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Semi-rigid Behavior of Joints in Wood Light-Frame Structures

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Abstract

The problem of behavior of the joint in wood light-frame structures were discussed in the paper. Two types of joint flexibility and their impact on the behavior of the structure were discussed. The results of the experimental investigation of rotational and axial stiffness of joint in wood light-frame structures were presented. The micro-scale tests of specimen representing joist-to top plate and top plate-to-stud joints in wall diaphragms were conducted. The representing load-displacement relationships for rotation and bearing at the joints were calculated. These relationship are described by the non-linear function. The comparison of rotational angle and relative translation obtained from experiment with displacement calculated using theoretical formulas were provided.

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Keywords: wood light-frame structures; semi-rigid joints; axial and bending joint stiffness, experimental tests.

1. Introduction

Wooden structures, especially wood light-frame structures are characterized by low entire stiffness in comparison to the steel, masonry, or RC structures. The high stiffness of these structure results from the rigidity of the joints. In masonry or RC structures the high stiffness of the junction is provided by the peripheral ties while in steel structures by the welded or bolted connections.

In contrast to above examples in wood light-frame structures joints show semi-rigid behavior of work. In most cases the joint are made as single connection not continues junction. The connection is often provided by dowel type

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fasteners. Despite the high elasticity modulus of steel, the small cross-section area of the fastener does not provide high stiffness of the single connector.

Due to small cross-section dimension relative to its length, the fasteners characterized semi-rigid and non-linear behavior of the work. The non-linear behavior of the fastener affects on behavior of entire structural element (wall, floor, roof) [1]. The non-linear shape of load-displacement function depends on number of fasteners, their positioning, their stiffness as well the level of external loads [2,3].

An important issue is distribution of external loads into individual wall and floor elements in the building. Lack of masonry or RC stiffening walls causes that lateral stiffness of the building is provided by wood light-frame walls.

The floor elements are loaded out-of-plane by the dead and live loads but also work in-plane as horizontal stiffening diaphragms under horizontal loads. The floor elements are supported by the walls so both kind of loads applied to the floor are transferred onto walls [4]. The wind pressure acted on the exterior wall is distributed by the floor element onto interior walls parallel to the wind direction.

Application of horizontal load (i.e. wind pressure) undergoes deformation in both floor and wall elements. Deformations in floor element are not significant due to great number of sheathing-to-framing fasteners. Stiffness of this connection is greater than arise directly from the load-displacement relationship because of sheathing-to-joist bearing under the vertical load [5,6].

Another situation is found in case of wall diaphragms. These elements are loaded in two ways. However in opposition to the floor diaphragms both kind of external loads are applied in-plane of wall. Live and dead load from floor is applied as uniform load onto top edge of the wall diaphragm while horizontal loads in applied as lateral force at the top corner of wall.

Vertical loads cause the compressive state of work in elements of wooden frame while lateral force induces overturning moment. This moment causes dual state of work of wooden studs. The compression appears in some of the studs while the tension in the rest of the elements. In case of large dimension of exterior wall and low stiffness of interior wall diaphragms there is a danger of destruction of these walls. The experimental studies show that wall lateral stiffness is not proportional to the length of the diaphragm [7,8].

2. Stiffness of wall diaphragms

During the experimental researches conducted at Bialystok University of Technology three different types of specimens were tested. There were conducted full-scale experimental test of wall diaphragms. Four wall specimen 1,25m long by 2,75m high with one sheathing panel and three specimen 3,75m long by 2,75m high with three separate panels of sheathing and window opening in middle section was tested [9]. During another experiment [10] three fully sheathed specimen with the same dimension as diaphragms with opening also were tested.

Table 1. Experimentally indicated stiffness of wood light-frame wall diaphragms [9,10].

Specimen	Average stiffness of wall diaphragm K [N/mm]	
	Lateral loading	Simultaneous lateral and vertical loading
Wall with single sheathing panel	310	722
Wall with three sheathing panel and opening	453	1933
Wall with three sheathing panel	1185	2514

The wall specimen were tested in two configuration of external loads. In the first phase of loading the lateral force representing wind pressure was applied, while in second phase lateral loading was combined with vertical loads representing live and dead loads transferred from the floor diaphragms. Indication of wall diaphragm stiffness in both phases were conducted at loading level equals 40% of expected ultimate loads.

Vertical loads applied to the wall diaphragm increase its lateral stiffness. Lack of vertical loads (i.e. uninhabited buildings) entails small lateral stiffness of wall and excludes lateral loads distribution. White [11] recommended not include shear walls with length less than 4 feet (122cm) in overall stiffness and capacity under lateral loads.