Research on Tight Seam Stitching Technology for Bidirectional Composite Floor Slabs

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Abstract. A new type of tight seam stitching technology for bidirectional composite floor slabs is proposed in this paper. Full-scale tests on the construction of slab joints were carried out and contrasted with cast-in-place slabs. The mechanical properties of a new type of joint structure were studied, including the failure mode, bearing capacity, stiffness, and crack distribution of a floor slab. Based on these tests, a refined finite element analysis was performed, and then the mechanism of the force transfer between a steel bar and the concrete at the joint was deeply studied. The results show that the mechanical performance of the new joint of the tight seam stitching was essentially the same as that of the cast-in-place structure. The numerical calculation agreed well with the experimental results. A closed annular steel bar at a joint location could effectively transfer the structural internal force.

Introduction

A composite floor slab is an assembled monolithic slab structure that consists of a prefabricated concrete floor with a cast-in-place reinforced concrete topping. It has the characteristics of industrialized production, standardized design, on-site assembly, and good integrity. Therefore, it has been widely used in the prefabricated concrete structures that have been promoted in China [1].

The connection joints between bidirectional composite floor slabs in China are specified by the Chinese technical specification for precast concrete walls [2] and the Chinese atlas for the connection details of precast concrete structures [3]. Using an integral seam is one of the methods, as shown in Fig. 1a. There should be 200-300 mm-wide post-cast belt formed between slabs. In addition, horizontal reinforcing bars should be left on the side of the bottom slabs (Fig. 2). However, this type of composite floor slab with a four-sided reinforcing bar faces problems such as a low degree of standardization and low construction efficiency. First, different sizing molds need to be processed according to the different spacing of the horizontal reinforcing bars during production. Then the reinforcing bars need to be placed one by one based on the openings in the molds. Second, it is necessary to use formworks and supports for the post-cast belts during construction (Fig. 3). Furthermore, during installation, there are severe collisions between the horizontal reinforcing bars of the composite floor slabs.
In order to overcome the above problems, the technical specifications [2] also provide another method that uses tight seams for bidirectional composite floor slabs, as shown in Fig. 1b. The technical specification puts forward higher requirements for the thickness of a cast-in-place reinforced concrete layer to ensure the effective height of the floor at the joint. The height should be no less than 100 mm and 1.5 times the height of the bottom plate, which results in a relatively higher comprehensive cost for the slab. The economic factors greatly affect the promotion of this type of slab. Zhai et al., Wang et al., and Jiang et al. conducted tests on tightly-bonded composite floor slabs that used additional steel bars at the joint [4–6]. The results showed that the cracks and the deflection deformation in the bottom of the composite floor slabs had obviously bidirectional force characteristics. Qian et al. [7] of the Harbin Institute of Technology performed tests and finite element analysis. The results fully proved that composite floor slabs with disconnect-type seams had bidirectional force characteristics. Ye et al. [8] stated that the arrangement of steel bars at the joints could control the development of cracks on the surface of composite floor slabs and improve the bearing capacity and ductility of prefabricated units. Jeong [9] also indicated that the bearing capacity of whole composite floor slabs depended on the structural design of the seam. Yun et al. [10] pointed out that the bearing capacity of tightly-bonded composite floor slabs was greatly affected by the
distance from the truss reinforcement to the joint. Based on the experiment results, the shorter distance led to larger bearing capacity and rigidity for the slabs. The distribution of the stressed cracks was also relatively uniform. In addition, the prefabricated units and the cast-in-place concrete layer had better integrity.

The above research shows that tightly-bonded composite floor slabs can achieve the same bearing capacity as that of cast-in-place structures, but the thickness of the cast-in-place layer needs to be increased, and the truss spacing between the two sides of the joint needs to be encrypted at the same time. In order to maintain the advantage of composite floor slabs without a four-sided reinforcing bar without increasing the thickness of the cast-in-place layer or the spacing of the truss reinforcement, this paper proposes a new tight seam stitching technology. The mechanical properties of the joints of the composite floor slabs, including the failure mechanism, bearing capacity, stiffness, and crack distribution, were studied via experimentation and finite element analysis. In addition, this paper focuses on the effect of closed annular steel bars on the performance of composite floor slabs. The force transmission mechanism between the steel and the concrete at the joint was also studied in depth using finite element calculation, which provided a basis for the design of the tight seam stitching of the bidirectional composite floor slabs.

**Joint Structure**

**Groove Design**

In order to increase the effective height of the floor slab at the joint location, a groove perpendicular to the slat was provided on the side of the prefabricated concrete bottom plate. The groove spacing was set between two stressed steel bars. The width was the difference between the spacing of the steel bars and 50 mm. The prefabricated concrete bottom plate depth was 60 mm, and the groove depth was 30 mm. The lapped steel bars were placed inside the groove, so the groove length was controlled by the lapped length of the steel bars. Generally, the lapped length of steel bars should not be less than 300 mm. It causes the length of the groove to be comparatively long. Thus, the 30 mm plate at the bottom of the groove was easily damaged during production, transportation, and construction.

**Closed Annular Steel Bar**

In order to reduce the groove length, a closed annular steel bar at the joint location was used instead of a conventional straight lapped steel bar. The closed annular steel bar was a self-contained independent closed-loop system. The anchorage failure bearing capacity was not controlled by the bond stress of the steel bar, but rather controlled by the shear bearing capacity between the detached concrete and the surrounding concrete. This greatly reduced the anchorage length of the steel bar. The closed annular steel bar was designed in the form of a herringbone tripod to facilitate the on-site placement and to improve the joint strength and rigidity of the joint. The steel bars acted as the longitudinal reinforcement of the bottom slab; i.e., the steel quantity for the closed annular steel bars was double than what it was in the longitudinal reinforcement at the bottom of the slab.

To improve the joint strength between the concrete of the closed annular joint enclosed area and the surrounding concrete, the longitudinally distributed steel bars in the prefabricated bottom slab were bent upward at the end of the plate so that they protruded from the prefabricated slab surface. This joint structure also further enhanced the bonding properties between the prefabricated slab and the cast-in-place laminate layer. The schematic representation for new tight seam stitching is shown in Fig. 4.
Static Loading Test

Description of the Specimens

The static loading tests of the cast-in-place slabs and the prefabricated slabs under the same conditions were carried out to study the mechanical properties of the tight-seam joints mentioned above. The prefabricated specimen was 1500 mm long and 780 mm wide. The prefabricated slab was 60 mm thick, and the cast-in-place laminated layer was 80 mm thick. The total length of the prefabricated specimen was 3000 mm, as shown in Fig. 5. There were five longitudinal steel bars with diameters of 8 mm in the upper and lower slabs. The protective layer for the longitudinal steels was 20 mm, and the concrete strength grade was C30. The prefabricated slab had four grooves at the end, and each groove measured 200 mm long×110 mm wide×30 mm deep. Except for the joint structure, the size and the reinforcement of the cast-in-place specimen were the same as those of the prefabricated specimen.

Material Properties

The concrete of the prefabricated specimen was poured in two parts. The prefabricated bottom slab was poured first, and the upper laminated layer was poured after one week. The cast-in-place specimen was poured while pouring the laminated layer of the prefabricated specimen. Three cubic specimens of 150 mm×150 mm×150 mm were reserved for each concrete pouring. The concrete specimen was cured for 28 days until the day of the test. The average compressive strength of the concrete specimen was measured as 35.3 MPa. The longitudinal reinforcement and the lapped steel bars in the specimen were all made of HPB300 steel. The yield strength of the steel was approximately 350 MPa.
Test Setup and Measurement

The diagram of static loading for the slab is shown in Fig. 6. The two ends of the slab were simply supported constraints. The support had a 100 mm out-extended section. A distributive girder was arranged on the slab, and there were spacers at the load location to avoid a local stress concentration. A distributive girder was used for three-point symmetric loading to produce a pure bending zone in the mid-span section. The length of the pure bending zone of the slab was 900 mm. The jack achieved monotonic static loading through the distributive girder system, and the load sensor controlled the loading value. The dial gauge displacement measuring points were arranged on the midspan section. The test data were collected automatically by a computer.

![Figure 6. Test loading diagram.](image)

Testing Phenomenon

When the cast-in-place specimen was loaded to 8 kN, the first crack started to appear at the bottom of the plate. When the load was increased to 10 kN, the second crack appeared. When the loaded was 15 kN, the number of cracks that could be observed by the naked eye reached five. Furthermore, the second crack became the main crack, which had penetrated the entire bottom of the slab. The maximum crack width reached 1.5 mm. At this moment, the mid-span deflection reached 2 mm. The loading could not be maintained when the cast-in-place specimen was loaded to 25 kN. The deflection and the crack width kept getting larger, and the specimen eventually lost its load carrying capacity.

The failure mode of the prefabricated specimen was essentially the same as that of the cast-in-place specimen. When the prefabricated specimen was loaded to 6 kN, the tight seam joint was pulled open, and the first crack that was visible to the naked eye appeared. When loading to 15 kN, the second and third cracks appeared one after the other. The first crack at the seam did not extend. When the load was increased to 23 kN, the number of cracks that could be observed by the naked eye reached six. The main crack was not at the joint location, but a 2-mm-wide crack neared the loading location. Simultaneously, the mid-span deflection reached 2 mm. The loading could not be maintained when the prefabricated specimen was loaded to 27 kN. The deflection and the crack width kept getting larger, and the specimen eventually lost its load carrying capacity. The crack distribution at the bottom of the slab is shown in Fig. 7.

![Figure 7. Shape of the cracks on the slabs.](image)
Finite Element Analysis

Model Parameter
It can be seen from the above test phenomenon that during the static loading process of the floor, with the increase of the load, the concrete in the tension zone of the bottom of the slab cracked. The rigidity of the concrete rapidly degenerated after the cracking. The tensile stress was mainly born by the steel bar. The degeneration process of the concrete cracking stiffness was a highly nonlinear process. The finite element software Abaqus general was applied to accurately simulate the nonlinearity of the concrete during the tension process of the slab (Fig. 8). Based on the concrete plastic damage model, the fine models of the specimen concrete slabs and the steel bars were established separately. The deformation characteristic of the slab loading process was analyzed. The concrete adopted the C3D8R solid element, and the steel bar adopted the T3D8 rod unit double-fold line-strengthening model.

The shear resistance of the laminated layer of the concrete composite floor slabs could ensure that the bending and shear bearing capacity of the laminated layer was fully exerted without shearing damage along the laminated layer using natural vibration, troweling, and galling. In addition, the concrete and steel bars were connected by embedding in the software because they had good bond stress. It was assumed that the steel bars and the concrete were co-deformed without slip deformation, and all the tensile cracking occurred in the internal concrete. Rigid spacers were arranged on the load application location of the finite element model to avoid stress concentration caused by the concentrated force. The model mesh size was approximately 25 mm×25 mm, as shown in Fig. 9. The center position of the left support of the model constrained the vertical and horizontal displacements, and the center position of the right support only constrained the vertical displacement and reached the boundary condition of the simply supported constraint.

Analysis of the Bearing Capacity and Stiffness
The load-deflection curves of the composite floor slabs and the cast-in-place slab are shown in Fig. 10. The Figure also presents the finite element numerical calculation curve. It can be seen from the Figure that the numerical calculation agreed well with the experimental results. Before cracking, the deformations of each specimen are linear. The deflections of each specimen were small, and there were only slight differences between them. After cracking, the deflections had tendencies that were clearly different than those they had before. In the finite element analysis, when the corresponding load was approximately 2 mm, the concrete in the tension zone of the slab cracked, and the load-displacement curve had a significant inflection point. When the load reached 15 kN, the slopes of the curves for each specimen began to decrease. Combined with the test phenomenon, the reason for this was that the concrete cracking on the surface of the slabs caused the floor stiffness to decrease under this load. After the load reached 25 kN, the displacement of each specimen increased rapidly. This indicates that the steel bars in the slabs had begun to yield. It can be seen from the test curve that the initial stiffness and the cracking load of the prefabricated specimen were slightly larger than those of the cast-in-place specimen. The main crack was not at the joint. From the perspective of the bearing...
capacity and stiffness, it can be considered the same as that of the cast-in-place structure. The main reason for this was that the actual amount of steel bars at the joints was larger than that of the bottom of the prefabricated slabs. Additionally, the effective thickness of the slabs was essentially the same as that of the cast-in-place slabs.

![Figure 10. The load-deflection curve.](image)

**Stress Analysis of Reinforcement**

In order to further analyze the stress and deformation characteristics of concrete and steel bars in the stress process of the new joints, the stress of the steel bars was further analyzed. When the composite floor slab and the cast-in-place slab corresponded to a mid-span displacement of 20 mm, the steel bar tensile stress in the slabs was as shown in Fig. 11. The three-point loading range of the cast-in-place slab corresponded to the uniform stress of the longitudinally stressed steel bar in the pure curved section, and the longitudinal steel bar stress reached a yield strength of 300 MPa. The longitudinally stressed steel bars of composite floor slab were broken at the joint. The stress of the eight closed annular steel bars at the joint was approximately 160 MPa. The stress values of the five tension bars in the prefabricated slab reached the yield strength. The tensile stress at the joint was effectively transmitted to the longitudinal tension of the prefabricated slab through the closed annular steel bars.

![Figure 11. Reinforcement stress contour.](image)

**Stress Analysis of Concrete**

In the finite element analysis, the concrete was simulated by a plastic damage model. The degenerate of the concrete stiffness was used to describe the work performance of the concrete after cracking. When the mid-span displacement of the slab increased to 20 mm, the distribution of the concrete damage factor for the composite floor slab and the cast-in-place slab was as shown in Fig. 12. The basic cracking damage stiffness of the concrete in the three-point loading pure bending section was significant. The damage factor was above 0.9. This indicates that the concrete in the tension zone was essentially disabled after cracking. The concrete damage factor distribution of the composite floor slab and the cast-in-place slab was essentially the same.
The internal force between the annular lapped steel bars and the longitudinal steel bars in the prefabricated slabs was transmitted by the shear bearing capacity of the shaded concrete in Fig. 13. The shear capacity of the concrete was

\[ F_v = n f_v A/2, \]  

(1)

where \( n \) is the number of the shear planes, \( f_v \) is the shear strength of concrete, and \( A \) is the shaded area. In this study, \( n \) was set to be 8, \( f_v \) was set to be 3.55 MPa, and \( A \) was set to be 20,000 mm\(^2\).

The shear capacity of the concrete in the core area calculated using the Eq. (1) was 284 kN. The tensile capacity of the five HPB300 steel bars was 75 kN, which is much smaller than the shear failure of concrete. Therefore, the specimen would not have concrete shear failure in the core area of the closed annular steel at the joint. Tensile yielding of the steel bars could only occur in the same manner as the cast-in-place slab outside the joint location. Thus, the bearing capacity of the joint was greater than the strength of the prefabricated unit.

**Conclusions**

This paper proposes a new type of tight seam stitching technology for bidirectional composite floor slabs. Through the static load testing of specimens of cast-in-place slabs without seams and composite floor slabs, the entire process from the crack initiation to the eventual failure of the slabs was observed. In addition, the deflection of the specimen under various loading levels, the change of the flexural capacity, and the development of the plastic hinge were analyzed. Based on the finite element analysis, the following conclusions were obtained.

1. The placement of steel bars in the groove of the prefabricated bottom slab increased the effective height of the floor at the joint location. Thus, the rigidity of the composite floor slab was improved, while the deflection was reduced. The slab achieved the design expectation.
2. The closed annular steel bars effectively shortened the length of the grooves. At the same time, they also increased the reinforcement ratio of the steel bars at the joint location and contributed to the improvement of the rigidity and strength of the joints. Therefore, the cracks no longer first appeared at the joint location. The joint location was also not the final plastic hinge position. The tight seam stitching technology achieved the original intention of an equivalent cast-in-place structure.

3. Through numerical simulation of a finite element method, the force transmission mechanism of the closed annular steel bars and concrete was further revealed. The new type of tight seam stitching technology formed a loop connection between the closed annular steel bars and the bent longitudinal reinforcement in the prefabricated slab. The numerical calculation essentially agreed with the experimental results. Moreover, the lapped length of the steel bar was greatly reduced.

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