Experiment on Precast Solid Concrete Floor Slabs resting on Cold-Formed Steel Framing Stud Walls

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**ABSTRACT**

In the United States, steel stud wall with hollow core plank floor system is widely used. This typical hollow core plank concrete floor slab system is supported by cold-formed steel (CFS) load bearing stud. In Thailand, however, the solid plank, instead of hollow core plank, is widely used in the reinforced concrete construction market and they are more economical in residential construction. The new combination of the precast solid concrete slab (concrete plank) with concrete topping (50mm thickness) resting on cold-formed steel framing stud wall is to be investigated. The main aim of this paper is to present the behavior of the new combination during the experimental tests. Especially, the newly invented connections between the solid plank and CFS stud wall are tested and investigated. Test specimen consists of two spans concrete slab with 3.0 meters length per span rest on three CFS framing walls with a height 2.0 meters. The overall dimension of the test specimen is 1.4x6.0x2.0 meter (WxLxH). A total of three specimens will be tested under compression load from the hydraulic jack through the load transferred steel frame until they are collapsed. The load capacities, failure modes and behavior of specimens and their connections during loading will be discussed.

**1. INTRODUCTION**

Cold-formed steel material is widely used in building construction because of its light weight, durability and fast installation as well as high strength and stiffness. In United States, Nabil (2006) reported that the cold-formed steel stud-plank system using hollow-core concrete slab resting on light steel frame stud wall is widely used. In Thailand, the use of cold-formed steel members in structural members was adopted and become more popular. However, it is still not widely used because of two reasons. The first reason is that the cost of construction material using structural steel is higher than reinforced concrete. The second reason is that the behavior and design of cold-
formed steel are more complicated than hot-rolled steel. When the floor of the building is made of cold-formed steel joist, the cost of construction is much higher compared to using the precast solid concrete slab (concrete plank) with concrete topping. However, the combination of the cold-formed steel wall frame with concrete plank as flooring members has never been tested.

The experimental wall frame and floor system is the combination of precast solid concrete slabs and cold-formed steel framing stud wall. The precast solid concrete slab is commonly used in residential and office building in Thailand such as apartment, school and dormitory. The purpose of this research is to study the behavior of the combined wall frame and floor system including the behavior of the connections.

2. TEST SPECIMENS

In this study, three test specimens have been tested and evaluated. All CFS framing are G550 grade which has the tensile yield strength of 550 MPa. The first specimen was conducted on two 3.0 m. spans of precast concrete slab resting on three CFS framing walls with 2.0 m. in height and 1.45 m. in width. Each wall frame consists of five cold-formed steel C-studs with 0.75 mm. thickness as shown in Fig. 1(a) and 350 mm. spacing, and the slab consists of four solid precast concrete slabs with five pre-stressed strands of 350 mm. in width and 50 millimeters in depth as shown in Fig. 2. The precast slabs were laid on top of the CFS wall directly. Without any fastener to connect between the precast slab and CFS wall. The schematic drawing of the test setup and the first specimen are shown in Fig. 3 and 4.

![Nominal Section of C-Stud G550 AZ150](a) C7575 (b) C7510
Fig. 2 Dimensions of concrete plank

Fig. 3 The first specimen schematic drawing
Fig. 4 The first specimen before loading

The second specimen was conducted on two 3.0 m. spans of precast concrete slab resting on three CFS framing walls with 2.0 m. in height and 1.45 m. in width. Each wall frame consists of four cold-formed steel C-studs with 1.0 mm. thickness as shown in Fig. 1(b), and 350 and 700 mm. spacing, and the floor consists of four solid precast concrete slabs, the same as the first specimen. For the second specimen, the precast slab was connected to the CFS walls by Hilti nail and through steel sheet as shown in Fig. 5. Fig. 6 and 7 show schematic drawing and the test setup of the second specimen.
Fig. 5 (a) Hilti nails (b) Shooting gun (c) Connection between walls and slabs

Fig. 6 The second specimen schematic drawing
The third specimen was the repeat test of the second specimen. Except, instead of using Hilti nail to connect between concrete plank and CFS wall, the concrete plank was resting on the CFS wall directly. However, the CFS angles were used as the shear key after the concrete is hardened as shown in Fig. 8. The Fig. 9 shows the third specimen.
2.1 Stud wall nominal axial strength

The nominal axial strength, according to the AISI Specification (2016), is the smallest of the values calculated in accordance with Sections E2 to E4 where applicable. The first value is the nominal axial strength, $P_{ne}$, for yielding and global buckling. The second value is the nominal axial strength, $P_{nl}$, for local buckling interacting with yielding and global buckling. The third value is the nominal axial strength, $P_{nd}$, for distortional buckling. AISI Specification (2016) allows the direct strength method to be used to perform an elastic buckling analysis for the entire cross-section by the finite strip method. Elastic buckling values ($P_{cre}$, $P_{crl}$, $P_{crd}$) are the loads at which the equilibrium of the member is neutral between two alternative states: the buckled shape and the original deformed shape. Elastic buckling analysis with the finite stripe method is performed using CUFSM program described by Li (2010). Table 1 shows the nominal axial strength of C7510 section is 1390 kg. The nominal axial strength of C7575 section is 914 kg.

Table 1 The critical elastic buckling loads and nominal axial strength

<table>
<thead>
<tr>
<th></th>
<th>C7575 (kg)</th>
<th>C7510 (kg)</th>
<th>C7575 (kg)</th>
<th>C7510 (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{cre}$</td>
<td>1142</td>
<td>1585</td>
<td>1002</td>
<td>1390</td>
</tr>
<tr>
<td>$P_{crl}$</td>
<td>1244</td>
<td>2945</td>
<td>914</td>
<td>1390</td>
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<tr>
<td>$P_{crd}$</td>
<td>2353</td>
<td>4343</td>
<td>3057</td>
<td>4767</td>
</tr>
</tbody>
</table>

2.2 Concrete plank nominal strength

Table 2 shows the load carrying capacity of the concrete plank from the manufacturer. The concrete plank used in the experiment has 5 strands of 4 mm. in
diameter steel reinforcement and can carry live load of 7.85 kPa (800 kg/m²). The minimum ultimate concrete strength is 18.6 MPa (190 kg/cm²). The plank can span at 3 m. In Thailand, the law only requires 1.47-2.45 kPa (150-250 kg/m²) of live load for residential design.

Table 2 Concrete Plank Load Table

<table>
<thead>
<tr>
<th>Length, m</th>
<th>2.00</th>
<th>2.20</th>
<th>2.40</th>
<th>2.60</th>
<th>2.80</th>
<th>3.00</th>
<th>3.20</th>
<th>3.40</th>
<th>3.60</th>
<th>3.80</th>
<th>4.00</th>
<th>4.20</th>
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<tbody>
<tr>
<td>Numbers of Strands</td>
<td>Live Load, kg/m² (5 cm. concrete topping)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 φ 4 mm.</td>
<td>1500</td>
<td>1300</td>
<td>1100</td>
<td>900</td>
<td>700</td>
<td>600</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 φ 4 mm.</td>
<td>1700</td>
<td>1500</td>
<td>1300</td>
<td>1100</td>
<td>900</td>
<td>800</td>
<td>650</td>
<td>550</td>
<td>400</td>
<td>300</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td>6 φ 4 mm.</td>
<td>2000</td>
<td>1700</td>
<td>1500</td>
<td>1300</td>
<td>1100</td>
<td>900</td>
<td>700</td>
<td>600</td>
<td>500</td>
<td>400</td>
<td>300</td>
<td>200</td>
</tr>
<tr>
<td>7 φ 4 mm.</td>
<td>&gt;</td>
<td>1900</td>
<td>1700</td>
<td>1500</td>
<td>1300</td>
<td>1100</td>
<td>900</td>
<td>700</td>
<td>600</td>
<td>500</td>
<td>400</td>
<td>320</td>
</tr>
</tbody>
</table>

3. TEST RESULTS

From the first specimen test results, failure occurred at a load of 11.5 tons as shown in Fig 10. The CFS wall frame was not damaged and the concrete plank failed in bending where concrete cracked. Cracking of the concrete slabs starts at the top of concrete plank over the internal support when the load was approximately 4.2 tons as shown in Fig 11. The crack at the bottom of concrete slab at the middle of span initially occurred at a load of approximately 8.4 tons as shown in Fig. 12. When a load of 9.5 tons was applied, the midspan region of concrete slabs cracked further due to the positive moment. After increasing an applied load, the positive moment cracking of concrete at midspan and negative moment concrete cracking at internal support have formed a failure mechanism at a load of 11.5 tons as shown in Fig. 10. In Fig. 13, the CFS wall track showed sign of elastic distortion of the flanges, however, when the load is no longer applied, the track flanges returned to original shape.

Fig. 10 The first specimen at failure
Fig. 11 Concrete crack over internal support

Fig. 12 Concrete crack at midspan due to positive moment

Fig. 13 Distortion in flanges of track

For the second and third specimens, the test results can be concluded as shown in Table 3. All three tests showed similar results and using shear key did not have any significant effect on the gravity load carrying capacity of the specimens. Fig. 14-16 shows the failure of the second and third specimens. Fig. 17 shows the plots of load versus deflection of the third specimens. The LVDTs are used to monitor deflections at
different locations. The results show that any significant deflections only occurred at the concrete plank.

Table 3 Concrete Plank Load Table

<table>
<thead>
<tr>
<th>Test #</th>
<th>Load at Initial Crack at Internal Support (tons)</th>
<th>Load at Cracks at Middle of Span (tons)</th>
<th>Load at Failure (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.2</td>
<td>8.4</td>
<td>11.5</td>
</tr>
<tr>
<td>2</td>
<td>4.0</td>
<td>8.0</td>
<td>11.5</td>
</tr>
<tr>
<td>3</td>
<td>3.5</td>
<td>6.0</td>
<td>10.4</td>
</tr>
</tbody>
</table>

Fig. 14 The second specimen at failure

Fig. 15 The third specimen at failure
Fig. 16 Concrete plank failure of the third specimen

Fig. 17 Plots load vs deflection of the third specimen
From the experiment, three structural assemblies have shown failures in the concrete plank. The failures were ductile because of the steel reinforcements in concrete plank. CFS wall frame did not have any damage, however, the top tracks have some distortion in the flanges during loading. Once unloading, the distortion disappeared. In the second and third specimens, the maximum spacing between studs is up to 0.7 m. The spacing of 0.7 m. is wider than the width of each concrete plank. From the experiment, there is no failure occurred at the top track where there is no stud to directly support some of the concrete plank. The concrete topping help spread the load over the wall studs. Fig. 18 shows the structural analysis of the total failure load over the concrete plank. Using formula in Fig. 19 to calculate the uniform loading at failure, the uniform loading can be calculated to be 275 kPa (2808 kg/m²) compared to 1.47-2.45 kPa (150-250 kg/m²) required for residential construction in Thailand. From the failure load of 11.5 tons for C7510 wall stud, it can be calculated that each stud could carry 1438 kg, the nominal capacity of the C7510 according to the AISI (2016) is 1390 kg. From the failure load of 11.5 tons for C7575 wall stud, it can be calculated that each stud could carry 1150 kg., the nominal capacity of the C7575 according to the AISI (2016) is 914 kg. The calculations indicated that the load exceeded the calculated nominal capacity of each stud. However, as a system, there is no sign of stud failure from three tests.
5. CONCLUSION

From the experiments of the structural assemblies using precast concrete plank resting on CFS framing stud walls, the failure mode were the failure of concrete plank in bending. The CFS wall stud showed no damage during the tests. Shear key did not have any significant effect on the gravity load carrying capacity of the assembly. The failure mode was ductile failure because steel reinforcements in concrete plank can hold the cracked concrete intact. The structural assembly has sufficient gravity load carrying capacity for residential construction loading in Thailand.

REFERENCES