Behavior of One-Way Concrete Slabs Reinforced with GFRP Bars

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Abstract

The replacement of conventional steel reinforcement with fiber-reinforced polymer (FRP) bars was investigated previously to overcome the problem of steel reinforcement corrosion and structural deterioration in concrete structures exposed to aggressive environments. However, the lower modulus of elasticity of FRP materials and their non-yielding characteristic results in large deflection and wide cracks in FRP-reinforced concrete members. Hence, there is a need for a suitable design philosophy and for methods that can provide a reliable estimate of such behavior.

This paper evaluates the behavior of simply supported concrete slabs reinforced with bars of glass fiber-reinforced plastic (GFRP) and subjected to four-point monotonic loading. The slabs had sizes of 4000 × 1000 × 150 mm and 4000 × 1000 × 200 mm with different reinforcement ratios. This research investigated the flexural and shear limit states of the slabs, including pre-cracking behavior, cracking pattern and width, deflections, ultimate capacities and strains, and failure modes. The information presented is valuable for future field application and development of design guidelines for FRP-reinforced concrete structures.

Keywords: concrete; one-way concrete slabs; glass fiber reinforced polymer (GFRP) bar; reinforcement ratio

1. Introduction

Corrosion of steel reinforcing bars is one of the major problems that shorten the lifetime serviceability of reinforced concrete (RC) structures; this has led to the development of new concrete-reinforcing materials. With their high strength and good corrosion resistance, fiber-reinforced polymers (FRP) represent a good alternative. In comparison to steel, the distinctive properties of FRP materials are high strength, relatively low elastic modulus, and elastic response to failure. Given these different properties, the behavior of concrete elements reinforced with FRP is likely to differ markedly from those that employ conventional steel reinforcement. This difference is characterized not only by a different load-deflection response, but also by a change in the mode of failure. The failure mechanism of FRP-reinforced concrete elements is due to their being relatively brittle, even in flexure. This gives rise to major concerns by structural engineers who are more familiar with the under-reinforced design philosophy developed for steel RC structures, which ensures a ductile failure to give plenty of warning of incipient collapse. To facilitate the rapid adoption of FRP in concrete construction, most researchers working in this field have attempted to provide simple design equations using modified versions of existing predictive equations based on the well-established philosophy for steel-reinforced structures.

Several research projects have been carried out at Hanyang University, Korea, on the behavior of FRP-reinforced concrete structural members as part of a Korea Institute of Construction Technology (KICT) (2005) task group whose aim is the development of design and construction technology for concrete structures using advanced composite materials. This paper presents the flexural behavior of one-way concrete slabs reinforced with glass FRP (GFRP) bars.

2. Objective and Scope

Strength and deflection predictions for steel-reinforced concrete elements are dependent on empirical performance constants. The empirical component reflects the material-specific composite behavior of steel and concrete. This study clearly shows that the behavior of concrete slabs reinforced with GFRP bars is different from that of slabs reinforced with steel bars.

The objective of this paper is to investigate the flexural strength, shear strength, and deflection behavior of one-way concrete slabs reinforced with GFRP bars. This paper should also provide engineers...
and researchers with a better understanding of the performance of these new composite materials in concrete structures.

3. Experimental Program

3.1 Materials

The slabs were constructed using concrete provided by a local supplier. The target compressive strength of the concrete was 30 MPa after 28 days of curing. The measured average cylinder compressive strength of the concrete used for the beams was 33 MPa at the time of testing.

Table 1 shows the material properties of the three types of reinforcement bars used in the experimental program: conventional steel and, two types of glass fiber-reinforced polymer (GFRP), developed by the Korea Institute of Construction Technology (KICT). The surface of the GFRP bar is wrapped with helical glass fiber strands to enhance its bonding characteristics. As for all other FRP bars, GFRP bars present linear elastic behavior up to failure.

3.2 Specimen details and testing

Two of the slabs (SS150 and SS200) were reinforced longitudinally in flexure with 16-mm-diameter deformed steel bars, while the remaining slabs (FS150, FS200, and NFS200) were reinforced with 13 mm diameter GFRP bars. The properties of the test specimens are summarized in Tables 1 and 2. The test slabs were 150 or 200 mm deep and, 1,200 mm wide, with a clear span of 3,600 mm. Each of the slabs was subjected to a four-point-bending load with shear span-to-depth ratios ranging from 5.8 to 8.0.

The reinforcement ratios were chosen such that both under-reinforced and over-reinforced conditions were achieved for each type of FRP reinforcement.

The balanced reinforcement ratios for the FRP-reinforced concrete sections were much lower than those for the steel-reinforced concrete sections. This is due to the higher tensile strength and lower modulus of elasticity of the FRP reinforcements relative to conventional steel. For practical ratios of FRP reinforcement, and to control deflection and cracking, most FRP-reinforced concrete sections are over-reinforced. Regardless of whether the FRP-reinforced concrete section is under-reinforced or over-reinforced, the flexural failure will be a brittle failure. This is because FRP reinforcements do not yield as do steel reinforcements.

Details of the slabs tested in this program are given in Fig.1. and Table 2.

| Specimens  | Dimension (mm) | Reinforcement material | $f_c$ (MPa) | a/d | $\rho$ | $\rho_f$ |
|------------|----------------|------------------------|-------------|-----|-------|-------|
| SS150-8    | 1,200 x 150 x 4,000 | steel                 | 30          | 5.8 | 1.09  | 0.34  |
| FS150-3    | 1,200 x 150 x 4,000 | GFRP (Type1)          | 8.0         | 0.96| 0.24  | 0.51  |
| FS150-6    | 1,200 x 150 x 4,000 | GFRP (Type1)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS150-8    | 1,200 x 150 x 4,000 | GFRP (Type1)          | 8.0         | 0.97| 0.73  | 1.55  |
| FS150-11   | 1,200 x 150 x 4,000 | GFRP (Type1)          | 8.0         | 1.22| 0.97  | 2.06  |
| FS150-14   | 1,200 x 150 x 4,000 | GFRP (Type1)          | 8.0         | 1.22| 0.97  | 2.06  |
| SS200-10   | 1,200 x 200 x 4,000 | steel                 | 30          | 5.8 | 0.96  | 0.30  |
| FS200-4    | 1,200 x 200 x 4,000 | GFRP (Type1)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS200-8    | 1,200 x 200 x 4,000 | GFRP (Type1)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS200-12   | 1,200 x 200 x 4,000 | GFRP (Type1)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS200-16   | 1,200 x 200 x 4,000 | GFRP (Type1)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS200-20   | 1,200 x 200 x 4,000 | GFRP (Type1)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS200-4    | 1,200 x 200 x 4,000 | GFRP (Type2)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS200-8    | 1,200 x 200 x 4,000 | GFRP (Type2)          | 8.0         | 0.73| 0.49  | 1.04  |
| FS200-12   | 1,200 x 200 x 4,000 | GFRP (Type2)          | 8.0         | 0.73| 0.49  | 1.04  |

The parameters considered were the area and type of reinforcement. The objective of the test program was to investigate the performance of FRP-reinforced concrete slabs loaded up to failure. Performance was measured in terms of deflection, crack pattern, ultimate capacity, and mode of failure.

The test slabs were divided into four series: SS, FS150, FS200, and NFS200. The designation of slabs uses the first letter S, F, or NF, referring to the three types of reinforcement used: steel, glass FRP, and new-type glass FRP bars, respectively. The second letter, S, indicates the slab. The numbers 150 and 200 indicate the slab depth, while the last numbers indicate the number of reinforcing bars.

All the tested slabs were fitted with electrical resistance strain gages bonded to reinforcing bars and to the top concrete surface at mid-span.
The tests were conducted under displacement control. At each load increment, crack widths were estimated with a hand-held microscope and the deflection profile of the slabs was measured by LVDTs placed at mid-span. The applied load, displacements, and strain reading were electronically recorded during the test using a data acquisition system monitored by a computer.

4. Experimental Results and Discussion

4.1 General Behavior and Failure

Table 3. reports the values of maximum load \( P_{\text{max}} \) and displacement \( \delta_{\text{max}} \) for the tested slabs, as well as their mode of failure.

As expected, the SS150 and SS200 slabs failed in flexure by tensile yielding of the steel. For GFRP-reinforced slabs, the failure mode depended on the reinforcement ratio and the shear span to effective depth ratio. The GFRP under-reinforced slabs (FS200-4, FS200-8) failed in flexure by FRP rupture.

Fig.3. shows the failure modes of the tested slabs. Slabs reinforced with more than the balanced reinforcement failed in flexural shear because of concrete failure. The failure was due to a combination of crushing and diagonal shear \((a/d=5.8)\). The diagonal shear crack propagated downward to the interface between the GFRP bars and concrete, and then developed into a horizontal shear fracture along the interface toward the edge of the slab, as shown in Fig.3. (b). In contrast, NFS200-12 failed by crushing of the concrete near the load point as shown in Fig.3. (c).

All slabs reinforced with GFRP bars cracked at an early stage. Similar characteristics of crack patterns were observed for the FS and NFS slabs. A crack was initiated in the flexural span between the two concentrated loads where the flexural stress was highest and shear stress was zero. With additional loading, cracking occurred at the constant moment zone when the applied moment exceeded the cracking moment causing a reduction in stiffness. This is due to the wider crack openings in the FRP-reinforced concrete slabs, which is attributed to the low modulus of elasticity of GFRP.

Cracking outside the pure bending zone started similarly to the flexural cracks, but as the load increased, shear stress became more important and induced inclined cracks.

As can be seen, increasing the amount of reinforcement for the same type of reinforcing material increased the post-cracking flexural stiffness.

The crack pattern, propagation, and crack height in FRP-reinforced concrete members were found to be different from those in steel-reinforced concrete members. The initial crack widths in concrete members reinforced with GFRP bars were larger than those in concrete members reinforced with conventional steel.

4.2 Deflection

Fig.4. presents the maximum deflection behavior of the four series of slabs tested. The steel-reinforced slabs of the SS series were characterized by a large plastic deformation due to yielding of the steel.

Due to the lower modulus of elasticity of the FRP, the flexural stiffness in the cracked state was lower, which resulted in a larger deflection. This finding is clearly reflected in the load-deflection curves. Figs.4. (b)-(d) show the load-deflection response of the slabs reinforced with GFRP bars. Initially, the slabs reinforced with GFRP bars were not cracked where they exhibited linear load-deflection behavior. This is attributed to the linear elastic characteristics of GFRP bars and concrete. During the experimental program, the concrete slabs behaved linearly up to
cracking. After cracking, the results showed a bilinear relationship with reduced stiffness.

When comparing the results for slabs FS150, FS200, and NFS200, it can be seen that increasing the reinforcement ratio greatly reduced the deflection after cracking. Slabs reinforced with Type 1 GFRP bars exhibit a significant reduction in stiffness after initiation of the first crack in comparison with beams reinforced with Type 2 GFRP bars. This behavior is attributed to the low elastic modulus of the Type 1 bars compared to that of the Type 2 bars.

### 4.3 Crack width

The crack width in a steel-reinforced concrete structure is an important parameter for measuring structural performance. Unlike steel reinforced concrete structures, the durability of FRP-reinforced concrete structures is reduced with concrete crack width. Fig. 5. shows a comparison of the measured crack width in the reinforced concrete slabs at different load levels and Fig. 6. shows a comparison of the measured crack widths in the reinforced concrete slabs at service load levels. The service load is considered to be approximately 50% of the ultimate load of the specimens. In Figs. 5. and 6., the reinforcement ratio shows a strong influence on the crack width. Higher reinforcement ratios in the slab can sustain the higher tensile force released by the concrete due to cracking. This therefore results in a smaller crack width when compared to crack widths for slabs with lower reinforcement ratios under the same applied loads. As shown in Table 4., the FS 200-8 slab showed a crack width of 2.1 mm under the service load, as compared to 0.7 mm for the FS200-16 slab. Furthermore, Table 4. shows that the GFRP-reinforced slabs generally yield a larger crack width than the identical steel-reinforced slab under the same applied loads.

### Table 4. Failure Mode of the Tested Slabs

| Specimens  | ρ/ρa | Crack width (mm) | Deflection (mm) | NA depth (c, mm) | Load at deflection limit** (kN) |
|------------|------|------------------|-----------------|-----------------|-------------------------------|
| SS150-3    | 0.34 | 0.2              | 10.33           | 49.3            | 47.9                          |
| FS150-3    | 0.55 | 2.1              | 89.6            | 14.1            | 6.8                           |
| FS150-6    | 1.11 | 2.4              | 132.4           | 24.5            | 7.9                           |
| FS150-8    | 1.45 | 2.2              | 102.3           | 18.9            | 14.4                          |
| FS150-11   | 2.00 | 1.9              | 98.4            | 20.4            | 16.5                          |
| FS150-14   | 2.55 | 1.4              | 79.5            | 20.2            | 16.2                          |
| SS200-10   | 0.30 | 0.2              | 10.9            | 75.3            | 96.5                          |
| FS200-4    | 0.51 | 1.1              | 62.4            | 14.8            | 16.5                          |
| FS200-8    | 1.04 | 2.1              | 94.9            | 22.6            | 29.2                          |
| FS200-12   | 1.55 | 1.3              | 74.2            | 17.6            | 37.8                          |
| FS200-16   | 2.06 | 0.7              | 60.1            | 21.8            | 45.4                          |
| FS200-20   | 2.60 | 1.1              | 54.1            | 23.3            | 50.4                          |
| NFS200-8   | 1.44 | 1.9              | 74.4            | 25.5            | 35.6                          |
| NFS200-12  | 1.92 | 1.4              | 65.1            | 25.5            | 35.6                          |
| NFS200-20  | 2.40 | 1.0              | 59.9            | 24.4            | 41.9                          |
| NFS200-12  | 2.89 | 0.9              | 55.49           | 35.7            | 46.5                          |

*Considered to be approximately 50% of the ultimate load of the specimens.
**Defined as L/360, the length of clear span between the supports of the beam, in accordance with KBC (Korean Building Code) 2005.

### 4.4 Strain distribution

Fig. 7. shows the typical behavior of the concrete and GFRP bar slabs. The strains for concrete and GFRP reinforcement remain essentially linear. Furthermore, the increase in the FRP reinforcement ratio decreased the strains measured in both bars.
and concrete. The measured strains in the FRP bars of the two slabs (FS150-6 and FS200-8) that were reinforced equivalent to the balanced reinforcement ratio were approximately 19,000 and 19,800 microstrains, respectively. These values are close to the ultimate strains of the FRP materials. The corresponding compressive strains in concrete for

Fig. 5. Moment Versus Crack Widths of Slabs.

Fig. 6. Crack Widths for Various Service Loads

Fig. 7. Strain Variations in Slab Concrete and Reinforcement
these two slabs were 4,100 and 2,800 microstrains, respectively. However, for the remaining over-reinforced slabs, the measured strains ranged between 14,000 and 19,000 microstrains, and between 16,000 and 19,000 microstrain for Type 1 and Type 2 GFRP bars, respectively. For these slabs, the maximum compressive strains in concrete were 4,000-4,500 microstrains. This indicates a higher strain capacity than the conventional value of 3,000 microstrains. Hence, the test program presented here suggests a strain capacity, \( \varepsilon_{cu} \), of about 4,000 microstrains. More research is needed to investigate whether this value is more widely applicable.

Table 4. summarizes the neutral axis depth at the service load level for all the slabs. The depth of the neutral axis from the compression face of the concrete section, \( x \), can be expressed as:

\[
 x = \left( \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_t} \right) d
\]

where \( \varepsilon_c \) is the compression strain at the top face of the concrete and \( \varepsilon_t \) is the tension strain in the reinforcement.

Table 4. shows that the neutral axis of slabs reinforced with GFRP bars increases quickly compared to the conventional steel-reinforced slabs. This shows an effect of the reinforcement ratio on slab behavior.

### 4.5 Deformability

For steel-reinforced concrete, ductility is quantified by the ratio of displacements or curvatures at ultimate failure to those at the initial yielding of the steel. The ductility of an element can be defined as its ability to sustain inelastic deformation without loss in its load-carrying capacity prior to failure. Because FRP materials do not yield, the term deformability was introduced by researchers as a means of assessing the displacement or curvature that occurs before rupture of the reinforcement. Jaeger et al. (1997) proposed a method to evaluate the ductility of a beam by introducing the following three factors:

\[
\text{Deformability} = \text{Strength factor} \times \text{Deflection factor}
\]

The deformability factor is defined as the ratio of the products of load and deflection at ultimate failure load and at service load.

A permissible value of the deformability factor (DF) ≥ 4 is adopted here for all concrete sections reinforced with FRP bars. As shown in Fig.8., all GFRP-reinforced concrete slabs in this study can be considered safe for design in terms of strength and deformability.

### 4.6 Shear strength

The effect of the reinforcement ratio and modulus of elasticity of the longitudinal reinforcing bars on the shear strength of the tested slabs is shown in Fig.9. The vertical axis represents the experimental shear strength \( V_{exp} \) of the tested slabs, while the horizontal axis represents the reinforcement ratio, \( \rho \). In the NFS series, increasing the reinforcement ratio by 32% (from 0.37 to 0.49%) increased the shear strength by 32%, and increasing the reinforcement ratio by 24% (from 0.49 to 0.61%) increased the shear strength by 17%. These results are similar to the effect of the reinforcement ratio on the shear strength obtained for FS slabs. Fig.9. also indicates that lower modulus of elasticity values in the reinforcing material result in lower concrete shear strength.

Due to the increased use of FRP reinforcements for concrete structures, there are international efforts to develop design guidelines. Past experiments have proven that concrete members reinforced with FRP bars as the main reinforcement and/or shear reinforcement have a considerably lower shear capacity than members with ordinary steel reinforcement. The reduction in shear capacity compared to members with FRP bars as the main reinforcement is probably due to the low modulus of elasticity in FRP. For this reason, equations to calculate the shear capacity of the FRP-reinforced member must be modified to take into account this effect. The specific methods for modification depend on the equations developed for members with ordinary steel reinforcement.

The JSCE Standard Specification and the ACI Building Code specify the use of Eq. 1 to calculate the shear capacity of reinforced concrete linear members \( V \).
with steel reinforcement:

\[ V = V_c + V_s \]  \hspace{1cm} (3)

where, \( V_c \) is the design shear capacity of members without shear reinforcement, and \( V_s \) is the design shear capacity carried by the reinforcing steel. To account for the strength reduction due to the low modulus of elasticity of FRP, the JSCE proposed multiplying \( V_c \) in Eq. 2.21 by \((E_f/E_s)^{1/3}\).

The concrete shear strength recommended by the JSCE is given by the following equation:

\[ V_{c,f} = \frac{130}{1000 + d} \lambda \phi \sqrt{f_{cd} b_d d} \geq 0.08 \lambda \phi \sqrt{f_{cd} b_d d} \]  \hspace{1cm} (13)

Eq. 13 represents the lower bound for the concrete contribution to shear strength of FRP-reinforced concrete members regardless of FRP reinforcing bar type.

In the ACI 440.1R-06 design guidelines, the concrete shear strength shear capacity, \( V_{c,f} \), of flexural members using FRP as the main reinforcement is provided as follows:

\[ V_{c,f} = 0.5 \sqrt{f_{cd} b_d} \]  \hspace{1cm} (14)

where \( b_d \) is the width of the web and \( c \) is the neutral axis depth of the cracked transformed section.

Eq. 14 may be rewritten as Eq. 15. This form of the equation indicates that Eq. 14 is simply the ACI 318-05 shear equation for steel reinforcement, \( V_c \), modified by the factor \((5/2)k\), which accounts for the axial stiffness of the FRP reinforcement:

\[ V_{c,f} = \left( \frac{5}{2} k \right)^{2} \sqrt{f_{cd} b_d} \]  \hspace{1cm} (15)

The shear strengths of the tested FRP slabs were predicted using the shear design provisions of the JSCE, CSA-S806-02, and ACI 440.1R-06 design guidelines. The predicted shear strengths were then compared to the experimental values, as provided in Table 5. and shown in Fig.10.

Both the JSCE and CSA predicted values are in good agreement with the experimental results. This is obvious from Table 5., as the mean value of \( V_{exp}/V_{pred} \) was 1.80 with a standard deviation of 0.62 for the JSCE method, and the mean value of this ratio was 0.87 with a standard deviation of 0.09 for the CSA method. However, the ACI 440.1 method is very conservative; the mean value of the ratio of \( V_{exp}/V_{pred} \) was 2.31 with a standard deviation of 0.29.

Table 5. Comparison of Predicted and Experimental Shear Capacities

| Slab     | Failure load (kN) | Experimental shear strength \( V_{exp}(\text{kN}) \) | JSCE | CAN/CSA-S806-02 | ACI440.1R-06 |
|----------|-------------------|-----------------------------------------------------|------|-----------------|---------------|
| F200-12  | 266               | 133.0                                               | 108.2| 123             | 151.0         |
| F200-16  | 289               | 144.5                                               | 110.0| 121             | 160.0         |
| FFS200-20| 301               | 150.5                                               | 126.4| 117             | 179.2         |
| KNS200-6 | 190               | 95.1                                                | 93.3 | 102             | 137.1         |
| KNS200-8 | 255               | 127.5                                               | 102.4| 117             | 150.5         |
| KNS200-10| 300               | 150.2                                               | 110.2| 136             | 162.0         |
| KNS200-12| 317               | 158.9                                               | 117.0| 136             | 172.0         |
| Mean     | -                 | -                                                  | -    | 1.23            | -             |
| Standard deviation | -       | -                                                  | 0.12 | -               | 0.85          |

Subsequently, for sections with an effective depth greater than 300 mm, with no transverse shear reinforcement or less transverse reinforcement than the minimum required by code, \( V_{c,f} \) is calculated using the following equation:

\[ V_{c,f} = \frac{130}{1000 + d} \lambda \phi \sqrt{f_{cd} b_d d} \geq 0.08 \lambda \phi \sqrt{f_{cd} b_d d} \]  \hspace{1cm} (13)
5. Summary and Conclusions
The measured deflections of 18 simply-supported FRP-reinforced concrete slabs were used to evaluate the deflection of members in bending. Based on the results of this investigation, the following conclusions can be made.

1. The behavior of the tested GFRP-reinforced slabs was bilinear elastic until failure. The stiffness of the slabs reinforced with GFRP bars was significantly reduced after initiation of cracks in comparison to the steel-reinforced slabs. To ensure enough flexural stiffness for deflection control, greater reinforcement ratios are needed.

2. The deflections and strains of concrete slabs reinforced with GFRP bars were generally larger than those for slabs reinforced with steel bars. This was due to the low modulus of elasticity ($E_{GFRP}/E_{steel} \approx 0.22 \sim 0.25$) and the different bonding characteristics of the GFRP reinforcements.

3. The balanced reinforcement ratios for concrete sections reinforced with GFRP bars were much lower than those for sections reinforced with steel. This was due to the low modulus of elasticity ($E_{GFRP}/E_{steel} \approx 0.22 \sim 0.25$) and the different bonding characteristics of the GFRP reinforcements.

4. The balanced reinforcement ratios for concrete sections reinforced with GFRP bars were much lower than those for sections reinforced with steel. For practical ratios of FRP reinforcements and for deflection and cracking control, most of the FRP reinforced-concrete sections will be over-reinforced.

5. Considering that the deformation of FRP is linear elastic and the failure mode of the FRP-reinforced slabs is often governed by concrete crushing, accurate modeling of the concrete stress-strain behavior becomes more important than for steel RC members. Although an ultimate concrete strain of $\varepsilon_{cu} = 0.003$ should be used according to ACI 440, it was shown that for slabs, higher ultimate strains should be applied to predict the ultimate load. Based on the performed tests, a value of $\varepsilon_{cu} = 0.004$ is suggested, although further research is needed to confirm this value.

6. The shear strength of FRP-reinforced concrete slabs can be estimated according to the ACI 440 design guidelines. The results show that the ACI 440 equation is overly conservative in estimating the shear capacity of the beams reinforced with GFRP bars. The JSCE and CSA S806-02 methods accurately predict the concrete shear strength of one-way concrete slabs with GFRP bars as flexural reinforcement.

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