Experimental Investigation on the Behaviour of Non-reinforced Ultra-High Performance Concrete Slabs

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Abstract. Ultra-high-performance concrete (UHPC) is expected to provide solutions for the development of lightweight, high-strength, and rapid construction of concrete bridges due to its excellent material properties. In order to study the influence of steel fiber on the bending performance of non-reinforced UHPC (NR-UHPC) slabs, the bending failure test of 8 NR-UHPC slabs was completed with the steel fiber content as a parameter, and the failure mode, load-deflection, crack widths and load-strain curves of the test slabs were analyzed. The test results show that the destruction process of the NR-UHPC slabs can be divided into three stages: elastic stage, cracking stage and failure stage. The load-deflection curve and load-strain curve of the specimen with steel fiber content of 0.5% -1% change linearly during the elastic phase. In the elastic phase, the load-deflection curve and the load-strain curve of the test piece with a steel fiber content of 2% -3% were not smooth after the linearity begins; the strain of the UHPC slab was almost the same within 0-10 kN. However, as the load increased, the larger the amount of steel fiber was, the smaller the strain was, that is, the UHPC slab had good crack resistance. According to the regression analysis of test results at home and abroad, recommended design formulas for the bending bearing capacity of NR-UHPC slab were put forward, which tally well with the test results.

Keywords. Ultra-high performance concrete slab, steel fiber content, failure state, flexural bearing capacity.

1. Introduction
Ultra high performance concrete (UHPC) is a type of cement-based composite, which is the most innovative product in concrete technology during the last 30 years. It was developed in Europe in the 1980s for specialized applications that demand superior strength and corrosion resistance [1]. UHPC, which has better mechanical properties and durability than reinforced concrete, can greatly reduce the weight of structures due to its high strength and reliability. UHPC with a high compressive strength of more than 200 MPa and an improved durability marks a quantum leap in concrete technology. This high-performance material offers a variety of interesting applications. It allows the construction of sustainable and economic buildings with an extraordinary slim design. Its high strength and ductility makes it the ultimate building material e.g. for high-rise buildings, long-span bridges and bridge decks [2-5].

More recently, the use of UHPC has expanded to applications requiring its high strength in narrow profiles, such as bridge spans and building façades in which the material’s strength, wear resistance, lighter weight and lower life cycle costs have been the driving determinates. The use of UHPC materials in field of architecture predates that in bridge engineering, on which UHPC first appears in
1997. There are quite a few UHPC bridges worldwide, and UHPC is used as a major or part of building materials in bridges mainly exist in Asia, Europe and North America. Existing research shows that it is completely feasible to apply UHPC slab to actual engineering. The world’s first engineering structure designed with UHPC was the Sherbrooke footbridge in Sherbrooke, Quebec, built in 1997, and the bridge deck is a thin slab 30 mm thick [6], which not only reduces the weight of the structure, but also allows the structure to withstand high humidity and freeze-thaw cycles. Besides, its durability under environmental conditions has significantly improved. Since then, countries around the world have used UHPC slabs in pedestrian bridges or highway bridges, and have achieved good results [7-8]. China's research and application of UHPC started late, but has developed rapidly. In 2003, the Shijingshan swivel cable-stayed bridge isolation zone in Beijing used unreinforced active powder concrete hollow slabs, which was the first application of UHPC materials in China [9]. In addition, small-scale applications of UHPC have been tried in Qianbaiqu Bridge of Qian-Cao Railway, Yigang Railway, and Qinghai-Tibet Railway [10-12]. The Changsha Beichen Hengsi Road Crossover Bridge, which was opened to traffic in January 2016, is the first comprehensive application of China’s first UHPC bridge in China's bridge structure [13].

Although this technology has been developed in recent decades, yet, its presence still remains in the infant stage for many developing nations. In 2015, Zheng et al. [14] studied the bending load of a new type of prestressed UHPC floor, analyzed the crack characteristics and deflection changes of this type of UHPC floor, and gave the applicable Calculation method of normal section bearing capacity. In 2016, Chen Hongwei et al. [15] carried out bending capacity test on UHPC slab without reinforcement. The test results show that the failure mode of UHPC slab is ductile failure, and the ultimate strength of slab is affected by the content and distribution of steel fiber. The theoretical calculation method of slab bearing capacity is analyzed, and the ultimate strength of slab is converted into ultimate bending strength Solution. In 2017, Mahmood et al. [16] carried out bending mechanical performance tests on hollow UHPC cover slabs with different parameters, and the results showed that the parameters such as the ratio of material bubble diameter to slab thickness have a greater impact on the bearing capacity of this type of cover slab. In 2019, Huang Fuyun et al. [17] carried out bending tests on UHPC slabs without reinforcement and ordinary reinforced concrete slabs. Studies have shown that the flexural failure of UHPC slabs is ductile failure, and the actual engineering is given by finite element parameter analysis. The recommendations of UHPC sheet steel fiber content, slab thickness, and material grade can provide a certain reference for the engineering practice and the formulation of related specifications.

So far, there has been lack of research on the flexural bearing capacity of NR-UHPC bridge decks. The theoretical calculation method involving the flexural bearing capacity of UHPC slabs is not yet perfect, which has seriously hindered the promotion and application of UHPC slab members in China. Therefore, in this paper, the bending bearing test of 8 NR-UHPC slabs was carried out to investigate the influence of the amount of steel fiber on the bending bearing capacity of UHPC slabs. It can provide a certain reference for engineering practice and corresponding specifications.

2. Test Program

2.1. Test Materials and Properties
The mixing sequence for UHPC material is described the previous studies [1-4]. It is noted that the process of mixing sequence is different from that of ordinary concrete, as a significantly lower water-to-binder ratio was used and coarse aggregate was excluded from the UHPC mix. The UHPC material mix with a target compressive strength of 150 MPa and a target split strength of 12 MPa was used in this study; its mix proportions are summarized in table 1. Type I Portland cement and silica fume were used for the cementitious materials with a water-to-binder ratio (W/B) of 0.16. Based on the packing density theory, silica sand with a grain size less than 0.6 mm was used as the fine aggregate, and silica flour with an average diameter of 0.1 μm and containing over 98% SiO₂ was used as the filler. Straight type micro-steel fibers with the length (Lf) of 13 mm and the diameter (df) of 0.2
mm, resulting in an aspect ratio (lf/df) of 65, were added to the UHPC material mixes up to 2% of the total volume in order to improve the tensile capacity of UHPC material. These micro-steel fibers are characterized by the density of 7.8 kg/m$^3$, the tensile strength of 2000 MPa, and an elastic modulus of 200,000 MPa. As significant volumes of cementitious materials and micro steel fibers can result in poor workability of UHPC material, the polycarboxylic acid high performance water-reducing admixture (super plasticizer) was added to provide proper fluidity.

Table 1. Mix proportions of UHPC material by relative weight ratios to cement.

| W/B | Water | Cement | Silica fume | Silica flour | Quartz sand | Super plasticizer | Steel fiber$^2$ |
|-----|-------|--------|-------------|-------------|-------------|------------------|--------------|
| 0.16| 0.224 | 1.0    | 0.25        | 0.3         | 1.11        | 0.02             | 2%           |

$^1$ W/B water-to-binder ratio.  
$^2$ Volume percent of steel fiber in a 1 m$^3$ UHPC material mix.

The test specimens were placed in two matched castings. After casting, they were covered with a thin insulation and waterproof film. After 24 hours, the specimens were demoulded. Afterwards, they were cured for 72 h in 80± 2 ℃ hot water and gradually cooled for another 48h in water. Those cubes and prisms for determining the properties of UHPC were cured at the same manner as the test specimens. At last, the beams were stored in the laboratory until the day of test. The properties of UHPC tested were summarized in table 2.

Table 2. Mechanical properties of UHPC.

| Cube compressive strength | Split strength | Modulus of Elasticity |
|---------------------------|---------------|-----------------------|
| 148.7                     | 12.9          | 42.5×10$^3$           |

2.2. Design and Production of Test Specimens

Eight NR-UHPC slabs of 750 m in length were designed with 300 mm in width and 60 mm in thickness. The main research parameter of the test was the steel fiber content, which was 0.5%, 1%, 2%, and 3%, respectively. For the convenience of expression, the specimens were numbered in the form of U-X-N, where U was UHPC; X was the steel fiber volume content, namely, 0.5, 1, 2, and 3, respectively; N was the serial number of the same group of specimens, and two specimens in each group were labeled as 1, 2 in turn. For instance, U-0.5-1 represented the first specimen in the series with the steel fiber volume content being 0.5%. The Details of specimens were shown in table 3.

Table 3. Details of specimens for NR-UHPC slabs.

| Specimen | Size (mm) length×width×thickness | Number of specimens | Steel fiber content (%) |
|----------|----------------------------------|---------------------|------------------------|
| U-0.5-N  | 750×300×60                       | 2                   | 0.5                    |
| U-1-N    | 750×300×60                       | 2                   | 1                      |
| U-2-N    | 750×300×60                       | 2                   | 2                      |
| U-3-N    | 750×300×60                       | 2                   | 3                      |

2.3. Test Setup and Procedure

The test setup consisted of a simply supported slab under centralized load, as shown in figure 1. At the supports, load was transferred to the beam using a 50 mm diameter roller and 100×150×25 mm steel plates. At the point load, a 50 mm diameter roller was sandwiched between 100×150×25 mm steel bearing plates.

The applied load was measured by a load cell while deflections were measured by linear variable
differential transformers (LVDTs) at mid-span, load points and the supports. In addition, three LVDTs were located in the bottom regions of the specimens to measure concrete strains. A schematic layout of the measuring points is shown in figure 2.

![Schematic layout of measuring points](image)

**Figure 1.** Test setup and instrumentation: (a) test setup, and (b) photo of test devices.

Pre-load the slab before the test loading to check whether the loading device and instrument are working properly. Load was applied in 1 kN increment up to the cracking load, then, 2 kN increment was used. After the beam entered into the remarkable nonlinear range, the loading was controlled using the deflection increment of 3-5 mm at mid span until final failure of the beam. During the loading process, the vertical load, the transverse crack width, and the vertical displacement are automatically recorded at a frequency of 5 seconds. When the bearing capacity of the slab starts to decrease, record the value of the transverse tensile crack width, and continue to load the specimen when the bearing capacity is less than 70% of the peak load, which can be regarded as the specimen failure.

**3. Main Test Results**

**3.1. Failure Form and Load-Span Deflection Curve**

Photographs of the failure modes of each test slab in the limit state are shown in figure 3, and the load-span deflection curve of each test slab is shown in figure 4. The failure process is divided into an elastic phase, cracking phase and failure phase.
Figure 3. Failure modes of specimens: (a) U-0.5-1~U-3-1, and (b) U-0.5-2~U-3-2.

Figure 4. Load-deflection curve: (a) U-0.5-1~U-3-1, and (b) U-0.5-2~U-3-2.

Elastic phase. It can be seen from figure 4 that from the beginning of loading, the test specimens have all gone through the elastic phase with a linear relationship between load and deflection. At this stage, the deflection in the mid-span is very slow, and the entire load-deflection curve rises quickly. From the load-deflection curve in figure 4, it can be seen that the greater the amount of steel fiber, the greater the elastic limit, that is, the longer the elastic stage, which indicates that the amount of steel fiber has a significant effect on the elastic stage of the test piece. When the deflection reaches 0.37 ~ 1.53mm, each specimen has reached the elastic limit load, which is the critical state of cracking of the specimen, and its corresponding load can also be called the cracking load. When the cracking load is reached, which is point A in figure 4, the first visible crack appears near the middle of the tensile zone span of the bottom of the specimen. The deflection corresponding to the cracking load of eight specimens increased with the increase of steel fiber content.

Cracking stage. At this stage, the slab begins to deform plastically, the deflection increases rapidly compared to the elastic stage, and the load-deflection curve begins to slow. The steel fiber under stress in the tensile zone of the slab began to yield, the bond between the concrete and the steel fiber began to be damaged, the steel fiber was gradually pulled out from the concrete, and the stress in the tensile zone was caused by the remaining matrix and steel fiber bonded together. The frictional force of the concrete being pulled out is taken together. In this state, the load on the test piece is still increasing, which is obviously different from that of ordinary concrete when it cracks, mainly because steel fibers can participate in the tensile force. At this stage, the load and deformation are no longer linearly proportional, and the development of deformation is faster, which is caused by the continuous development and extension of cracks. As can be seen from table 4, the ultimate load of the UHPC slab is increasing with the increase of the steel fiber content, that is, point B in figure 4, but as the content of the steel fiber is increasing, the ultimate load of the UHPC panel is increasing slowing down.
The destruction stage. At this stage, the bearing capacity of the slab began to decline, and the curve load began to decline slowly after reaching the peak. From figure 4, it could be seen that as the amount of steel fiber increases, the bearing capacity of the slab decreases faster, with the content of 3%. UHPC slabs are the most obvious. During the entire process of the destruction phase, the steel fibers were gradually pulled out of the concrete. At this time, the cracking sound of the steel fibers being pulled out from the concrete can also be heard. The crack at the bottom also rapidly extended, and the deflection value was still increasing. When each specimen was loaded to a bearing capacity of about 70% of its ultimate load, the specimens were not loaded and the specimens were not completely broken. During the entire loading process, each test piece was ductile, indicating that the steel fiber provided a certain reinforcing steel bridging effect during the bending process, so that the test piece would not be brittlely broken. With the increased of steel fiber content, the ductility of UHPC slab was better.

3.2. Strain at the Bottom of the Slab

For a more vivid description of the development of the strain at the bottom of the slab, and the strain at the bottom of the slab at 0-7000 micro strain were showed in figure 5.

![Figure 5](image)

**Figure 5.** Load-strain curve: (a) U-0.5-1–U-3-1, and (b) U-0.5-2–U-3-2.

From figure 5, it can be seen that the load-strain curve of each test piece is linear in the range of 0-10 kN, and the strain of each test piece is almost the same. The test specimens with 0.5% and 1% steel fiber content crack and the strain suddenly changes. In the load range of 15-23 kN, the load-strain
curve of the test specimens with 2% and 3% steel fiber content is not smooth, and it is still in the elastic loading stage. Under this stage, the UHPC slab plain concrete cracks gradually, causing a sudden change in strain. When the load is 15 kN, we can clearly see that the strain of the test piece is gradually reduced when the steel fiber content is 1-3%, that is, the steel fiber can effectively limit the development of cracks, and the larger the steel fiber content.

4. Calculation of Bending Capacity

4.1. Overview of Calculation Methods for UHPC Slab Bending Theory

The test shows that when the UHPC slab reaches the ultimate limit of bending capacity, the cracks have developed to a certain height, part of the matrix in the tensile zone loses the bearing capacity due to cracking, the steel fibers in the tensile zone have reached the ultimate bonding strength, and the cross-section compression zone. At this time, it is still in the elastic stage. Due to the lack of reinforcement, the height of the compression zone is small when the bearing capacity limit state is reached, and the corresponding section compressive strain is also small. Based on the flat section assumption and the UHPC constitutive relationship given in [18], a calculation model for the bending capacity of UHPC slabs can be established. According to the principle that the magnitude and position of the UHPC combined force in the compressed and tension zones are unchanged, the compressed zone can be equivalent to a triangle. Sectional strain and stress distribution at the limit state are shown in figure 6.

\[ \varepsilon_{top} = \frac{x}{h-x} \varepsilon_{tu} \]  

\[ \sigma_{top} = \frac{x}{h-x} \varepsilon_{tu} E_c \]  

The resultant force in the compression zone is:

\[ C = \sigma_{top} \frac{1}{2} b x = \frac{\varepsilon_{tu} E_c}{2} \left( \frac{x^2}{h-x} \right) \]  

The combined force in the tensile zone is:

\[ T_1 = f_{to} \frac{\varepsilon_{el}}{\varepsilon_{tu}} (h-x) \frac{b}{2} \]
Theoretical moment of a section:

\[ T_2 = \frac{f_{ct}}{2} \left( 1 - \frac{\varepsilon_{ct}}{\varepsilon_{tu}} \right) (h - x) b \]  

(5)

The ultimate bending moment of each test piece can be obtained from the following equation:

\[ T = T_1 + T_2 \]  

(6)

Where \( \varepsilon_u \) is the ultimate compressive strain of UHPC slab with steel fiber content of 1% or more, which is obtained from regression of experimental data, \( \varepsilon_u = 0.01 \), where the value of \( \varepsilon_u \) in reference [19] is 0.01, \( \varepsilon_{ct} \) is the ultimate compressive strain of UHPC slab with steel fiber content less than 1%, and \( E = 2\varepsilon_{ct}, \) \( E \) is the elastic modulus of UHPC. According to the axial force balance condition, \( \sum N = 0 \), there were: \( C = T \), which can be solved for \( \varepsilon \). The bending bearing capacity of NR-UHPC slab can be obtained from equation (7):

\[ M_u = T_1 \left[ x + \frac{\varepsilon_{ct}}{\varepsilon_{tu}} (h - x) \right] \frac{2}{3} + T_2 \left[ \left( h - x - \frac{\varepsilon_{ct}}{\varepsilon_{tu}} (h - x) \right) \frac{1}{2} + \frac{2}{3} x + \frac{\varepsilon_{ct}}{\varepsilon_{tu}} (h - x) \right] \]  

(7)

Therefore, the calculation method for calculating the flexural capacity of the non-reinforced UHPC slab can be obtained.

4.2. Verification

The test data of 8 NR-UHPC slabs in this paper and the test data of ten NR-UHPC test specimen carried out in [20-24] were used to verify the calculation formula of the bending capacity. Table 5 shows the comparison between the theoretical calculation value \( M_u^c \) and the test value \( M_u^t \) of the ultimate bending moment of each test piece.

| Specimen | \( V_f \) | \( b \) | \( h_0 \) | \( L \) | \( f_{cu} \) | \( f_{ct} \) | \( \varepsilon_u \) | \( M_u^c \) | \( M_u^t \) | Source |
|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|--------|
| U-0.5-1  | 0.5      | 300      | 60       | 750      | 149      | 0.0003   | 6.99     | 7.41     | 1.85     | 1.96   | 1.062  |
| U-1-1    | 1        | 300      | 60       | 750      | 149      | 0.0100   | 6.99     | 7.84     | 3.05     | 3.62   | 1.188  |
| U-2-1    | 2        | 300      | 60       | 750      | 149      | 0.0100   | 6.99     | 8.69     | 4.29     | 3.86   | 0.901  |
| U-3-1    | 3        | 300      | 60       | 750      | 149      | 0.0100   | 6.99     | 9.54     | 5.00     | 4.10   | 0.821  |
| U-0.5-2  | 0.5      | 300      | 60       | 750      | 149      | 0.0003   | 6.99     | 7.41     | 1.73     | 1.96   | 1.131  |
| U-1-2    | 1        | 300      | 60       | 750      | 149      | 0.0100   | 6.99     | 7.84     | 3.08     | 3.62   | 1.176  |
| U-2-2    | 2        | 300      | 60       | 750      | 149      | 0.0100   | 6.99     | 8.69     | 4.05     | 3.86   | 0.953  |
| U-3-2    | 3        | 300      | 60       | 750      | 149      | 0.0100   | 6.99     | 9.54     | 5.04     | 4.10   | 0.815  |
| U-1      | 1        | 100      | 100      | 500      | 136      | 0.0100   | 6.40     | 7.02     | 3.26     | 3.03   | 0.932  |
| U-2      | 2        | 100      | 100      | 500      | 165      | 0.0100   | 7.76     | 9.27     | 3.46     | 3.85   | 1.114  |
| U-3      | 3        | 100      | 100      | 500      | 158      | 0.0100   | 7.42     | 9.60     | 4.56     | 3.88   | 0.851  |
| NR-U     | 3        | 300      | 100      | 1600     | 174      | 0.0100   | 8.17     | 10.56    | 12.45    | 12.77  | 1.025  |
| 3-PL     | 5.5      | 145      | 50       | 600      | 148      | 0.0100   | 6.94     | 9.81     | 1.45     | 1.40   | 0.967  |
| 4-PL     | 5.5      | 145      | 50       | 600      | 148      | 0.0100   | 6.94     | 9.81     | 1.43     | 1.40   | 0.977  |
| SS20     | 2        | 100      | 100      | 400      | 227      | 0.0100   | 10.68    | 12.76    | 6.19     | 5.22   | 0.844  |
| LS       | 1        | 100      | 100      | 400      | 227      | 0.0100   | 10.68    | 12.88    | 3.71     | 5.09   | 1.374  |
| M-A      | 1        | 125      | 125      | 1000     | 247      | 0.0100   | 11.59    | 12.75    | 12.18    | 10.46  | 0.859  |
| MF15     | 2.5      | 100      | 100      | 400      | 170      | 0.0100   | 8.01     | 11.77    | 3.60     | 4.54   | 1.264  |
| Average  |          |          |          |          |          |          |          |          | 1.014   |        |
| Standard deviation |          |          |          |          |          |          |          |          | 0.164   |        |

From Table 5, the average value of 8 UHPC slabs \( M_u^c / M_u^t \) in this paper was 1.006, and the average value of 10 UHPC slabs \( M_u^c / M_u^t \) in literature [20-24] was 1.021. Figure 7 is a comparison chart of
The average value of $\frac{M_{uc}}{M_{ue}}$ was 1.014, and the standard deviation was 0.164. It could be seen that the calculated value agreed well with the experimental value, and the calculated value had higher accuracy and reliability.

![Comparison of calculation results](image)

**Figure 7.** Comparison of calculation results.

According to the regression analysis of test results at home and abroad, recommended design formulas for the bending bearing capacity of NR-UHPC slabs were put forward, which tally well with the test results, which also can provide a certain reference for engineering practice and related specifications.

5. Conclusions
In this paper, the flexural behavior of NC-UHPC slab with different fiber content was studied through 8 specimens. The main conclusions are as follows:

1) The test results show that the destruction process of the NR-UHPC slabs can be divided into three stages: elastic stage, cracking stage and failure stage.

2) The cracking load of UHPC slab increases with the increase of steel fiber content, but the value of cracking load increases with the increase of steel fiber content, especially the steel fiber content is between 2% and 3%. This phenomenon is particularly obvious for UHPC slabs, which indicates that steel fibers have a certain cracking resistance.

3) The ultimate bearing capacity of UHPC slab also increases with the increase of the amount of steel fiber, but the rate of increase of the ultimate load also slows down with the increase of the amount of steel fiber. Volume ratio is controlled at 1.5% ~ 2.5%.

4) According to the regression analysis of test results at home and abroad, recommended design formulas for the bending bearing capacity of NR-UHPC slab were put forward, which tally well with the test results.

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