

Quasi-static test and strut-and-tie modeling of precast concrete shear walls with grouted lap-spliced connections



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HIGHLIGHTS

- Nine large-scale shear walls were tested including seven precast specimens and two cast-in-place specimens.
- PC shear walls used lap splice that one of the spliced bar was embedded into a grout-filled hole reserved by metal bellow or expanded metal mesh.
- A reasonable strut-and-tie model was proposed to investigate the force transfer mechanism of the specimens.

ARTICLE INFO

Article history:

Received 10 October 2016

Received in revised form 21 February 2017

Accepted 21 May 2017

Keywords:

Lap splice

Grouted connection

Metal bellow

Expanded metal mesh

Precast concrete shear wall

Bearing capacity

Strut-and-tie model

ABSTRACT

This paper discusses the seismic performance of nine full-scale shear walls including seven precast specimens and two cast-in-place specimens. The precast specimens were designed to emulate monolithic cast-in-place specimens. The precast upper shear wall and the base were connected at their interface through lap splice that one of the spliced bar was embedded into a grout-filled hole, which was reserved by metal bellow or expanded metal mesh. The geometries and reinforcement of precast specimens were identical with those of corresponding cast-in-place specimens in each group. But their stirrups were different. Based on the two types of grouted connections, the specimens were tested under low cyclic lateral loading to study their seismic performance by comparing their carrying capacities, crack patterns, ductility, hysteretic curves and energy dissipation. In general, shear walls with metal bellows at the joint exhibited better structural behaviour than the shear walls with expanded metal mesh due to their well confined effect on the spliced bars and surrounding concrete. Thus, this kind of precast connection may emulate cast-in-place connection. Lastly, a reasonable strut-and-tie model was developed based on ACI 318-14 code to investigate the force transfer mechanism of the precast shear wall. The calculated results were compared with the test results and the shear strength results calculated by empirical expression in the ACI 318-14 code.

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1. Introduction

Shear walls are known to be quite effective to resist wind and earthquake action due to their high lateral stiffness. Therefore, concrete shear walls are widely used for buildings as the first lateral load resisting system when new structures are designed. It is the fact that majority of researches were focused on the cast-in-place (CIP) concrete shear walls in recent ten years [1–8]. However, there are many disadvantages to construct by traditional cast-in-place construction, such as requiring much more labours and resources. As is known that almost all of main concrete structures (especially tall buildings) in mainland of China were designed and constructed by traditional cast-in-place pattern. Nowadays, precast concrete

(PC) structures [9–13] develop quickly due to the cost of labour increasing.

The PC elements are basically connected at their joints and installed at the construction site. The seismic performance of PC structures mainly depends on the behaviour of the connecting joints which are often poor compared with cast-in-place joints under the same condition. The joints of PC shear wall can be divided into dry- and wet-type joints. And at present the main joints used in PC walls in China generally include joint of grouted spliced sleeve, lap-spliced joint by restraint grouting-anchoring and lap-spliced joint by grouting metal bellow [14] (see Fig. 1). The grouted spliced sleeve is used widely in PC members in Asian, Europe and North America [15–19]. The lap-spliced joint by grouting metal bellow can also be used in PC walls due to its good economy and convenient construction. This joint is connected through lap splice in a grout-filled hole which is reserved by metal bellows

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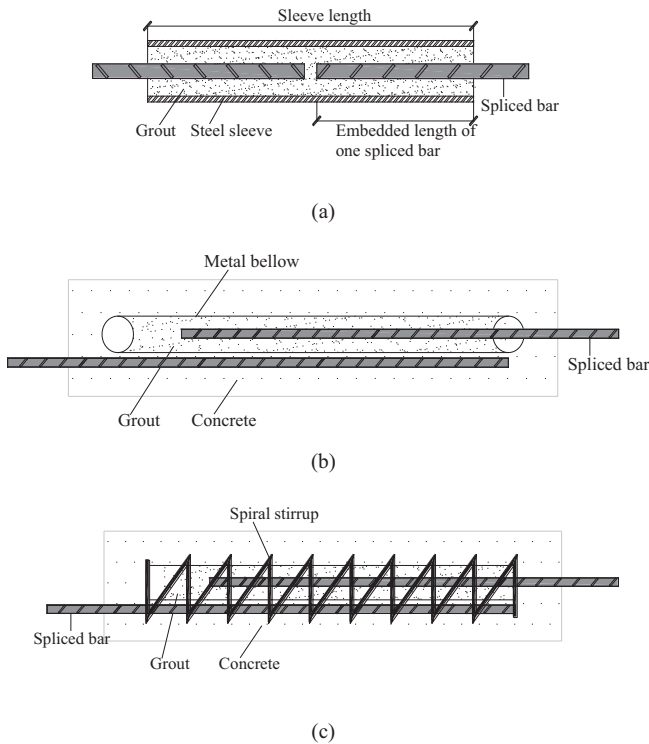


Fig. 1. The vertical joints used in PC walls. (a) Grouted-coupling sleeve; (b) Lap-spliced joint by grouting metal bellow used in this paper; (c) Lap-spliced joint by restraint grouting-anchoring.

during the construction. Thus, the confinement is provided by the metal bellows to increase the bond strength between the spliced bars and surrounding concrete. However, this joint test in PC walls was rare in the past experimental research.

To design a PC shear wall, its lateral load and displacement capacities are significant criteria for any joints considered in seismic regions especially for walls with moderate-aspect-ratios that are ratios between approximately 1.5 and 2.5 [20]. These shear walls may be exhibited significant nonlinear shear behaviour under lateral loading. Therefore, traditional sectional design approaches, which are based on the theory of plane section remaining plane and adopt a parallel chord truss model, may be not applicable for the design of low- and moderate-aspect-ratios members [21–23]. The strut-and-tie model (STM) is an effective design method for reinforced concrete members, such as deep beams, beam-column joints, corbels, and shear walls, in which regions are so-called disturbed or St. Venant regions (D-regions). The STM method is based on lower-bound plasticity theorem, thus this technique can provide conservative and safe designs [24–25]. In the past few years, most of the STM researches focused on the deep beams. Important differences exist with respect to internal stress flow between the shear walls and deep beams due to their different boundary conditions.

In this paper, the principal objective is to study the seismic performance of PC shear walls with lap splice in a grout-filled hole, which is reserved by metal bellows or by a pore-forming expanded metal mesh in the plastic hinge zone. The spliced bars in the expanded metal mesh are headed or U-shaped bars for anchorage capacity strengthening. Installation of spliced bar in the expanded metal mesh does not require high precision compared with that of metal bellows. The carrying capacities, crack patterns, ductility, hysteretic curves and energy dissipation of all specimens are studied based on the experiment. A reasonable strut-and-tie model according to the features of shear walls was developed based on

ACI 318-14 code [26] to investigate the force transfer mechanism of the precast shear wall. The calculated results were compared with the test results, the calculated flexural strengths and the shear strengths calculated by the empirical expression of the shear walls in the ACI 318-14 code.

2. Experimental program

2.1. Tested specimens

There are nine full-scale pieces of non-prestressed reinforced concrete shear walls including two conventional CIP benchmark shear walls and seven PC walls with different connection details and stirrups in the potential plastic zone. These specimens were divided into two groups: the first group consisted of specimens from SW1-1 to SW1-6; the second group consisted of specimens SW2-1, SW2-2 and SW2-3 (SW1-2 and SW1-3, SW1-5 and SW1-6, SW2-2 and SW2-3 had the same dimensions and reinforcement layouts, respectively). Each specimen in each group had identical dimensions and applied the same axial load. Specimens SW1-1 and SW2-1 were CIP specimens, while the others were PC specimens. Table 1 shows the test matrix of nine shear walls. The dimensions of the specimens were 3450 mm tall, 1600 mm wide and 200 mm thick in Group 1, while the specimens of Group 2 were 3650 mm tall, 1700 mm wide and 200 mm thick. The axial and lateral loads applied by a jack and an actuator, respectively, were transferred through a rectangular RC block at the top of each shear wall with the dimensions of 240 mm width and 250 mm height. Thus, the corresponding shear span ratios of Groups 1 and 2 were 2.07 and 2.08, respectively. The footing of the specimens were designed to remain elastic during the test, and were fixed to the strong ground with high strength fining twisted steel bars to prevent the specimens sliding horizontally and overturning under cycle lateral loading.

Fig. 2 shows the specimens' reinforcement layouts at the cross sections of potential plastic regions. Fig. 3 shows the side views of the reinforcement cages of PC specimens. Difference can be found at the connections and boundary elements. In group 1, the confinement of SW1-2 to SW1-6 in the boundary was rectangular spiral stirrups and was compound stirrups in group 2. The PC panel connections in SW1-2, SW1-3, SW2-2 and SW2-3 were lap splice in the grout-filled hole which was formed by metal bellows, and those in SW1-4, SW1-5 and SW1-6 were lap splice in a pore-forming expanded metal mesh. The difference between the latter specimens was that the shape of spliced bar in SW1-4 was headed bar (welded short bars) and those in W1-5 and SW1-6 were U-shaped bars for anchorage capacity strengthening. The lap splice lengths of PC specimens were all 600 mm.

2.2. Material properties

The main material properties of the tested shear walls are listed in Tables 2 and 3. The shear walls were cast and fabricated on two separated dates with the same nominal concrete compressive strength of 35 MPa. The actual strength of concrete was tested by cylinders with dimensions of 150 mm × 150 mm × 300 mm according to Chinese Code of Testing Methods for Normal Concrete Mechanical Property [27]. The average 28-day strength of grouting material was 75.1 MPa that was tested using cuboids with dimensions of 50 mm × 50 mm × 200 mm. The longitudinal reinforcing bars were HRB400 steel bars, and the stirrups were HRB235 steel bars. The horizontal and transverse distribution reinforcements were also HRB400 steel bars.

2.3. Fabrication process

The shear walls were fabricated in a prefabricated component factory. Some pictures related to the construction of the PC specimens are shown in Fig. 4. The upper PC shear wall panels and lower footing were cast separately, while the spliced bars in the footing were protruded with lengths of 620 mm from the surface. When fabricating, a layer of bedding mortar with thickness of 20 mm was laid onto the footing, and then the upper shear wall was placed onto the bedding mortar. The protruded spliced bars were inserted into the reserved holes of the upper wall. Finally, the metal bellows and pore-forming expanded metal meshes were grouted by a hand-held pump at a reserved grout inlet at the surface of shear walls according to the manufacturer specifications till the grout was flowed out from the inlet. After fabricating, all specimens were cured more than 30 days.

2.4. Test setup and loading sequence

The test setup is drawn in Fig. 5. The setup was constructed by taking into account the equipment including concrete reaction wall, hydraulic jacks, hydraulic actuator, screw stem, steel girder, steel strand, etc. Cyclic lateral loads were applied by a 1000 kN hydraulic servo control system that was mounted to a reaction wall. Vertical load was applied by two hollow core jacks placed on the top of the specimen with two prestressed steel strands. The applied axial loads were 600 kN in specimens SW1-1-6 and 750 kN in specimens SW2-1-3, which corresponded to

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