ductility Class Low (DCL) as stated in Eurocode 8 (EN1998). Consequently, the interior wall-slab joint designed according to BS 8110 will not survive under moderate to strong earthquake events.

6. Conclusions

Based on the analysis of results, discussion, and visual observation during the experiment, the following conclusions can be drawn:

1) Approximately, the joint has yielded at 0.25% drift and reached the ultimate state at 0.75% drift. Beyond 0.75% drift, the joint has failed and ultimately fractured at 1.5%.
2) The calculated displacement ductility for specimen was less than 3 and it is classified as Ductility Class Low (DCL) as stated in Eurocode 8.
3) Insufficient reinforcement provided in the interior wall-slab joint consequent low ductility that lead to brittle failure modes of the joint.
4) The BS8110 code is no longer realistic to be adopted in designing the tunnel form building based on the current seismic demand in Malaysia.

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